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Day1

Keynote Lecture Ⅰ:

Authenticity of Foundation of Heritage Structures and Special Character of "Angkor Sand" for the Central Tower of Bayon, Angkor Thom, Cambodia

Dr. Yoshinori Iwasaki

Authenticity of Foundation of Heritage Structures and Special Character of "Angkor Sand" for the Central Tower of Bayon, Angkor Thom, Cambodia

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ABSTRACT

The basic principle of conservation of cultural heritage had been considered to keep the original structure with the original material. Authenticity is referred to the quality of being the truth of the heritage. The authenticity has been discussed only for the upper part of heritage structures and the foundation of the structure was neglected. However, the foundation is one of components of the heritage structure and ICOMOS established the concept of the authenticity of foundation in 2010. The restoration work for Pisa Tower started in 1990 and completed in 2001 without any discussion of the authenticity of the foundation. However, the authenticity of the foundation was completely reserved. Main tower of Bayon temple, Angkor Thom, Cambodia was found to have been standing by direct foundation on a sand filled mound with thickness in 15m for more than 700years under monsoon climate condition. The sand used for construction of the mound was found to have very high strength at low water content due to negative high water suction pressure. The high strength of the sand and a unique rain type of "squall" made it possible for the shallow foundation on the thick sand mound to withstand the rainy climate. This paper describes the authenticity of foundation and shows the finding process of the authenticity of foundation of the sand mound in Angkor as an amazing example of "Angkor Sand".

Keywords: authenticity, heritage structure, authenticity of foundation, Pisa Tower, Angkor, Bayon

1 INTRODUCTION

Problems of stability and deformation of heritage structures may be related with soils and foundations. Foundation of heritage structure have been ignored in the past. However, the foundation is one of the structural elements. This paper reviews how the foundation was treated and discuss how to handle in the future.

2 MATSUMOTO CASTLE, JAPAN

The Matsumoto castle was estimated to be constructed during 1592~93. The constructed area was soft soil ground. The Castle tower of 1,000tons of weight was supported by 16 wooden piles of 5m in length as shown in Figure-2. The base of the wooden piles was put upon a raft foundation with horizontal wooden logs cross over each other.

The Matsumoto Castle was designated as a national treasure in 1910 and further upgraded to national treasure in 1915 by Japanese government. Many castles were destroyed during the World War II, The Matsumoto Castle was one of a few historical castle which survived without damage. The last preservation work of the castle had been carried out from 1950 to 1955. Geological boring was performed, and the ground was found soft soil condition.(Matsumoto castle, 2025)



Figure-1 Matsumoto Castle

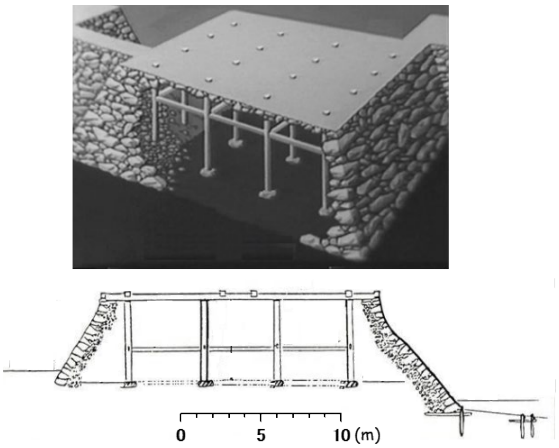


Figure-2 Foundation of Matsumoto Castle

The foundation was replaced by reinforced concrete structures and the authenticity of the foundation was lost.

3 PISA TOWER, ITALY

The Construction work on the Tower began in 1173 (Fig. 3) and completed in about 1370. The inclination of the Tower at the end of the construction was about 2 degrees and gradually increased to about 5.5 degrees in 1990 toward south.

It is nearly 60 m high, and the foundation is 19.6 m in diameter; the weight is 141.8 MN.

The character defining element of the authenticity of Pisa Tower is being inclined without any support. To keep the authenticity of the upper structure of the Tower, any intrusive intervention on the Tower had to be kept to an absolute minimum and permanent stabilization schemes.(Burland et.al, 2003)

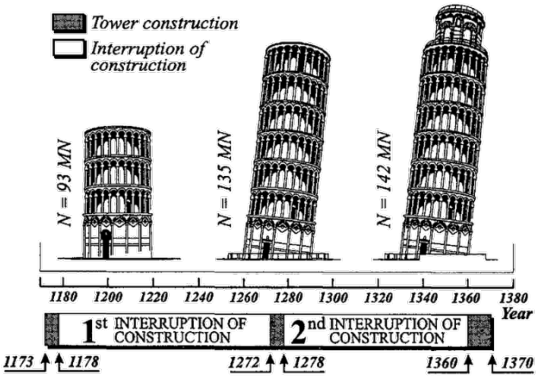


Figure-3 Construction of Pisa Tower

Any disturbance to the ground beneath the south side of the foundation is very dangerous; therefore the use of conventional geotechnical processes at the south side, such as under pinning, grouting etc., involved unacceptable risk. The extraction of soil is finally selected and successfully completed in 2001. The original foundation system was amazingly reserved without discussion of the authenticity of the foundation.

4 AUTHENTICITY OF FOUNDATION

In 2005, when an international standard cord for existing structure, ISO-13822 Bases for design of structures — Assessment of existing structures, was reviewed, the authenticity of foundation was included in the Annex-Heritage Structure in 2010.

Foundation is an important parts of heritage structure and should be assessed as the heritage value.

5 MAIN TOWER OF BAYON, ANGKOR

Bayon temple was built in the late 12th or early 13th century and is located at the center of Angkor Thom. The height of the main tower is about 42m from the ground surface. The tower stands upon thick sand mound of 15m in thickness by shallow direct foundation as shown in Figure-4.

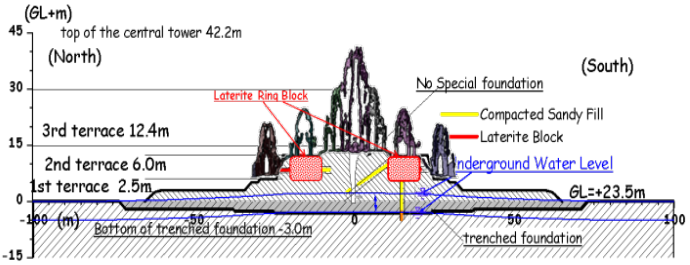


Figure-4 Main Tower of Bayon and foundation

The standard penetration tests of the sand mound show the value of SPT, N=100~200. Sampled soil looks like soft sand rock but collapses under water. The extra high strength is due to negative water pressure by the unique character of the grainsize distribution of the "Angkor Sand." As shown in Figure-5. The unique character of the grainsize distribution resulted in the special capability of standing by the shallow direct foundation on thick sand mound under rainy climate weather in more than 700 year in the past. The grain size distribution is the character defining element of the authenticity of the Angkor Soil.(Iwasaki et_al, 2021)

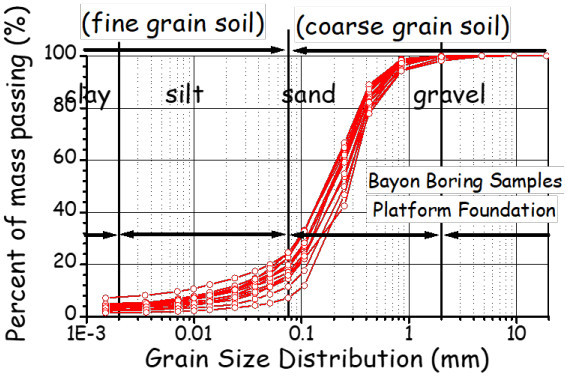


Figure-5 Grain size distribution of Angkor Sand

6 CONCLUSIONS

Geotechnical engineering for preservation heritage has been to consider how to make the upper structure safe in the past. At present and in the future, it needs to discuss the character defining element of the foundation and to maintain the foundation when necessary.

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Keynote Lecture Ⅱ:

Utilization of Geoinformatic database and its application to assessment of settlement of the reclaimed marine foundations due to the construction of Kansai International Airport

Prof. Mamoru Mimura

Utilization of Geoinformatic database and its application to assessment of settlement of the reclaimed marine foundations due to the construction of Kansai International Airport

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ABSTRACT

In order to create a realistic three-dimensional model of the subsoil structure of Kansai International Airport, we propose a method to combine 270 boring data, geological interpretation, and wide-area seismic reflection surveys. In boring surveys, geological interpretation is essential to divide the strata and to connect the same strata laterally. The ground off the coast of Senshu, where Kansai International Airport is located, is composed of a thick clay layer and a thin, discontinuous drainage gravel layer. For this reason, it is meaningful to accurately model the three-dimensional distribution of the gravel layer that controls consolidation. In this paper, a numerical analysis was performed using the multi-dimensional elasto-viscoplastic finite element method with the created foundation model to examine the excess pore water pressure and settlement of the seabed foundation ground due to the construction of the airport island. It is confirmed that the analysis results are able to adequately represent the measured results in each layer, and it is found that the construction of a three-dimensional ground model plays a major role.

Keywords: Geoinformation, ground modeling, finite element analysis, settlement, excess pore water pressure

1 INTRODUCTION

Kansai International Airport was built by reclaiming land off the coast of Senshu, Osaka Bay. From the beginning, the problem was the long-term large subsidence of the thickly deposited alluvial and Pleistocene clayey and gravel soil layers on the seabed. The first and second airport islands are large-scale offshore reclamation covering an area of 4km x 4km, the sea depth is 18m to 20m, and the overburden load due to the reclaimed layer thickness, which is expected to cause subsidence, will be several hundred kN/m^2, which will have an impact on the deep Pleistocene clay layer, especially layers that undergo consolidation yielding, and large consolidation settlement is expected. It was commonly recognized that predicting this behavior was extremely important. The ground survey conducted prior to construction was also required to be wide-area and deep, and a large number of boring surveys were actually conducted in the relevant areas. The columnar section obtained from the boring is very valuable in that it gives the correct value of the soil distribution in the depth direction at that point, but it also has the problem that the information is laterally discrete and there are information gaps between the survey points. When filling a large area such as Kansai International Airport, it is highly necessary to know how the soil classification obtained at a certain point develops laterally, that is, to know the three-dimensional structure of the foundation ground. It goes without saying that the existence of an accurate ground model constructed in this way directly leads to highly accurate deformation prediction. In this paper, based on the case of analyzing the stress deformation behavior associated with the construction of Kansai International Airport, we discuss the role that collaboration between geology and geotechnical engineering, which was implemented at the foundation ground modeling stage, plays in the modeling of the ground.

2 MODELING PROCEDURE OF THE SUBSOIL FOUNDATIONS AT KIX

The samples obtained from the drilling survey are given names of soil types, which are classified as clay, silt, sand, and gravel. A wide-area ground model is created based on multiple discrete borings, and it is necessary to connect layers of the same phase. In the seabed, the layer boundaries become deeper offshore and the layer thickness varies, so the same soil layers do not necessarily exist at the same altitude. Therefore, it is necessary to identify the soil layers that appear in each drilling. For the foundation ground of Kansai International Airport, a detailed geological analysis[1)] was conducted, mainly of the clay layers, and the origin (depositional age) of each clay layer was determined. Although the layer thickness varies at each point, the stratigraphy of the seabed ground representing the Kansai International Airport site is shown in Fig. 1. In this figure, Ma indicates marine clay, and it is known that Ma13, which is deposited on the top seabed surface,

is so-called alluvial clay, and the layer below that is Pleistocene. Here, Ds indicates a sand and gravel layer, and it can be seen that the clay and gravel alternating layers are composed of marine clay layers from Ma13 to Ma6 sandwiched between 10 layers of gravel. Data that provide an elevation and soil type classification as shown in Fig. 1 can be obtained for each of the finite number of discrete borings, and by linking these, a three-dimensional model can be constructed. At this stage, it is necessary to identify the origin of each clay layer, which cannot be achieved with geotechnical engineering knowledge alone. Geological analysis was also carried out in the study of the foundation ground for Kansai International Airport, making it possible to develop each layer laterally. shown in Fig. 1 can be obtained for each of the finite number of discrete borings, and by linking these, a three-dimensional model can be constructed. At this stage, it is necessary to identify the origin of each clay layer, which cannot be achieved with geotechnical engineering knowledge alone. Geological analysis was also carried out in the study of the foundation ground for Kansai International Airport, making it possible to develop each layer laterally.

Next, to confirm the horizontal distribution of each layer, a seismic reflection survey was carried out on the airport island and the surrounding sea area.[2)] This enabled the distribution characteristics (extent and thickness) of Ds1 to Ds10 to be understood by checking

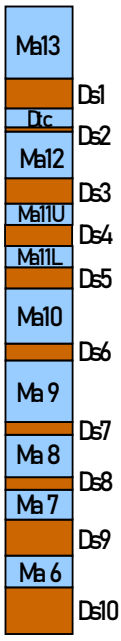


Fig.1 Representative stratigraphy of the foundation ground of Kansai International Airport

the reflection surfaces of each gravel layer in particular. This can be seen in Figure 2. On the other hand, although gravel can be seen in part of the first airport island in the Pleistocene Ds6, it is clear that the grain size is significantly finer (green) toward the offshore, and areas where no gravel is present extend both offshore and on the land side. The soil properties of the two gravel layers Ds1 and Ds6 are quite different, and it is expected that Ds6 will have worse permeability than Ds1.

3 ASSESSMENT OF SETTLEMENT USING FINITE ELEMENT ANALYSIS

Based on the boring data and sonic exploration results, a three-dimensional ground model of the foundation ground of Kansai International Airport was created2). Based on this ground model, Fig. 3 shows a cross-sectional two-dimensional ground model created along a measurement line that crosses the first and second airport islands, passing through the settlement and water pressure measurement point (labeled Monitoring point 1) for each layer on the first airport island. As shown in the figure, it is possible to see that each layer increases in depth toward the offshore, and the thickness of each layer changes. When performing deformation analysis using the ground model with the finite element method, the soil parameters for the clay layer, which is responsible for compression, are set based on soil tests using samples with little disturbance. On the other hand, the most important constant for the gravel layer is permeability. As shown in Fig. 2, the gravel layer, which is responsible for drainage, is not distributed thick and uniform in the foundation ground

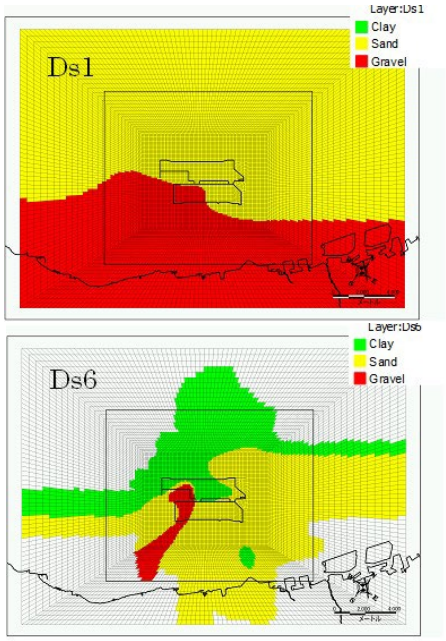


Fig.2 Distribution of permeable layers and their soil classification

of Kansai International Airport, and the layer thickness and continuity are heterogeneous and partially disappear, so there was a difficult problem that the drainage capacity according to the permeability

coefficient required in the indoor permeability test could not be guaranteed.

A series of elasto-viscplastic finite element analyses were conducted on the mesh shown in Fig. 3. Figure 4 shows the comparison of the calculated and measured results of settlement at various subsoil layers, such as the Holocene as well as the Pleistocene clays. Good agreement can be seen between the calculated and measured settlement results as shown in Fig. 4. It is found that the modeling and calculation in this particular case are successfully done.

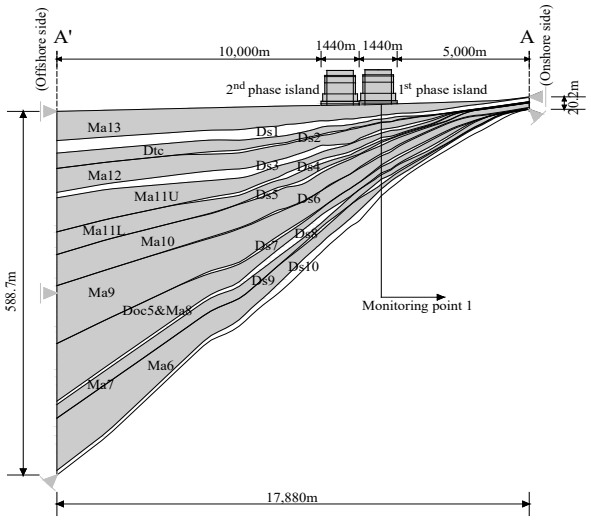


Fig. 3 Exemplified cross-section of the foundations of Kansai International Airport

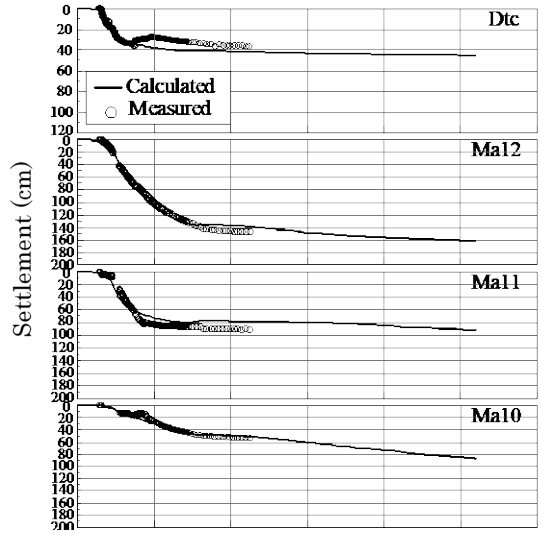


Fig. 4 Comparison of calculated and measured settlement of the clay layers

4 CONCLUSIONS

The modeling of the foundation ground of Kansai International Airport was used as an example to explain the necessity and effectiveness of collaboration between geology and geotechnical engineering. By combining knowledge from geological analysis and geophysical exploration with geotechnical engineering, it is possible to create a well-founded ground model, which in turn improves the reliability of the resulting numerical analysis results.

A series of numerical analyses was performed using the multi-dimensional elasto-viscoplastic finite element method with the created foundation model to examine the excess pore water pressure and settlement of the seabed foundation ground due to the construction of the airport island. It is confirmed that the calculated performance is able to adequately describe the measured results in each clay layer,

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Keynote Lecture Ⅲ:

Geological application of computerized geotechnical borehole database Subsurface geology and sedimental environment in Kansai Area

Dr. Naoko Kitada

Geological application of computerized geotechnical borehole database Subsurface geology and sedimental environment in Kansai Area

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ABSTRACT

The Kansai Geo-informatics Network holds digitized geological information, physical properties, and soil test results obtained from borehole surveys in the Kansai region. In addition, it also borehole drilling survey data that has undergone physical exploration and analysis, including detailed sedimentological characteristics and depositional age information, allowing for the study of the region's ground environment and sediment characteristics. With over 80,000 pieces of data, it is possible to study the surface ground characteristics and geological sedimentary environment of not only the Osaka Plain, but also the Kansai region, including the Kyoto Basin, Nara Basin, Wakayama, Omi Basin, and Kobe.

The collected borehole information is not only used to study the regional ground characteristics and understand the associated geological risks, but also to create maps predicting liquefaction damage during earthquakes and to forecast earthquake damage, etc. In this presentation, we will introduce the regional ground characteristics that have been revealed using the borehole data.

Keywords: borehole data, KG-NET, sedimentary environment, liquefaction

1 VARIATIONS IN THE STUDY AREA

The area around Osaka is home to many active faults, and because of tectonic movements along the faults, mountainous areas have developed in areas of uplift and lowlands have developed in areas of subsidence. In the bay area surrounding Osaka Bay, alluvial fans have developed in the Kobe area, where faults are close to the coastline, and their ends directly form the coastline.

On the other hand, in central Osaka, lowlands formed by the floodplains of large rivers are widely distributed. The Kyoto Basin and Nara Basin, which correspond to the inland area, are lowlands formed between mountain areas formed by faults.

On the other hand, glacial and interglacial periods repeat periodically. During glacial periods, precipitation decreases, and river flow is low, so mountain gravel is not transported. Sea levels fall and land expands. During interglacial periods, precipitation increases, rivers carry more gravel, and at the same time, sea levels rise, and lowlands sink into the sea.

A combination of tectonic and climatic changes creates regional variations in landforms and lowland deposits. Furthermore, if the geological composition of mountain areas is different, the diversity is even greater. In this paper, we discuss the sedimentary characteristics of the region using the Osaka Plain and the inland Kyoto Basin as examples.

2 BAY AREA AND OSAKA PLAIN

The most remarkable feature of the Osaka Plain is the presence of a narrow upland called “Uemachi upland.” It is about 20 m above sea level at its highest elevation, and it extends up to the center of the Osaka Plain in a north–south orientation. This upland divides the Osaka Plain into two parts called the West Osaka Plain in the West and Kawachi Plain in the East. The Uemachi fault traverses through the middle of the Osaka Plain on the eastern edge of the West Osaka Plain (Fig.1). The Uemachi fault system lies in a N–S direction and is more than 40 km in length. The main part of the Uemachi Fault

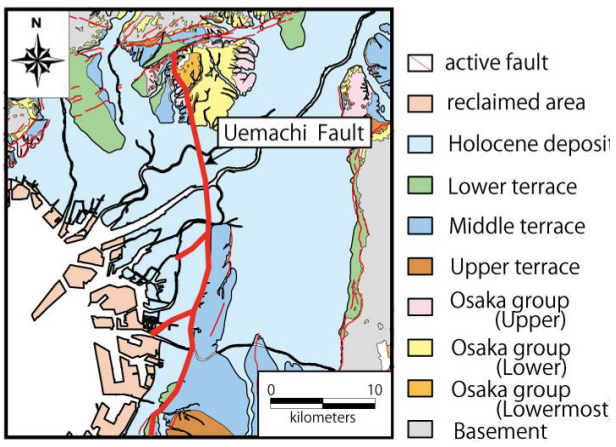


Fig. 1 Geological and tectonic map of area around Osaka Plain (Active fault map in urban area, 1996)

is subparallel to the Uemachi upland. Areas where the coastline repeatedly advances and retreats due to crustal movement

The most remarkable feature of the Osaka Plain is the presence of a narrow upland called "Uemachi upland." It is about 20 m above sea level at its highest elevation, and it extends up to the center of the Osaka Plain in a north–south orientation. This upland divides the Osaka Plain into two parts called the West Osaka Plain in the West and Kawachi Plain in the East. The Uemachi fault traverses through the middle of the Osaka Plain on the eastern edge of the West Osaka Plain (Fig.1). The Uemachi fault system lies in a N–S direction and is more than 40 km in length. The main part of the Uemachi Fault is subparallel to the Uemachi upland. Areas where the coastline repeatedly advances and retreats due to crustal movement and climate change are characterized by warm-period marine clay layers. Marine clay layers are widely distributed in lowlands. Figure 1 shows the distribution map of the bottom elevation of the Holocene marine clay layer (Ma13) distributed in the Osaka Plain. and climate change are characterized by warm-period marine clay layers. Marine clay layers are widely distributed in lowlands. Figure 1 shows the distribution map of the bottom elevation of the Holocene marine clay layer (Ma13) distributed in the Osaka Plain.

Pliocene to Quaternary sediment "Osaka Group" and terrace sediment are found to be deposited in the Osaka Plain. These are covered with Holocene deposits in the lower plain and around the Osaka bay area. These deposits include 15 layers of marine clay; referred to as Ma-1, Ma0, and Ma1 to Ma13, caused by changes in the sea level. Ma12 and Ma13 are Late Pleistocene and Holocene sediments, respectively, and are distributed near the surface. Fig.2 shows the distribution of bottom elevation of Ma13. Marine clay layer (Ma13) covers most of the Osaka Plain.

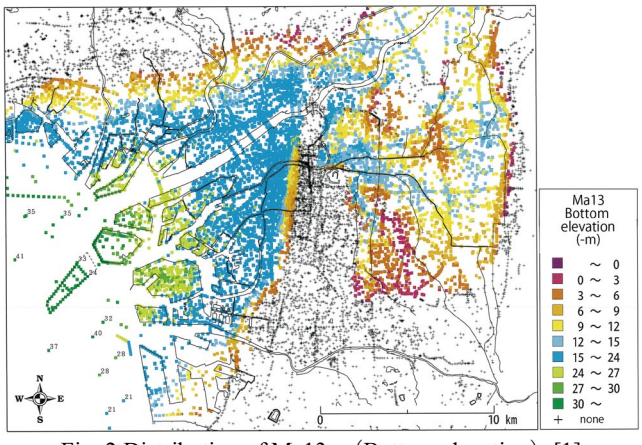


Fig. 2 Distribution of Ma13 （Bottom elevation） [1]

3 SURFACE GEOLOGY OF INLAND BASINS

The Kyoto Basin is surrounded by mountains and has active faults running from east to west. The basin subsides and develops into an alluvial fan from north to south, with the deepest area being along the Uji River, creating a topography that is just like opening a book. This area does not experience marine intrusion during warm periods, so Holocene clay layers (marine clay) are not present. However, since a thick gravel layer was formed before the warm period of the Holocene, it is possible to classify the depositional period using the distribution of this gravel layer.

Fig. 3 shows a north-south cross section of the Kyoto Basin. The thick layer of gravel shown in red is Pleistocene deposits.

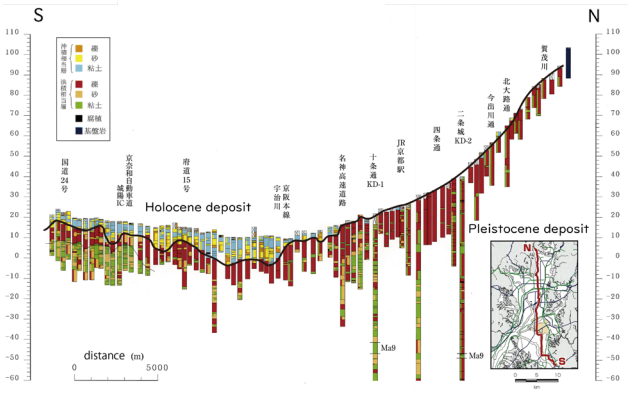


Fig. 3 NS cross section of the Kyoto Basin.[2]

Fig. 3 shows a north-south cross section of the Kyoto Basin. The thick layer of gravel shown in red is Pleistocene deposits. The block in which Holocene deposits can be seen from near the center to the south is thought to have been formed as a result of movement of an active fault called the Ujigawa Fault (southward drop) at the boundary with the block to the north.

4 CONCLUSIONS

By collecting drilling data and examining them in a dense manner, we can understand the characteristics of the region's ground and structural movements. In such an attempt, it is necessary to connect the characteristic deposits of the region as key layers laterally, and we believe that by examining each drilling result together with a geological sense and a geotechnical sense, it will be useful for identifying geological risks.

ACKNOWKEDGMENT

This research is the result of studies conducted within KG-NET: Geotechnical Research Group using drilling information. I would like to express our gratitude here.

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Keynote Lecture IV:

Risk assessment of rainfall-induced landslides considering regional characteristics with machine learning

Prof. Tatsuya Ishikawa, Ren Ono
, Tomoki Ueno, Kiyonobu Kasama

Risk assessment of rainfall-induced landslides considering regional characteristics with machine learning

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ABSTRACT

In Japan, the risk assessment of rainfall-induced landslide disasters is based on the risk base line of sediment disasters occurrence (CL: Critical Line) set for each 1 km mesh using two values: the 60-minute accumulated rainfall (RI: Rainfall Intensity), a short-term rainfall index, and the soil precipitation index (SWI: Soil water Index), a long-term rainfall index. However, the current Japanese early warning system is set without considering the ground and topographical characteristics that are the endogenous factors of slope failures, and it assumes debris flows and large-scale landslides that occur intensively and does not target small-scale landslides such as cut slope collapse and embankment collapse. This study aims to develop a wide-area slope disaster risk assessment method for small-scale landslides that occur locally along roads by proposing a method for setting slope disaster risk base line that can be applied to various regions with the machine learning based on the numerical data representing regional characteristics, including topography and geology, in addition to meteorological data such as temperature and precipitation.

Keywords: slope failure, heavy rainfall, early warning system, machine learning

1 INTRODUCTION

At the current Japanese Early Warning System (JEWS, Osanai et al. 2010), the risk assessment of rainfall-induced landslide disasters is judged based on the risk base line of sediment disasters occurrence (CL: Critical Line), which is set for each 1 km mesh using two values: the 60-minute accumulated rainfall (RI: Rainfall Intensity) as a short-term rainfall index, and the soil precipitation index (SWI: Soil Water Index, Okada 2001) as a long-term rainfall index. In Hokkaido, the north island of Japan, the "Hokkaido Sediment Disaster Warning Information System (HSDWIS, Hokkaido 2009)", which is an adjusted version of the JEWS to suit local conditions, has been operated by the local government since 2008. However, the current CL settings do not reflect the ground characteristics that predispose to the occurrence of landslides, and the RBFN (Radial Basis Function Network) values for CL settings are set uniformly for each municipality without considering the topographical characteristics, resulting in a very high strikeout rate of 96% (Matsuoka and Tsurumaki 2021) for sediment disaster warning information and low prediction accuracy. Besides, since the CL currently adopted in the HSDWIS (hereinafter referred to as "Basic CL (BCL)") is primarily designed to predict the occurrence of sediment disasters like large-scale landslides and debris flows that would occur intensively and require the evacuation of residents, thus making it difficult to apply to the prediction for small-scale landslides like cut slope collapse and embankment collapse that occur locally along roads, and then unsuitable for managing wide-area road networks. In addition, the BCLs are set only for 1 km meshes that are judged to be necessary based on hazardous areas, topography, land use, etc., which is about 30681 out of 103956 meshes for the entire Hokkaido prefecture. In this case, mountainous and mountainous areas that are at high risk of landslides but do not require evacuation action of residents are often excluded from the target areas. Considering these points, the CL of the JEWS is not suitable for managing wide-area road networks.

The objective of this study is to develop a wide-area slope disaster risk assessment method for small-scale landslides as well as large-scale landslides by proposing a method for setting slope disaster risk base line (CL) that can be applied to various regions with the machine learning based on the numerical data representing regional characteristics, including topography and geology, in addition to meteorological data such as temperature and precipitation. The wide-area slope disaster risk assessment method could be employed for different purposes, such as road management and evacuation warnings, by appropriately selecting the CL for small-scale and large-scale landslides.

2 SUMMARY

This study developed a wide-area slope disaster risk assessment method for two types of different scale landslides by proposing a method for setting slope disaster risk base line that can be applied to various regions with the machine learning based on the regional characteristics such as topography and geology as well as the meteorological data. The following findings can be mainly obtained:

- Based on regional characteristics such as temperature, precipitation, topography, and geology for tertiary meshes (1 km meshes) in Hokkaido, a machine learning (ML) model was created to estimate two kinds of rainfall index values, namely RI_{cross} and SWI_{max} (Fig. 1), on the critical line (CL) which can properly predict past slope disasters. The prediction accuracy of the created ML model was verified based on R^2, RMSE, scatter plots, and SHAP values by comparing the observed and predicted values of both indexes, and it was shown that it could predict the reference CL (RCL) when a landslide occurs with sufficient accuracy for practical use.

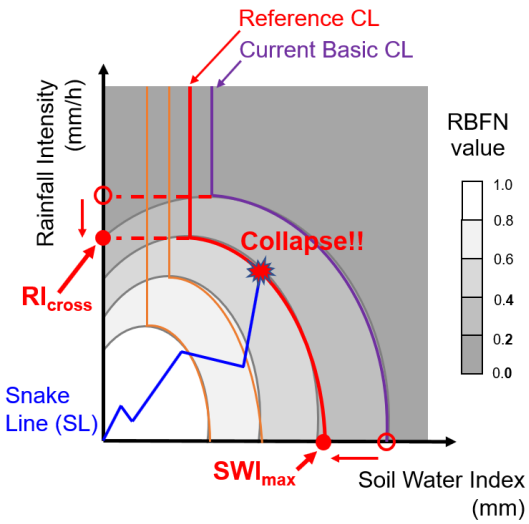


Fig. 1. Prediction points on CL for ML models.

- The created ML model was used to predict the two rainfall index values RI_{cross} and SWI_{max} for all tertiary meshes in Hokkaido, and a Slope Disaster Risk Map for the entire Hokkaido prefecture was created based on the distribution of the estimated rainfall index values (Fig. 2). To verify the reliability of the created slope disaster risk map, it was compared with a Landslide Disaster Probability Map created by the NILIM taking into account regional characteristics, and it was confirmed that the areas evaluated as having a high slope disaster risk on both maps were generally consistent.
- These results show that the ML model created using regional characteristics such as temperature, precipitation, topography, and geology of each tertiary mesh can appropriately improve the prediction accuracy of slope disasters, and that the ML model can be used to evaluate the risk of slope disasters throughout Hokkaido, including areas where the BCL has not been set. Besides, the wide-area slope disaster risk assessment method developed using the ML model is capable of evaluating the risk of slope disaster toward two types of landslides with different scales by properly selecting the basic CL and the reference CL, which demonstrate the usefulness for both the road management and the provision of early warning information in snowy cold regions.

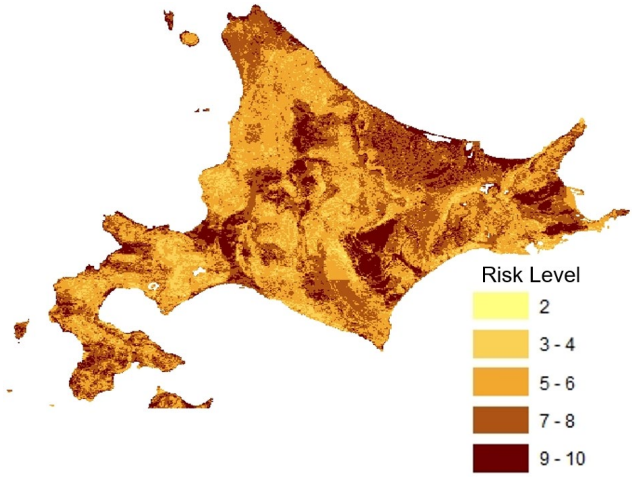


Fig. 2. Slope Disaster Risk Map (SDRM) for the entire Hokkaido prefecture using the ML model.

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Keynote Lecture V:

Development and application of geological and geotechnical information databases in Japan and the ATC10's challenge on uncertainties

Mrs. Rie Wada

Development and application of geological and geotechnical information databases in Japan and the AsRTC10's challenge on uncertainties

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ABSTRACT

The current state of the development and application of geological and geotechnical information databases in Japan was outlined. Various types of databases have been opened to the public, including borehole and soil test data, thematic maps, point cloud data, 3D geological models, and digital twins of cities, as well as the APIs for these data. These databases are used to create hazard maps, determine the causes of disasters, and improve the accuracy of geological surveys for construction projects. However, it is crucial to clarify and evaluate uncertainties in data and models, as well as define accuracy and resolution parameters for geological and geotechnical models when applying data in Building Information Modeling and Digital Twin.

Keywords: geological database, geotechnical database, uncertainty, Building Information modeling, Digital Twin

1 INTRODUCTION

As part of the promotion of digital transformation (DX), various types of geological and geotechnical data obtained through infrastructure development, including roads, bridges, and earthworks, have been made available to the public in Japan. Furthermore, Building Information Modeling (BIM) has been mandatory for public design and construction works since 2023 (MLIT, 2024a) and is also recommended for investigations. This paper provides an overview of the current state of, and challenges in the developing and applying these data, as well as the activities in this area driven by the ISSMGE Asian Regional Technical Committee 10 (AsRTC10), which focuses on urban geo-informatics.

2 THE STATUS OF GEOTECHNICAL INFORMATION DATABASE DEVELOPMENT AND APPLICATION IN JAPAN

The website of the Japanese Ministry of Land, Infrastructure, Transport and Tourism (MLIT) provides not only borehole and soil test data, but also various types of data, including thematic maps, point cloud data, and digital twins of cities (MLIT, 2023a). Since the APIs for the data are also publicly available (MLIT, 2023b), we can utilize and analyze them in conjunction with private data. In addition, 3D geological models can be browsed for some cities (IGG of AIST,2022). Fig.1 and Fig.2 show the digital twin (MLIT, 2024b) and the 3D geological model of Tokyo area (IGG of AIST,2022), respectively. Fig.3 shows an overlay of borehole data

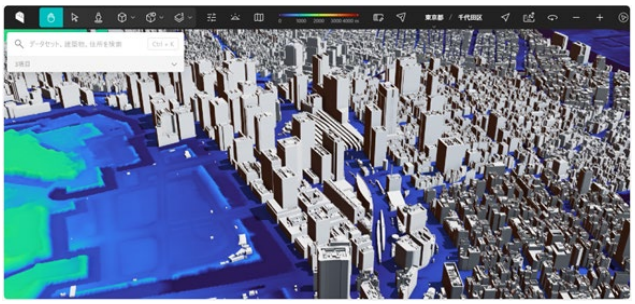


Fig.1 A screenshot of the digital twin of central Tokyo. The background map shows elevation (MLIT, 2024b).

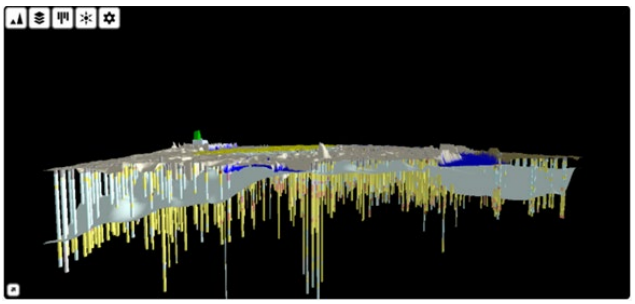


Fig.2 A 3D geological model of central Tokyo, height is multiplied by x10 (IGG of AIST, 2022).

and an aerial photograph of the Noto Peninsula, where the 2024 Noto Peninsula Earthquake occurred on January 1, 2024. In this area, immediately after the earthquake, almost within a week, a lot of data owned by several public sectors was made available (National Geo-Information Center, 2024) and used by many geotechnical engineers and researchers for damage

assessment and rapid recovery.

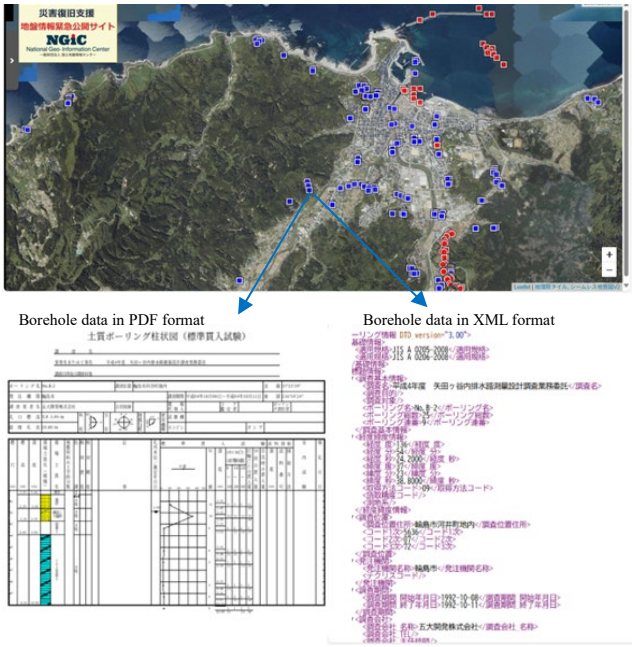


Fig.3 Borehole location map of the area damaged by the Noto Peninsula earthquake. Red marks are national data, blue are data of local governments. The background is an aerial photograph (National Geo-Information Center, 2024).

In this way, open data is useful for many applications, like creating hazard maps, analyzing the causes of disasters, and improving the accuracy of geological surveys for construction projects.

3 CHALLENGES IN APPLYING GEOTECHNICAL INFORMATION IN PRACTICE

Although a variety of geological/geotechnical data is now available online, there are several issues that must be addressed when using this data. This chapter discusses two primary issues: the uncertainties in the data and models, and the concept of model accuracy and resolution in BIM.

3.1 Evaluation of the uncertainties in data and models

Geological/geotechnical data and 2D/3D models created based on this data must contain uncertainties due to the factors including the equipment specifications, the amount and type of data, and engineer interpretation. Underestimating these uncertainties could lead to consequences such as construction accidents and cost overruns.

Fig.4 shows a photograph of a road collapse that occurred during the construction of an urban subway tunnel in 2016, as well as two geological cross sections: one section was interpreted based on 9 borehole data obtained prior to planning, and the other section was interpreted based on 23 borehole data, some of which were drilled after the accident (PWRI, 2017). The overburden from the tunnel crown was designed to be in an impermeable layer consisting of weathered sedimentary rock called the D2 layer. An investigation after the accident revealed that the thickness of the D2 layer was thinner and more inhomogeneous than estimated. Furthermore, this layer was found to be more fragile due to microcracks, faults, and weathering. According to the report on this accident (PWRI, 2017), it was thought to be caused by several factors, including complicated geological and geotechnical conditions, a lot of uncertainties due to insufficient data for modeling, and an underestimation of uncertainty by designers and contractors lacking geological/geotechnical knowledge.

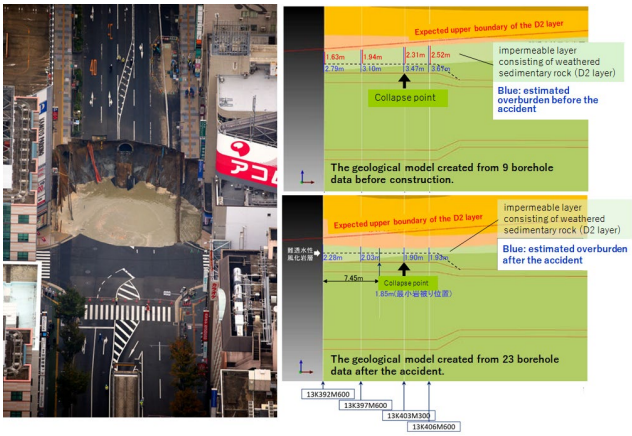


Fig.4 A photo and geological sections of a road collapse occurred during the construction of an urban subway tunnel (PWRI, 2017).

To prevent this type of accident, we need a modeling scheme to build models based on sufficient data including open data and business workflows that enable us to share information about the uncertainties in data and models. Fig.5 shows the classification system of uncertainties considered by AsRTC10(AsRTC-WG1,2017). Stakeholders should share information about these parameters on uncertainties throughout the infrastructure's life cycle.

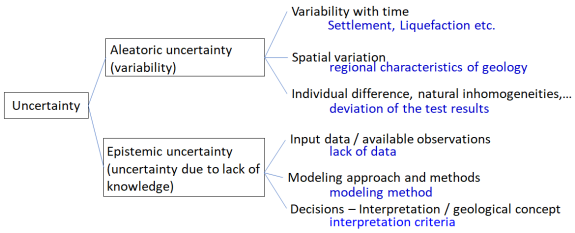


Fig.5 A typology of uncertainties (AsRTC10-WG1, 2017).

3.2 Definition and implementation of the LOIN, LOD and LOI for geological/geotechnical models in BIM

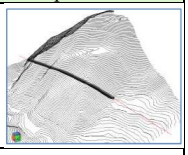
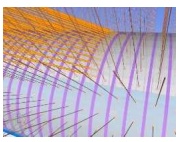
According to ISO 19650, Building Information Modeling (BIM) is defined as "the use of a shared digital representation of a built asset to facilitate design,

construction, and operation processes to form a reliable basis for decisions"(ISO 19650-1, 2018). BIM could enable us to share information about uncertainties in geological/geotechnical models, as well as business risks related to geological/geotechnical conditions, because all the stakeholders can share information and communicate with each other via a Common Digital Environment (CDE) throughout a project's lifecycle.

On the other hand, the Level of Information Need (LOIN), the Level of Detail (LOD), and the Level of Information (LOI) must be explicitly defined for geological/geotechnical models, because these models would be required to be integrated with building and infrastructure models in BIM. The LOIN is defined as "the level of detail and information required to support a specific decision or process at a particular point in the project lifecycle". The LOD refers to the geometric granularity of the information, while the LOI refers to the non-geometric data and documentation (ISO 19650-1, 2018).

Table 1 shows an example of the LOD for mountain tunnel construction (MLIT, 2022). Both the LOD and the LOI evolve over time. Regarding the LOD, geometric granularity such as size, shape, and location becomes more precise and detailed as the construction phase proceeds. Regarding the LOI, the type and volume of information, such as attributes and properties, increases as the construction phase progresses.

Table 1. An example of the LOD for mountain tunnel construction, outlined based on Table 4 in the draft guidelines for 3D model deliverables (MLIT, 2022).

LOD	Definition	Example
100	/A model of an object with symbols, lines, or simple shapes indicating its location. /A rectangular shape or linear model which is adequate for showing the layout of the tunnel.	
200	/A model with an adequate shape to confirm the structural type. /Model with the center alignment of the planned road and the standard cross-sections of the tunnel. /Tunnel mouth sections are not modeled, but their locations are indicated.	
300	/Model with accurate geometry of main structure. /The shapes of the widened portion of the evacuation passageway, etc. are modeled. The mine entrance should be accurately modeled. /The support pattern and auxiliary methods should be shown using symbols. /The pavement configuration and internal facilities such as drainage are modeled. /The location of the box cutout should be modeled in a pattern and the installation should be indicated by symbols, etc.	
400	/An accurate model created by adding detailed structures such as rock bolts and reinforcement into the LOD 300 model. /It generally represents the geometry of the tunnel body, tunnel mouth and box cutout sections including the reinforcement, interior slabs, and support patterns.	
500	/A model that represents the shape of reality, made by adding the completed version into the model utilized in the design and construction phases.	—

The characteristics of geological/geotechnical models, which are interpreted by engineers and contain uncertainties, make it difficult to define explicit and unique LOD and LOI for them. However, since disaster prediction, such as seismological simulations using digital twins with BIM models will provide more precise and accurate results. Therefore, we must solve this challenge as soon as possible.

4 ASRTC10'S APPROACH TO THE CHALLENGES

The AsRTC10 has studied the evaluation and reduction of uncertainties included in geological/geotechnical data and models.

Specifically, we have reviewed the data and models from past real projects, including mountain tunnels, a dam in a complex geological setting, and a high-rise building on thick alluvial mud layers. We identified the uncertainties and their sources and examined how the projects addressed them. Finally, we proposed properties related to uncertainties that should be shared by all the stakeholders.

To clarify and evaluate the uncertainties in the data and models, we are currently tackling three challenges. One challenge involves the utility of databases of geophysical investigation and sounding results to improve model reliability. Another involves searching for and reviewing guidelines from other countries that describe LOD and LOI for reference. The third challenge involves using machine learning to evaluate uncertainties.

5 CONCLUSIONS

This paper reviews the development and application of geological/geotechnical information databases in Japan, as well as the challenges related to the uncertainties in data and models, particularly in BIM. Based on the Japanese government's data-driven innovation policy, the quantity and type of available geological/geotechnical information will increase. This will create more opportunities for non-experts in these fields to utilize the data. It is crucial to reveal the uncertainties in the data and models to prevent improper application, accidents, and other issues. AsRTC10's activities will contribute to this effort.

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Invited Lecture I:

Spatial and Temporal Analysis of Landslide Susceptibility– for the Case in Taiwan

Prof. Keh-Jian Shou, SY. Sung

and PL.Chen

Spatial and Temporal Analysis of Landslide Susceptibility– for the Case in Taiwan

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ABSTRACT

Due to the impact of climate change, the increasing occurrence of extreme rainfall events, with concentrated rainfalls, commonly cause landslide hazard in the mountain areas of Taiwan. However, there are uncertainties for the predicted rainfall as well as the landslide susceptibility analysis.

This study employs machine learning approached, including the logistic regression method LR to analyze the landslide susceptibilities. Together with the predicted temporal rainfall, the predictive analysis of landslide susceptibility was performed in the adopted study area in Central Taiwan. The uncertainty of the rainfall prediction was firstly investigated before applied to the landslide susceptibility analysis. The uncertainty of the landslide susceptibility mode was further investigated to increase the accuracy of landslide susceptibility analysis. The results of landslide susceptibility predicative analysis, with the discussions on the accuracy and uncertainties, was performed and can be applied for the landslide hazard management.

Keywords: landslide susceptibilities, logistic regression method, landslide disaster resilience.

1 INTRODUCTION

The mountain areas of Taiwan are prone to landside hazard due to steep topography and intense rainfalls. Landslide susceptibility analysis is crucial for disaster prevention and early warning. Most existing studies focus on comparing different statistical or machine learning methods. However, there are very few exploring the uncertainties of the landslide susceptibility methods.

This study adopted landslides triggered by Typhoon Mindulle, which had a significant impact on the mountain areas in Central Taiwan, as the training samples. For predictive analysis, the performance was validated using data from Typhoon Morakot in 2009. Logistic Regression model was established. The accuracy of the model was investigated under different conditions, and was evaluated using the ROC curve and AUC values.

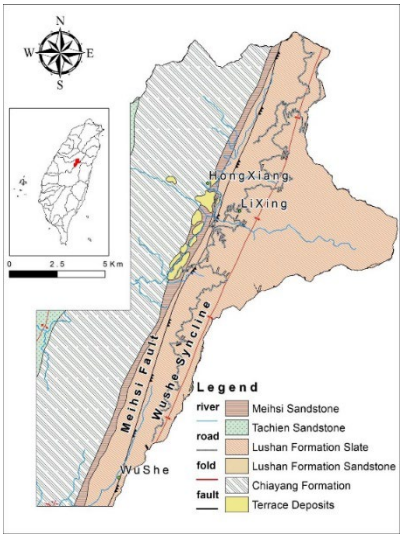


Fig. 1. The geology of the study area.

2 LANDSLIDE INTERPRETATION AND FACTOR LAYERS

In this study, ArcGIS was used to process SPOT imagery from the Typhoon Mindulle event into layers with three different grid sizes. These layers were overlaid with the new-born landslide layer from the same

event. A grid cell was considered a landslide sample point if the landslide area within the cell exceeded half of the grid cell's area. This method was used to generate the number of landslide sample points corresponding to each grid size for analysis.

According to the results, the number of landslide sample points for grid sizes of 5×5 m, 10×10 m, and 20×20 m were 79,999, 19,975, and 5,039 points, respectively. These findings indicate that smaller grid sizes not only allow for a more accurate representation of landslide shapes, but also enable the sampling of smaller landslide areas.

Nine control factors were selected for assessing the landslide susceptibility analysis in the study, including F1 the distance to road, F2 the distance to RIVER, F3 the distance to fault, F4 elevation, F5 aspect, F6 slope, F7 NDVI, F8 IDS, and F9 the landslide rainfall index. In order to examine the impact of grid size on landslide susceptibility analysis, three grid sizes of 5m x 5m, 10m x 10m, and 20m x 20m of Elevation, slope and NDVI were used. Also, two different distance-to-fault factors were produced, using intervals of 100 meters and 500 meters, respectively

3 RAINFALL INDEX FOR SLIDING, ID

This study adopts the Rainfall index for sliding as the rainfall factor. The index is derived by extracting the maximum hourly rainfall and cumulative rainfall from landslide sample points during specific events. These data are then used to plot a distribution chart in Excel, with the maximum hourly rainfall on the x-axis and the cumulative rainfall on the y-axis.

The upper-rightmost and lower-leftmost points define the boundary of the landslide rainfall distribution plot. The midpoint of this boundary range is then used to determine the slope of the upper and lower threshold lines. By translating this slope to the upper-right and lower-left corners of the boundary, the upper threshold line L1 and the lower threshold line L2 are obtained.

Table 1. L1 and L2 of different grid size of Typhoon Mindulle

Grid Size：20m*20m	
L1	y=-6.0763+1811.121
L2	y=-6.0763+1108.831
Grid Size：10m*10m	
L1	y=-6.0939+1813.041
L2	y=-6.0939+1108.295
Grid Size：5m*5m	
L1	y=-6.0825+1811.881
L2	y=-6.0825+1106.498

Then, the maximum hourly rainfall and cumulative rainfall are substituted into Equation 3 as x and y, to calculate their distances d1 and d2 from the upper and lower threshold lines.

$$d_1 \& d_2 = \left| \frac{ax_0 + by_0 + c}{\sqrt{a^2 + b^2}} \right| \quad (3)$$

Finally, the values of d1 and d2 are substituted into Equation 4 to obtain the distribution map of the Rainfall index for sliding (Id), with values ranging between 0 and 1.

$$Id = \frac{\mathrm{d2}}{d1 + d2} \quad (4)$$

4 LANDSLIDE SUSCEPTIBILITY ANALYSIS

Different susceptibility models were constructed based on combinations of grid sizes and image resolutions, all using the same fault proximity interval. The table presents the factor weights for each model.

1. Slope Factor Variation – In steeper areas and smaller grid sizes, terrain details are described more accurately, and small-scale landslides can be detected, resulting in an increased weight for the slope factor.

2. Elevation Factor Variation with Different Resolutions – Differences in image resolution impact the representation of elevation, influencing factor weights.

3. High-Resolution Images Significantly Improve Model Construction Accuracy – AUC analysis shows that finer resolution generally leads to better training performance.

4. At 20m Resolution and 5m Grid Size – The lowest accuracy was observed. While small grids capture terrain detail better, low image resolution cannot adequately describe vegetation conditions, making NDVI overly dominant. The mismatch between NDVI input quality and terrain detail causes a drop in accuracy.

5. At 10m Resolution and 5m Grid Size – The highest model accuracy was achieved. This suggests that a 5m grid size is more sensitive to image resolution and benefits from medium-high image quality.

6. Overall Comparison – The model built with 20m resolution showed the highest AUC value in some cases, but as grid size decreases, over-detailed terrain description can overcomplicate the model, reducing its accuracy during construction.

Table 4. Weights and AUC of Susceptibility Analysis Factors Under Different Conditions

Grid Size	20*20	10*10	5*5	20*20	10*10	5*5	20*20	10*10	5*5	20*20	10*10	5*5
SPOT	20*20			10*10			20*20			10*10		
DFAULT	100						500					
DROAD	0.438	0.335	0.302	0.436	0.347	0.342	0.539	0.436	0.413	0.497	0.415	0.413
DRIVER	2.110	2.229	2.194	2.151	2.280	2.220	2.174	2.300	2.268	2.193	2.330	2.272
DFAULT	0.690	0.626	0.679	0.464	0.451	0.475	0.546	0.558	0.588	0.328	0.381	0.393
DTM	1.374	1.268	1.263	0.612	0.557	0.594	1.150	1.030	1.019	0.487	0.399	0.435
Aspect	1.008	0.885	0.769	1.045	0.904	0.800	1.011	0.888	0.777	1.040	0.899	0.799
Slope	1.290	1.673	1.737	1.225	1.552	1.661	1.335	1.736	1.801	1.249	1.594	1.703
NDVI	4.666	4.383	4.201	4.399	4.185	4.214	4.684	4.397	4.212	4.378	4.156	4.178

Ids	0.65	0.92	0.7	0.65	0.92	0.7	0.67	0.94	0.7	0.66	0.93	0.7
	9	2	37	5	4	05	9	1	51	7	5	72
Id	1.04	0.94	0.9	0.95	0.86	0.8	0.93	0.83	0.8	0.90	0.79	0.8
	3	2	49	6	4	92	7	0	23	0	3	18
AUC	0.77	0.77	0.7	0.78	0.78	0.7	0.77	0.78	0.7	0.78	0.78	**0.7**
	5	9	77	7	6	89	9	3	81	9	9	**92**

5 PREDICTION OF LANDSLIDE SUSCEPTIBILITY MODELS

To verify the predictive capability of the susceptibility models, Typhoon Morakot in 2009 was selected as the target event. From the models developed in the previous section, key models were chosen for prediction. The maximum hourly rainfall and accumulated rainfall from Typhoon Morakot were input into the Typhoon Mindulle-based models under different conditions.

Table 5. AUC values of Different Landslide Susceptibility Model.

Grid Size	20m*20m	10*10m	5*5m		
SPOT Resolution(m*m)	10*10				20*20
Distance of Fault (m)	500			100	500
AUC(Build)	0.789	0.789	**0.792**	0.789	0.781
AUC(Predict)	0.715	0.723	**0.724**	0.719	0.734

6 LANDSLIDE DISASTER RESILIENCE

Nine control factors were selected for assessing the landslide resilience in the study, including population density, average electricity consumption per household, elder ratio, shelter ratio, fire fighter ratio, medical manpower ratio, income tax, and reconstruction budget.

This study further applied linear regression analysis based on the resilience of landslide disasters defined by Eq. (5) and Eq. (6). Karen (2011) quantified the resilience of landslide disasters with time dependence, using Eq. (5) to define the incremental resilience r(t), as the cost per unit time C(t) multiplying unit time Inc(t) and dividing by total time (NI). In Eq. (6), by integrating r(t) according to time, the resilience Re can be obtained.

$$\mathbf{r(t)} = \frac{\mathbf{C(t)\times Inc(t)}}{\mathbf{NI(t)}} \quad (5)$$

$$\mathbf{R_e(t_s)} = \int_{t_0}^{t_s} \mathbf{r(t)\,dt} \quad (6)$$

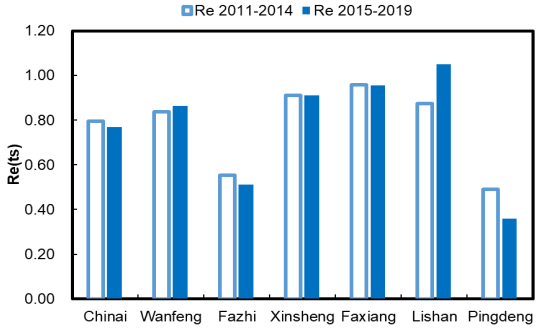


Fig. 11. Re(ts) Based on Landslide Susceptibility as the Hazard

7 CONCLUSIONS

1. NDVI might be the most important causative factor, followed by distance to river; this explains why there are more landslides occurred near river channels or areas with less vegetation,

2. The smaller the grid size, the higher the weight for slope gradient factor. Models with smaller grids show better prediction accuracy.

3. Higher resolution SPOT images yield more accurate vegetation info, However, may increase risk of overfitting in prediction models.

4. It suggests that 500-meter interval of distance to fault is better in accuracy comparing 100-meter interval one. Andin accuracy.

5. The results suggest that 5-meter grid + 500-meter fault interval could obtain the highest landslide susceptibility prediction accuracy in this study area.

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Special Lecture I:

Geobrugg experience in changing risk zones to no-risk zones by means of natural hazard protection measures. Experience in Switzerland and Kazakhstan

PhD. Alexander Barinov

Special Lecture Ⅱ:

Soil Mixing Around the World – Application and Solution Examples

Mr. Franz-Werner Gerressen

AsRTC10 «Urban Geo-informatics»

Principles of creating an effective territorial planning model in complex engineering-geological conditions

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ABSTRACT

Today, when developing territorial planning schemes for the subjects of the Russian Federation, as well as individual municipalities, it is necessary to identify territories at risk of rapidly developing natural hazards and man-made processes. This requirement is mandatory and reflected in the relevant regulatory documents. To achieve this, using modern algorithms for processing available information, including machine learning and artificial intelligence methods, a modern geoinformation engineering and construction model of planning restrictions for territories at risk of natural hazards is being created. This model allows approach the scenario of the hazardous processes development with a probabilistic risk assessment, achieving high forecasting accuracy for most types of geological processes. This model can also be used as a basis for the development of methods for calculating and constructing effective foundations of buildings and structures located within such territories, and their practical implementation in built-up areas subject to the simultaneous influence of several factors of hazardous processes.

Keywords: Engineering-geological conditions, geoinformation territory model, categories of complexity, foundation construction methods.

1 INTRODUCTION

Modern urban planning increasingly involves construction in areas with complex engineering-geological conditions, influenced by soil composition, groundwater, seismic activity, and other adverse factors identified during surveys. These conditions require advanced foundation design methods to ensure structural safety. Regulatory frameworks, such as SP 47.13330.2016, mandate early identification of zones at risk from natural and man-made hazards in territorial planning [1, 2]. This study focuses on developing machine learning algorithms to assess the combined impact of hazardous processes in high-risk areas, proposing engineering protection measures and optimized foundation solutions to enhance safety and stability.

2 MATERIALS AND METHODS

To assess engineering-geological risks for planned construction projects, surveys are conducted to forecast changes in ground conditions over a facility's lifecycle, particularly in complex hydrogeological settings [3]. In Southern Russia, much of the territory is classified as Category III complexity under SP 47.13330.2016, yet empirical data from high-rise construction reveals shortcomings in current classification systems. Actual conditions often exceed regulatory thresholds due to nonlinear soil behavior, anthropogenic influences, and shifting hydrogeological regimes, necessitating improved categorization methods [4, 5].

A state geospatial database is being developed, integrating digital terrain models, geological maps, hazard assessments, and seismic zoning in a unified coordinate system. These datasets are converted into matrix structures, with binary indices representing combined thematic layers, forming the basis for machine learning. Two training approaches were used: expert-guided parameter input and automated feature extraction from reference layers, both generating combinatorial event scenarios for neural network training [6, 7].

The neural network architecture employs input (WIH) and hidden (WHO) weight matrices, trained via backpropagation to classify hazardous processes – karst, floods, and four landslide types (shallow, flows, block-type, collapses). Validation involved 867 landslides from

1:100,000 maps and 109 from 1:10,000 maps, with contour accuracy verified. The model achieved 87–92% prediction accuracy, outputting risk maps and planning recommendations.

A dedicated software suite was developed for map-to-table conversion, task generation from legend encodings, neural network training, and predictive mapping. Post-training, the system enables point assessments, linear hazard profiling, and spatial forecasts, validated against 10–15 years of field data. Key advantages include adaptability to diverse geological processes and integration with multi-scale topographic data. The methodology enhances foundation design and territorial planning under multifactorial hazards.

3 RESULTS

The reliability of neural network modeling results was verified through comprehensive comparison with 10-15 years of field observation data. Analysis demonstrated strong alignment between predictions and actual conditions, with 78% of landslides occurring in clay-sand and sandy-clay formations, consistent with regional geological patterns. The model incorporated 430 recorded structural deformations, active landslide zones, flood areas, and surface runoff axes derived from digital elevation data.

To optimize foundation design in complex geological settings (particularly under multiple simultaneous hazards), a classification system was developed based on extensive construction experience. This system includes scientifically validated foundation types with detailed technical specifications and quality control methods. Key approaches feature: pile foundations with transfer layers, settlement regulation through enhanced rigidity, terrain-adaptive designs, and sequenced expansion joint implementation.

The integrated methodology combines neural network hazard forecasting with practical construction solutions, enabling effective territorial planning and development in high-risk areas like Sochi [8]. This approach bridges predictive modeling with engineering implementation, facilitating safe construction in challenging environments while incorporating modern high-rise building technologies.

4 DISCUSSION

The neural network-based approach for territorial risk assessment and foundation design optimization shows significant advantages over traditional engineering-geological methods while complementing them. Achieving 87-92% prediction accuracy for various geological processes (landslides, floods, karst) confirms the model's reliability in complex urban planning scenarios.

This machine learning-enhanced geospatial analysis overcomes key limitations of conventional classification systems by accounting for combined anthropogenic impacts, hydrogeological changes, and simultaneous interactions of multiple risk factors (e.g., seismic activity with slope instability and groundwater fluctuations) – aspects often oversimplified in regulations. The backpropagation algorithm enables continuous model refinement as new monitoring data becomes available.

The accompanying foundation classification system provides optimal technical solutions for high-risk zones identified by the model. This integrated approach has proven effective in multi-story construction along Russia's Black Sea coast, demonstrating substantial practical benefits.

5 CONCLUSION

The research demonstrates that current methods for assessing engineering-geological conditions need substantial improvement due to their limited accuracy in predicting complex natural-technogenic processes. Modern algorithms incorporating machine learning and artificial intelligence are being used to develop an advanced geoinformation model for planning restrictions in hazard-prone areas. This model allows probabilistic risk assessment of hazardous processes with 87-92% prediction accuracy for most geological phenomena. Field tests in Southern Russia's challenging conditions confirmed the methodology's effectiveness, particularly its innovative foundation solutions for complex sites. Future work should focus on integrating neural networks into territorial planning systems to enable safe, sustainable development of high-risk areas while minimizing geotechnical and environmental risks.

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Changes in land reclamation seen through borehole data

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ABSTRACT

In Osaka, The World Exposition will be held in Yumeshima is an artificial island. In this study, we used borehole data around Yumeshima areas to examine the characteristics of the geological ground, including changes over time. The past seabed elevation was clarified from the information from the boreholes conducted before the reclamation, and the distribution and thickness of each marine clay layer was extracted from the information from the boreholes conducted after the reclamation, and changes therein were studied.

The elevation of the seafloor surface at that time can be determined from the borehole survey data taken before the land was reclaimed. As land reclamation progresses, the elevation of the sea floor tends to gradually decrease due to overburden pressure, and at present data shows a subsidence of about 5 meters compared to before land reclamation. In the first stage of reclamation, a layer of sand is laid down on the ocean floor as a drainage layer. This sand layer can be recognized as the boundary with the natural ground from the borehole data after the start of reclamation.

Keywords: borehole data, artificial island, reclamation, sedimentary environment

1 INTRODUCTION

Yumeshima was reclaimed as part of the development of Osaka's bay area. Fig. 1 shows a map of Yumeshima and its surrounding area, as well as the coastline of the time in the early Meiji period. The Yodo River flowing down the northern edge of the figure. It is a spillway (artificial river) that was excavated in 1910 between Kema and Osaka Bay. The old Yodo River course split into the Aji River, Shirinashi River, and Kizu River.

Sakishima was reclaimed in 1967 and began to be used as a container terminal. Maishima was reclaimed in 1979, and Yumeshima began reclamation in 1985. Before reclamation, alluvial clay (Ma13 layer) was deposited directly below the seabed (depth about -10 m), and reclamation was carried out after a sand layer



Fig. 1 Map of Osaka Bay Area and old coastline and river

and surrounding revetments were constructed on top of it. The reclaimed soil contains dredged soil and waste, and since it was brought to the site at different times, the characteristics of the reclaimed soil are expected to be different depending on the location.

2 CONDITIONS BEFORE LANDFILL

Fig. 2 shows a cross section of drilling data (1980s) extracted from before landfilling. The core was located east-west of the north coast of Yumeshima. A Holocene clay layer (Ma13 layer) has been deposited on the sea floor (approximately -10 m above sea level), and below that the top of Pleistocene gravel layer (Dg1 layer) can be confirmed to be approximately 6 to 8m thick at approximately -40 m above sea level. Below that, a diluvial clay layer (Ma12 layer) can be confirmed. Both are marine clay layers, but below the Ma13 and Ma12 layers, alternating layers of sand and silt formed during the marine transgression are included in the description, and the somewhat coarse-grained silty clay layer is also included as part of the series of marine clay layers. natural water content (Wn), liquid limit (WL), plastic limit (Wp) are shown on the right side of the columnar diagram. The lower layer of the Ma13 layer contains silt and sand, so the natural water content Wn and liquid limit WL are relatively small. However, as the transgression progresses and the sea level rises, a stable clay layer is eventually deposited (below -30 m above sea level). Then, the natural water

content Wn and liquid limit WL change significantly, from about 50% to over 100%, and the natural water content Wn is also Wn≒WL. The area around -30m to -15m above sea level corresponds to the period when the sea was deep at the peak of the transgression, and the liquid limit WL is highest during this period. It can be confirmed that there are sections in the lower Ma12 layer where Wn and WL become small in the sediments during the transgression. As the marine transgression progresses, marine clay accumulates and Wn and WL tend to increase, but even at the peak of the marine transgression, Wn < WL. The top of the Ma13 layer is the seabed before landfill, but the values tend to vary in the section from the seabed to a few meters above it. This is thought to be because the top has been disturbed by dredging or a thin layer of artificial fill soil is distributed. The Dg1 layer distributed between the Ma13 and Ma12 layers consists of gravel and serves as a drainage layer when the clay layers above and below are consolidated.

3 CONDITIONS AFTER LANDFILL

The boring survey data before and after reclamation are compared and summarized in Fig. 3. As reclamation progresses, clayey soil such as the Ma13 layer is consolidated. Comparing the boring survey conducted in 1981 before reclamation with the three periods of 1991-1999, 2000-2004, and 2019-2020, the seabed elevation (upper surface of Ma13) before reclamation is about -10m. In contrast, after reclamation in 2019-2020, it is about -20 m. Comparing the bottom surface of the Dg1 layer, the elevation before reclamation was about -45 m, but in 2019-2020 it was about -50 m, and a subsidence of about 5 m was confirmed. Similarly, the bottom surface elevation of the Dg2 layer was about -75m immediately after reclamation, but about -80 m in 2019-2020.

ACKNOWLEDGMENT

This research was conducted using the KG-NET boring database as part of the "Research Committee on Yumeshima's Ground Properties and Subsidence Properties" (representativ e: Akihiko Oshima) established at the Kansai branch of the Geotechnical Society of Japan. We would like to take this opportunity to express our deepest gratitude.

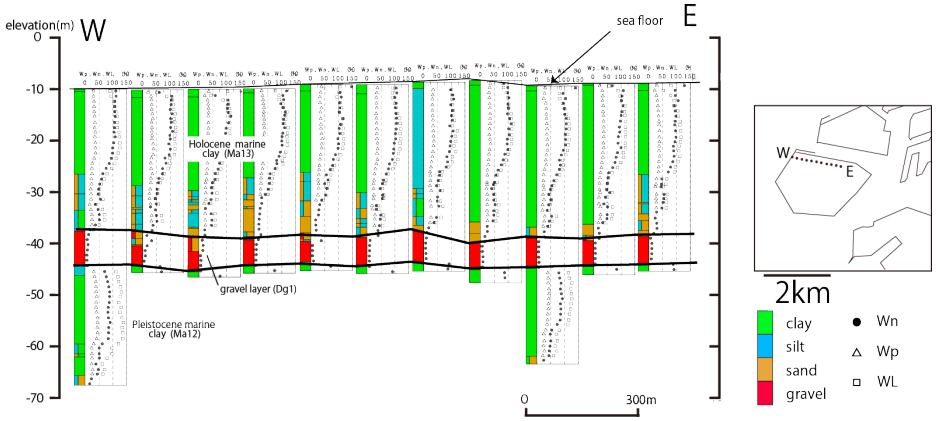


Fig.2 Cross-section of the borings in the Yumeshima area

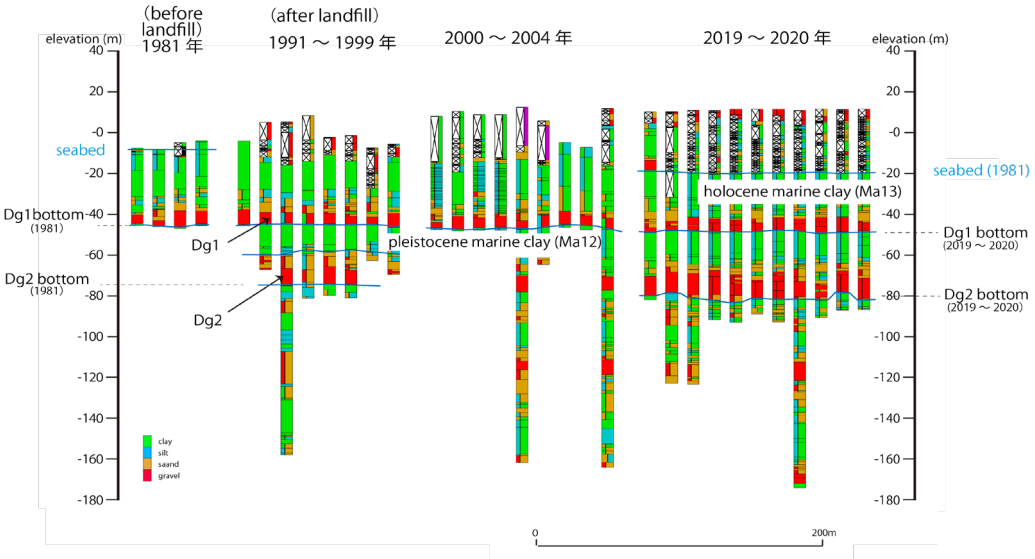


Fig.3 Yumeshima borings for all ages

Application of the engineering geological database of Taiwan to develop Vs prediction models

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ABSTRACT

The Engineering Geological Database (EGDT) was constructed for the Taiwan Strong Motion Instrumentation Program (TSMIP). The EGDT contains comprehensive data, including stratum descriptions and results from soil physical property tests such as grain size distribution, uniformity coefficient, coefficient of gradation, void ratio, water content, specific gravity, unit weight, liquid limit, and plasticity index (PI). The database also includes soil classification, P- and S-wave velocities, and SPT-N values. EGDT provides sufficient information for developing empirical correlations between engineering properties and for evaluating the influence of various factors on these properties. This paper presents the application of EGDT in developing Vs prediction models based on soil properties and SPT-N values.

Keywords: Engineering Geological Database, soil physical property, Vs, SPT-N

1 INTRODUCTION

The shear wave velocity (Vs) of subsurface sediments is a critical parameter in evaluating seismic wave amplification and conducting site response analyses. While this information is often unavailable during routine field investigations, the standard penetration test blow count (N) is commonly recorded. Consequently, numerous studies have aimed to establish empirical correlations between N and Vs for practical engineering applications. However, these existing correlations exhibit considerable variability in their mathematical forms and are often limited in their applicability to specific soil types, such as sands or clays. To address this limitation, this study develops a unified empirical correlation applicable to a broader spectrum of soil types. This new model leverages the extensive Engineering Geological Database from the Taiwan Strong Motion Instrumentation Program, incorporating several key soil properties, including liquid limit and plasticity index (PI), in addition to the routinely measured Vs and N of soil strata.

2 DATABASES

The Taiwan Strong Motion Instrumentation Program (TSMIP) Engineering Geological Database (EGDT) compiles extensive geotechnical information, including stratum descriptions, soil physical properties (grain size, uniformity/gradation coefficients, void ratio, water content, specific gravity, unit weight, Atterberg limits, PI), soil classification, P- and S-wave velocities (Vp and Vs), and SPT–N values. Beyond direct measurements of N and Vs, EGDT provides sufficient data for regression analysis. Effective vertical stresses (σo') are calculated using depth, unit weight, and water table elevation. When groundwater elevation is missing, the Vp profile helps estimate its approximate level, typically indicated by a sharp increase from below 500 m/s to above 1500 m/s in boring logs.

SPT–N values in EGDT are obtained using an automatic hammer system at 1.5 m intervals (or 3-5 m in gravel) and at notable drilling discontinuities. Suspension PS logging generally measures Vp and Vs every 0.5 m, with 1 m intervals in some earlier borings.

Boring IDs align with TSMIP station codes, categorized by regional abbreviations (TAP, TCU, CHY, KAU, TTN, HWA, ILA) based on province rather than geological units. Data from TAP, TCU, CHY, HWA, and ILA were used for regression. Refusal conditions (N > 50) were excluded. For non-plastic soils, PI was set to 1; similarly, FC was set to 1 for soils without fines content. The regression analysis utilized a substantial dataset of 3,684 points from 334 sites (borings), encompassing Vs, N, σo', FC, PI, and estimated OCR. This dataset shows Vs predominantly between 150-400 m/s (mean 258 m/s), N between 1-30 (mean 12.6), σo' mostly below 300 kPa (mean 167 kPa), OCR mostly below 3 (mean 1.6), PI mostly below 5 (mean 2.6), and a relatively even distribution of FC (mean 40.7%). The uniform distribution of FC across the database is advantageous for developing a robust, unified model. The soil types represented are clay (29%), silt (21%),

sand (48%), and gravel (3%).

3 VS MODEL

A unified empirical model is proposed according to the conditional prediction procedure (Kishida and Tsai, 2017) as follows

$$\ln Vs = 4.52 + 0.22\ln N + 0.11\ln\sigma_0' - 0.03\ln FC + 0.02\ln PI \quad (1)$$

The standard deviation of σ_{ln} is 0.27, which is approximated as σ = 68 m/s given mean Vs = 270 m/s. However, If the model form only includes σ_o' as typically modeled by others (e.g., (Brandenberg et al., 2010), then σ becomes high (Vs = 94 m/s).

In (Kuo et al., 2011), the dataset similar to that used in the present study was grouped into clay and sand for regression analysis, and the obtained σ of the individual Vs–N model were 67 m/s and 77 m/s. Although we used the entire dataset in the regression analysis, the standard deviation obtained in the present study is lower than that reported by (Kuo et al., 2011).

CONCLUSIONS

EGDT provides sufficient information for developing empirical correlations between engineering properties and for evaluating the influence of various factors on these properties. This paper presents the application of EGDT in developing Vs prediction models based on soil properties and SPT-N values. The model that includes additional prediction parameters, such as FC and PI, considerably improve the prediction accuracy. The model can be applied to general conditions for estimating Vs by N and is not limited to a specific soil type.

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Creation of the representative ground model utilizing Geo-informatic database and its application to deformation analysis due to reclamation

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ABSTRACT

In the present paper, boring data derived before construction of Yumeshima, that is the site of the World Expo 2025, are extracted from the database and used to develop an initial representative ground model of Yumeshima for numerical assessment. A series of finite element analyses is carried out to assess the stress and deformation of the reclaimed marine foundation and reclamation soil due to construction of Yumeshima Island. The calculated performance shows that in reclaimed dredged soil and alluvial clay layers with ground improvement had been carried out, consolidation completed early, whereas in Pleistocene clay layers, it took a long time for excess pore water pressure to dissipate, resulting in continued long-term settlement.

Keywords: Geoinformation, ground modeling, finite element analysis, settlement, excess pore water pressure

1 INTRODUCTION

Yumeshima Island in Osaka Port, that is the site of the World Expo, was originally planned as a disposal site for dredged soil from the Osaka Bay navigation channel. Yumeshima Island is an artificial island reclaimed with dredged clayey soil, resulting in a ground environment in which large compression occurs in the reclaimed soil layer as well as the natural foundation ground. In this study, we focused on Yumeshima Island in Osaka Port, and the representative subsoil ground model is created as an initial ground model prior to reclamation based on the pre-construction boring data. Using this ground model, a series of one-dimensional elasto-viscoplastic finite element analyses is carried out to examine the stress and deformation of the dredged clay and natural marine deposits of Yumeshima Island due to reclamation.

2 REPRESENTATIVE GROUND MODEL OF YUMESHIMA ISLAND BASED ON THE PRE-CONSTRUCTION BORIRING INFORMATION

A representative ground model of Yumeshima was created using pre-construction boring information from the database of Kansai Geoinformation Network (KG-NET) (KG-NET, 2023). The boring information of Yumeshima stored in the database includes a mixture of data from offshore boring before island construction and the ones from boring on the reclaimed land after construction. In this study, we used 52 boring data from before reclamation that were not affected by consolidation settlement to reproduce the initial ground conditions. Fig. 1 shows a representative soil profile model of Yumeshima before reclamation based on the 52 boring data stored in the geoinformatics database.

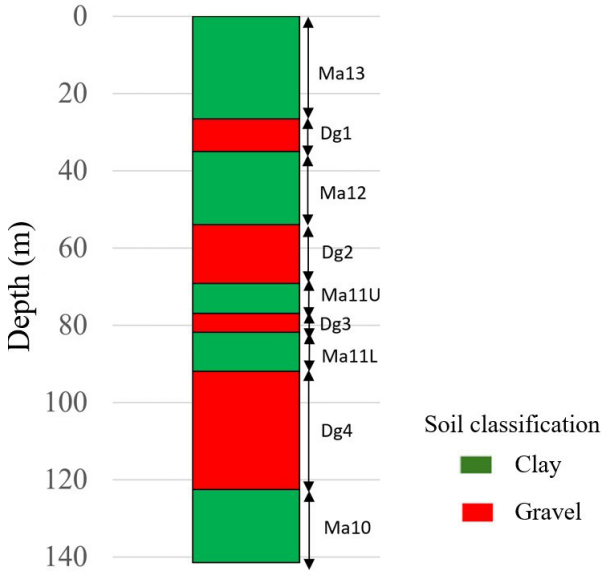


Fig. 1. Representative soil profile model of Yumeshima based on the geoinformatics database

3 CALCULATED RESULTS AND DISCUSSION

A series of one-dimensional finite element analyses were carried out (Mimura et al, 2025). The constitutive model used here is the elasto-viscoplastic model (Sekiguchi, 1977) that can describe a time dependent behavior of clays. Fig. 2 shows the depth distribution of excess pore water pressure throughout the ground. In each clay layer, the drainage effect from above and below causes a parabolic distribution of excess pore water pressure with a maximum value at the center of the clay layer. As of 2009, when loading due to reclamation was completed, excess pore water pressure remained in all clay layers, but as of 2020, excess pore water pressure has almost dissipated in Ma13, where the sand drains were poured, the culvert, the dredged clay layer, and Ma11U, which has a thin layer thickness. Furthermore, consolidation will be almost complete in all clay layers as of 2070.

The time-subsidence relationship for each clay layer is shown in Fig. 3 From the figure, the consolidation behavior of each clay layer due to reclamation can be confirmed. The Pleistocene clay layers (Ma12, Ma11U, Ma11L, Ma10) are in the pseudo-overconsolidated region in the early stages of landfilling, so they do not show significant compression, but the alluvial clay layer (Ma13) shows an increase in settlement from 1985, when the sand drains were cast. In the dredged clay layer, the amount of settlement has increased since 1993, when the plastic board drains were cast. The settlement rate of the dredged clay layer is faster than that of the alluvial clay layer (Ma13). As of 2020, the amount of settlement of the alluvial clay layer (Ma13) was 5.4 m. In addition, if there is no new loading after 2020, it is found that settlement has mainly occurred in the Pleistocene clay layer, and that gradual long-term settlement will occur in the Pleistocene clay layer.

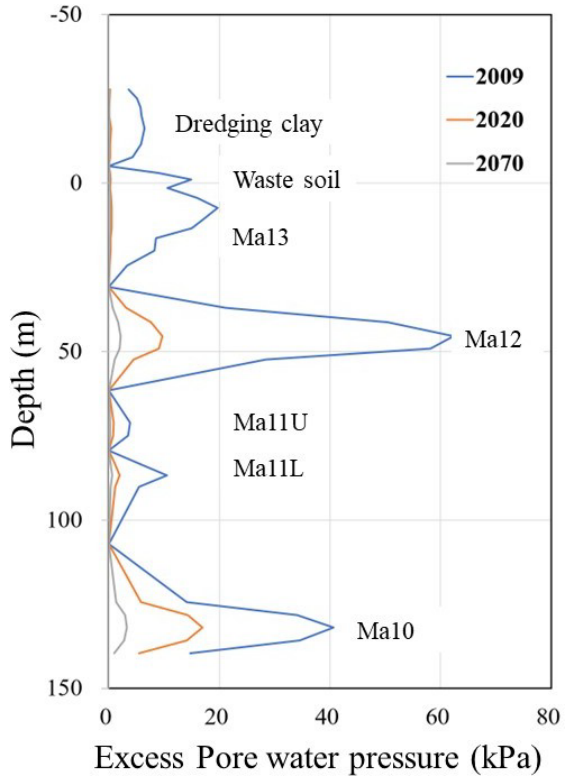


Fig. 2. Calculated distribution of Excess pore water pressure with depth

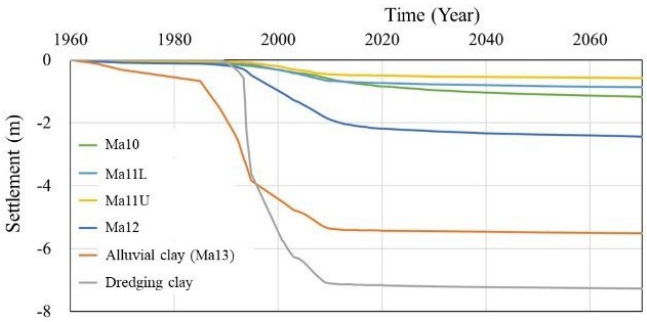


Fig. 3. Calculated compression of the clay layers with time

4 CONCLUSIONS

Using the Geoinformatic database of KG-NET, a representative initial ground model before reclamation was constructed for Yumeshima in the Port of Osaka. Numerical stress deformation analyses were performed using the one-dimensional elasto-viscoplastic finite element method. The calculated results showed that large settlement occurs in Ma13 and the reclaimed dredged clay layer, but consolidation was completed by 2020. On the other hand, long-term settlement will continue in the Pleistocene clay layer, and it is necessary to consider the impact on structures.

ACKNOWLEDGEMENTS

The research was carried out using the database of KG-NET. We would like to express our sincere gratitude to them.

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Testing GIS techniques for the determination of soil intermediate properties

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ABSTRACT

The article is devoted to enriching geotechnical surveying during the construction of buildings and facilities by incorporating GIS technologies and spatial analysis of soil properties. We propose the use of such spatial interpolation techniques as IDW, Kriging, and TopoToRaster to forecast intermediate geotechnical properties of soils laid between boreholes. The soil geotechnical properties of one of the construction fields in Astana, Kazakhstan, will be used to test the proposed approach. The expected results of spatial interpolation include special heatmaps showing the change in the distribution of the main geotechnical properties that influence the accuracy of the bearing capacity estimates of bases. This approach may be adapted to diverse engineering and geological conditions, including those from other regions.

Keywords: soils, geotechnical survey, properties, spatial interpolation, GIS.

1 INTRODUCTION

Engineering-geological investigations play a fundamental role in the planning and construction of buildings and infrastructure, as they provide key data on the mechanical and physical properties of soils. [1,2]. The primary method for collecting such data—borehole drilling and laboratory testing—offers information only at specific discrete locations. According to current regulations, boreholes are typically spaced 20 to 100 meters apart, depending on project requirements [3]. While this practice is standard, it limits the understanding of spatial continuity and layering in the subsurface, especially in heterogeneous soil environments [4,5]. As a result, it becomes necessary to estimate geotechnical characteristics in unsampled areas in order to create a more complete understanding of subsurface conditions

Recent research has questioned the adequacy of assuming uniform soil behavior across large areas and within a given layer [6,7]. Borehole data often fail to capture significant transitions and local variations between sampling points, leading to oversimplified interpretations of the underground profile. This can compromise the reliability of foundation design, especially when intermediate values at unsampled depths are required [8].

Table 1. Comparison of interpolation methods

Criteria	IDW	Kriging	TopoToRaster
Type	Deterministic	Geostatistical	Deterministic
Assumption	Closer points are more similar	Spatial autocorrelation	Hydrologically-correct surfaces
Data required	Coordinates, values	Coordinates, values, variogram	Elevation + value data
Accuracy	Moderate, sensitive to outliers	High, robust to noise	Moderate
Complexity	Low	High	Medium
Output	Smooth gradient	Spatially adaptive surface	Smooth continuous grid
Software	QGIS, ArcGIS	ArcGIS, R, SAGA	ArcGIS Pro

2 METHODS

In this paper, we apply and compare three interpolation methods—IDW, Kriging, and TopoToRaster—to predict intermediate values of Young's modulus (E), cohesion (c), and friction angle (φ) using data from boreholes drilled at a construction site in Astana. We also compare our approach with recent studies in the field and discuss the implications for foundation design.



Fig. 1. Construction field.

Table 2. Data base

No.	E_1	c_1	φ_1	E_2	c_2	φ_2	…	E_i	c_i	φ_i	*lon*	*lat*
1												
2												
…												
i												

3 RESULTS

The heatmaps produced by spatial interpolation revealed significant variability in the soil mechanical properties across the selected construction site in Astana:

At 5 meters depth, interpolated values of Young's modulus (E) ranged from 11–27 MPa, cohesion (c) from 14–73 kPa, and friction angle (φ) from 13°–28°. Although soil classification at this depth indicated loamy sand, the spatial patterns suggested notable heterogeneity in geotechnical behavior, in line with findings [12].

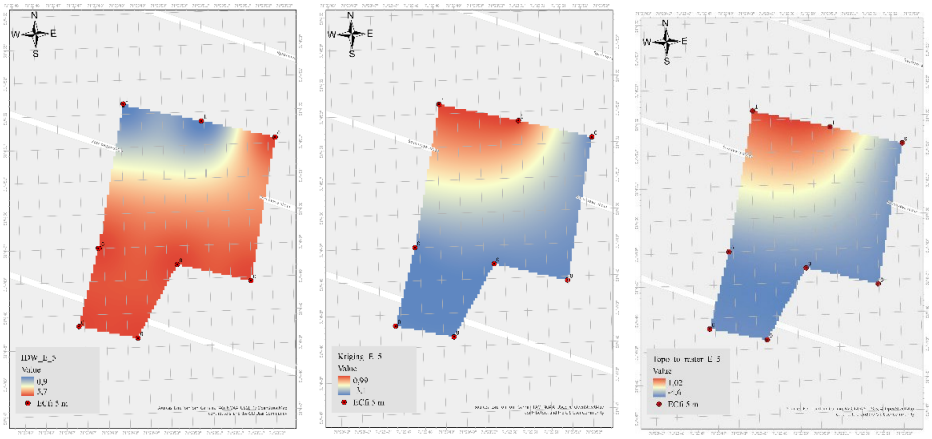


Fig. 3 *E*-distribution at $D = 5$ m: (a) IDW; (b) Kriging; (c) TopoToRaster

4 CONCLUSIONS

GIS-based spatial interpolation provides an effective means to estimate soil mechanical properties between boreholes. Our comparison of IDW, Kriging, and TopoToRaster showed that each method has strengths depending on the context:

Kriging is suitable for detailed spatial analysis with sufficient data.

IDW is simple and effective for preliminary assessment.

TopoToRaster aids in broader trend visualization.

This methodology enhances foundation design by accounting for intermediate property variations, thus reducing geotechnical uncertainty. Future work will focus on integrating these techniques into automated tools for multidimensional raster data modeling using formats such as NetCDF or HDF.

ACKNOWLEDGEMENTS

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Innovative approaches to system design considering the mutual influence of urban environment

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ABSTRACT

Excavation design requires in-depth analysis and consideration of many interrelated factors. When conducting a geotechnical assessment, it is necessary to consider not only the geological characteristics of the soils, but also their interaction with structural elements, groundwater and surrounding infrastructure. The use of a software package such as Plaxis allows modeling the behavior of excavations under various loads and influencing factors. The article considers an example of analyzing the comparison of a flat and three-dimensional formulation of a calculation model, which led to a change in technical solutions and a reduction in construction costs. During the calculations in the basement area, a critical deformation was identified for historical buildings exceeding 5 mm, which required the development of measures to strengthen the soil. However, after analyzing the three-dimensional model, it turned out that the implementation of additional measures, such as creating a geotechnical screen, could lead to unjustified costs and a decrease in the effectiveness of the project. This approach not only optimizes design solutions, but also allows for more accurate prediction of the future behavior of the structure, which is a prerequisite for achieving reliability and sustainability of construction projects. Further research in this area should be aimed at improving the modeling of mutual influences and developing recommendations for minimizing risks in the design of pits.

Keywords: investigations, structural stability, soil modeling, deformation prediction.

1 INTRODUCTION

The residential complex "Mypriority Paveletskaya" (Fig. 1.) is a modern business-class project in the center of Moscow, implemented by the developer "Granel" on a territory of about 3.5 hectares with two buildings of different heights. The complex combines modern architectural solutions with elements of the historical development of the area, offering a variety of layouts and developed infrastructure.

Particular attention is paid to geotechnical aspects during construction, since the complex engineering and geological conditions of the central part of the city require the use of specialized methods for strengthening the pit.



Fig. 1. Residential complex "Mypriority Paveletskaya". Render.

The article discusses the experience of ELBA Engineering company in identifying and solving geotechnical problems that arose during the design of the excavation of this object.

ELBA Engineering Company (ELBA Engineering Ltd.)

is an engineering organization specializing in geotechnical surveys, design and construction project management. The company acted at the site as a general designer and as a result of working documentation development the company performed optimization, confirming the need to take into account the mutual influence of the designed excavation and the existing urban development.

2 ACTUALITY

Excavation design studies require consideration of the interaction of soil masses with the urbanized environment. It is important to integrate numerical modeling of the stress-strain state of soils using programs such as Plaxis and optimize enclosing structures with verification through geotechnical monitoring. Nonlinear soil behavior models and sedimentary troughs analysis remain key aspects to ensure safety in areas of historical development. For the Mypriority Paveletskaya residential complex, located in the dense historical district of Zamoskvorechye, ELBA Engineering applied an integrated approach to minimize the risks of deformation of surrounding buildings.

3 MAIN PART

The design of excavations in dense urban areas requires an in-depth analysis of geological characteristics, the influence of existing structures and minimization of environmental impacts. Innovative methods such as three-dimensional modeling and finite element method are the basis for accurate assessment of stress-strain state of soils. Geotechnical monitoring using modern technologies provides control over deformations and allows prompt response to changes, which is critical for construction safety.



Fig. 2. Preliminary architectural and urban planning concept.

An important stage of this process is the study of similar cases and scientific research, which contributes to the formation of reasonable design solutions. Taking into account the mutual influence of buildings, underground utilities and other factors requires high-quality engineering surveys and the development of integrated solutions. Optimization of design taking into account cost, safety and environmental sustainability is a prerequisite for achieving reliable results in complex urban environments.

4 COMPANY EXPERIENCE

During the implementation of the Mypriority Paveletskaya residential complex project, located in a historic development (fig.2), the task of ensuring the stability of the excavation while minimizing the impact on the environment arose. Initial calculations performed in Plaxis 2D revealed critical deformation zones. However, an alternative approach proposed by ELBA Engineering, based on 3D modeling in Plaxis 3D, allowed a more accurate assessment of the bearing capacity of the soils and justified the absence of the need for the expensive geoscreen requested by the 2D calculation, which in turn reduced project costs.

The analysis of the performed calculations emphasized the importance of using adequate geotechnical modeling methods in dense urban development. The design of excavations should take into account the geometric features of the object and the nonlinear properties of soils, which allows for a more accurate prediction of the structure-soil system interaction. Familiarization with similar cases and scientific research in this area facilitates the exchange of experience and the introduction of best practices, which is essential for improving the efficiency and safety of construction projects.

5 CONCLUSIONS

The results of the analysis emphasize the necessity of using adequate methods of geotechnical modeling in the design of excavations in dense urban areas. The use of three-dimensional models, taking into account geometric features of the object and nonlinear properties of soils, allows to predict more accurately the behavior of the system “structure-soil” and avoid excessive costs for expensive geotechnical measures.

The obtained results correspond to the modern trends of geotechnical engineering development, focused on increasing the efficiency and safety of construction through the use of advanced computational methods and optimization of design solutions.

This example demonstrates that competent application of modern technologies allows not only to optimize costs, but also to increase reliability and safety of construction projects.

Geotechnical investigations in earthquake-damaged burial mounds toward restoration

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ABSTRACT

This study reports the physical and mechanical properties of the Sakuradani Tumulus damaged by the 2024 Noto Peninsula Earthquake. We evaluated in-situ shear strength with needle penetration test and measured dry density and water content with the radioisotope method in an excavated trench for archaeological investigations. The uniaxial strength estimated from needle penetration test was relatively large in the zones where large dry density was measured. The measured dry density was smaller than that achieved with manual compaction energy. This indicates that the mound was developed by piling up soil layers without compaction and had small seismic resistance.

Keywords: compaction, needle penetration test, radioisotope moisture-densimeter, crack

1 INTRODUCTION

Burial mounds developed from the 3rd to 8th century are Japan's important cultural heritage. Earthquakes and rainfall easily damage the manually compacted ancient earth mounds. Geotechnical approaches are useful for analysing damage mechanisms and developing restoration methods with sufficient mechanical stability. This study investigated the physical and mechanical properties of the Sakuradani Tumulus No.1 where a large tensile crack[1)] occurred due to the 2024 Noto Peninsula Earthquake.

2 METHODS

The mound soil was sandy soil with a fine fraction content of 20.3%. The burial mound had a terrace in the middle, and the lower part was thought to be disturbed as it had been formed due to the collapse of the upper part. Field investigations were performed in an archaeological trench excavated at the cracked part. Soil moisture and dry density were measured with the RI method (JGS1614), and uniaxial compression strength was indirectly measured with needle penetration test (JGS3432).

3 RESULTS AND DISCUSSIONS

The dry density of the upper intact part (1.26 g/cm^3) was larger than that of the disturbed lower part (1.11 g/cm^3). The measured dry density of the intact zone was smaller than that achieved with manual compaction energy equal to 10-20% of the Standard Proctor[2)]. Fig. 1 shows the needle penetration test results of the intact zone. The measured uniaxial strength was from 50 to 100 kN/m^2, 3 times larger than in the disturbed zone. These results are consistent with the dry density difference between the intact and disturbed zones.

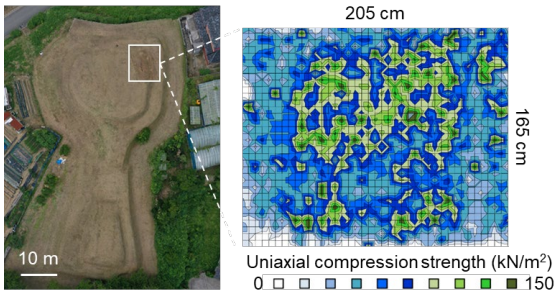


Fig. 1 A plan view and needle penetration test results.

4 CONCLUSIONS

The field test results showed the difference in density and strength between the intact and disturbed zones. These results provide insights into the formation of the burial mound and are useful for restoration.

ACKNOWLEDGEMENTS

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TC-305 Geotechnical Infrastructure for Megacities and New Capitals

Simulation of the resistance of a deep-sea soil base by moving loads

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ABSTRACT

The object of the study is the deep–sea silty base of the mineral collection unit. The subject of the research is the study of the deformation features of oceanic silts. The purpose of the work is to develop a method for predicting the resistance of a silty base when interacting with mobile local loads. Methods: laboratory determination of the physical, strength and rheological characteristics of soils; physical modeling; statistical treatment of test results. The approach is verification of the viscoelastic soil model with the Coulomb-Mohr fluidity condition. Main results: development of testing equipment and instruments; obtaining the values of the parameters of the strength and rheological properties of the silty base of the collection unit.

Keywords: soil base, oceanic silt; soil model, rotary cut, bench tests, silt strength and viscous parameters

1 INTRODUCTION

According to experts, due to the expected shortage in the markets of lithium, nickel, cobalt, the development of deposits of ferromanganese nodules in the World Ocean can become profitable no earlier than 2030.

The safety of design, construction and operation of deep-sea complex for solid mineral extraction requires a reliable assessment of the behavior of soil base when interacting with moving mineral collection units (MCUs).

Testing the mechanical properties of bottom sediments in the ocean seems to be the most reliable, but very difficult and expensive solution. Therefore, an empirical approach to the assessment of the stress-strain state of soil base of MCU by the method of physical modeling is quite relevant.

2 MATERIALS AND METHODS

Soil samples for testing were taken with a box sampler in the Clarion-Clipperton ore province in the Pacific Ocean in the bottom depth range of 0.1-1.0 m. The natural structure of the soil was significantly disrupted during sampling, packaging, and transportation of samples.

Tests of the bottom sediment by moving loads were carried out by deforming a set of model objects on a stand for modeling the operation of soil bases (SMOB-1) (Fig.1).

The soil was placed in a stand tray, stirred to a paste-like state, covered with a layer of seawater and seasoned to restore structural strength.

Models in the form of rectangular thin flats were placed in the silty base perpendicular and along the direction of their movement, respectively. Models in the form of skis were loaded with vertical pressure.

In the course of the experiments, the traction forces T and displacements U of model objects were measured and recorded, from which the soil resistance was calculated.

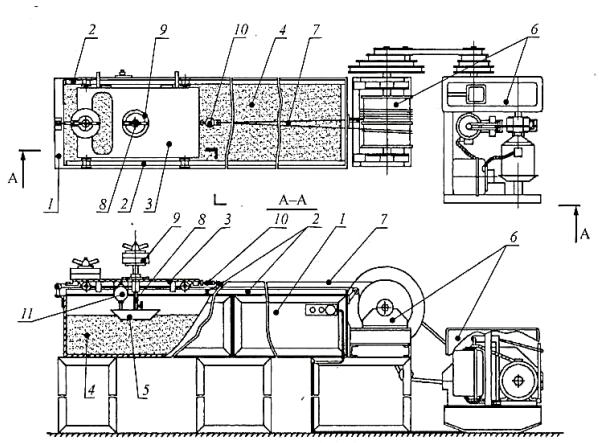


Fig.1. Stand for modeling the operation of soil bases (SMOB-1): 1 – soil tray; 2 – rails; 3 – truck; 4 – soil; 5 – model; 6 – winch; 7 – rope; 8 – stock; 9 – goods; 10 – sensor measuring resistance of soil; 11– sensor measuring soil base settlement of model (c.c. 1385798 USSR)

To describe the results of determining the bottom soil resistances on the frontal σ_x, lateral τ_{xy}, and plantar τ_{xz} surfaces of the models, the viscoplastic body equation with the Coulomb-Mohr flow condition was

used:

$$\sigma_x = K_p\gamma h + 2c\sqrt{K_p} + \eta d\gamma/dt; \quad (1)$$

$$\tau_{xy} = c + \sigma_{yg} tg\varphi + \eta d\gamma/dt; \quad (2)$$

$$\tau_{xz} = c + \sigma_{zp} tg\varphi + \eta d\gamma/dt; \quad (3)$$

where $K_p = tg^2(45° + \varphi/2)$ – coefficient of passive pressure; φ – angle of internal friction; γ – saturation unit weight; h – depth of immersion of the model; c – intercept cohesion; η – viscous coefficient; $d\gamma/dt$ – strain rate; σ_{yg} – horizontal pressure from the soil's own weight; σ_{zp} – vertical pressure from external load.

The values of the strength and rheological parameters of the soil required to solve the equations [formulas (1, 2 and 3)] were determined by testing silt samples on a device of rotational cut (Fig. 2).

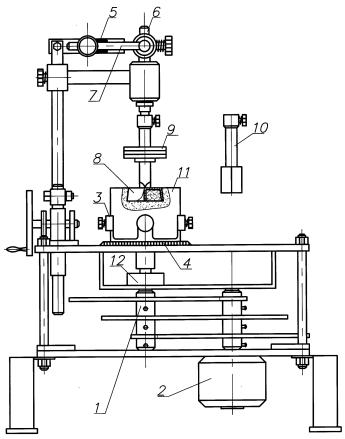


Fig. 2. Rotary cut device (RCD-3): *a* – design scheme; *b* – general view. 1 – mechanical gearbox; 2 – electric motor; 3 – rotary table; 4 – limb; 5 – strain gauge plate; 6 – shaft; 7 – L-shaped stop; 8 – vane stamp; 9 – weights; 10 – vane; 11 – ring with soil; 12 – angle sensor (c.c. 1481683 USSR)

3 RESULTS

Typical graphs of deformation of silt samples at constant rates are shown in Fig. 3, *a*, *b*.

Distinctive features of the graphs are the presence of peak and residual shear strength (Fig. 3, a), dependence of silt strength on deformation rate (Fig. 3, b).

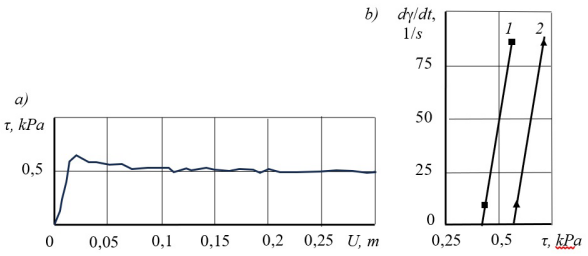


Рис.3. Кривые сопротивления срезу океанического ила: *a*) $\tau_i = f(U_i)$; *b*) $d\gamma/dt = f(\tau_i)$. Обозначения: 1 – остаточное сопротивление; 2 – то же пиковое

The results of the experimental and calculated [formulas (1, 2 and 3)] determination of the resistance values of the silty base depending on the degree of thixotropic hardening, the overall dimensions of the models and the deformation rates are presented in Table 1.

Table 1. Strength properties of silt base

Resistance moving load	Calculated values		Experimental values	
	Peak	Residual	Peak	Residual
Frontal σ_x, kPa	2.4 – 3.9	–	3.9 – 12.5	–
Lateral τ_{xy}, kPa	1.1 – 3.9	0.28 – 0.59	0.44 –1.38	0.44 –1.38
Plantar τ_{xz}, kPa	1.1 – 2.6	0.4 – 1.0	0.4 – 1.3	0.08 – 0.14

The experiments recorded an increase of 2-3 times in the resistances of the silt base σ_x, τ_{xy} and τ_{xz} with an increase in the values of normal pressures σ_{yg} or σ_{zp} also approximately 2-3 times. While the calculation of such a significant influence of friction components $\sigma_{zg}tg^2(45° + \varphi/2)$, $\sigma_{yg}tg\varphi$, $\sigma_{zp}tg\varphi$ on resistance values does not confirm.

4 CONCLUSIONS

Since the value of each of the silt base resistance parameters σ_x, τ_{xy} and τ_{xz} separately is simultaneously influenced by several variable parameters, such as thixotropic hardening, overall dimensions and deformation rate, then in order to introduce correction factors into the initial equations [formulas (1, 2 and 3)] in further work it is necessary to use the theory of similarity and multifactorial correlation analysis.

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Mechanical properties of soil during scenario earthquakes

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ABSTRACT

The article highlights the results of a study of the resistance to vibration liquefaction of sandy and clayey soils of the foundation of an industrial construction site during scenario earthquakes.
In order to study the dynamic stability of clay layers and water-saturated sandstones from the standpoint of assessing the possibility of their liquefaction under seismic impacts corresponding to the design (predicted) seismic activity of the site, soil tests were carried out in the laboratory for triaxial cyclic loading equivalent in force to the seismic impact. Based on the analysis, an assessment was made of the dynamic stability (vibration liquefaction) of sandy and clayey soils of the foundation of varying degrees of water saturation of the construction site during scenario earthquakes, taking into account the features of the soil stratification.

Keywords: water-saturated sandstone, clay, dynamic loading, triaxial compression, seismic action, soil deformation, vibration liquefaction, pore pressure, accelerogram

1 INTRODUCTION

Design of hazardous industrial facilities requires safe design solutions in case of earthquakes. Earthquakes with an intensity of 7 points on average soils are predicted in the territory of the Republic of Tatarstan, and, as a result, it is necessary to assess the dynamic properties of foundation soils during surveys and to use anti-seismic reinforcement during the design and construction of structures. Taking into account the above, when designing foundations for critical structures, it is necessary to take into account the influence of seismic load from a possible earthquake on changes in the physical and mechanical properties of soils (Mirsayapov 2012, 2021, 2023, Seed 1996).

2 GENERAL PART

The site under study is located in the Kama River basin in the northeast of the Republic of Tatarstan. The results of seismic microzoning of the construction site show that the seismic activity of the site with the given foundation soils is estimated as 7 points with an acceleration of 143 cm^2/s with short and medium periods of oscillations.

The territory is composed of alternating layers of clay and water-saturated sandstone to a depth of 28 m.

The geological structure of the survey site includes interbedded Middle Permian eluvial clays, sandstones, as well as siltstones and limestones. In some areas, the Middle Permian alluvial deposits are covered by a thin cover of Quaternary eluvial-deluvial loams. From the surface, the Quaternary eluvial-deluvial and Middle Permian eluvial deposits are covered by fill soils with a thickness of 0.2 to 4.0 m. The foundation soils are located in the zone of groundwater fluctuations.

To study the dynamic stability of clay layers and water-saturated sandstones from the standpoint of assessing the possibility of their liquefaction under seismic impacts corresponding to the design (predicted) seismic activity of the site, laboratory tests were conducted for triaxial cyclic loadings equivalent in force to the seismic impact.

When modeling the deformation conditions in which the soil is located under seismic impact in real field conditions, before cyclic loading, a predicted seismic load is modeled based on the methodology proposed in (Seed 1996).

According to this method, seismic load can be characterized by the value of the reduced cyclic shear stress (CSR) for an earthquake of a given frequency and intensity:

$$CRS = \frac{\tau_{av}}{\sigma'_v} \quad (1)$$

where τ_{av} is the average value of expected cyclic shear stress for a given magnitude; σ_v' is the vertical confining stress.

The number of loading cycles (N) in a laboratory experiment simulating seismic action depends on the duration of the earthquake, and therefore on the magnitude of the earthquake. The approach gives the maximum value of the expected cyclic shear stresses during an earthquake (τ_{av}), which, when conducting triaxial dynamic tests, corresponds to half the axial dynamic load (fig. 1).

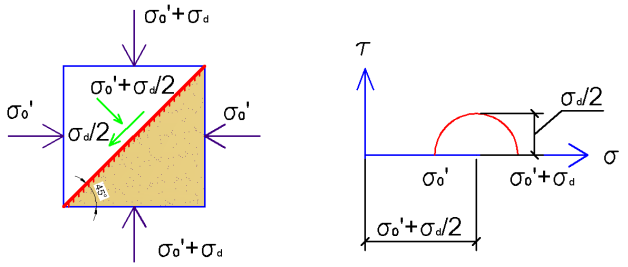


Fig. 1. Modeling stresses in a soil sample under triaxial compression under cyclic loading.

Based on the above-considered methodology, a calculation model of the stress-strain state of a layered foundation composed of sandy and clayey soils of varying degrees of water saturation during scenario earthquakes has been developed, taking into account the possibility of soil mass acceleration in three directions, as well as the mutual influence of layers of different rigidity.

For a quantitative assessment of the liquefaction of layers of water-saturated sands and clays under random irregular multidirectional seismic action, correction functions are introduced to adjust the cyclic strength obtained under stationary cyclic loading in order to take into account the above-mentioned features of real seismic loading C_2 and C_5 (Seed 1996). Based on the developed calculation model, equivalent parameters of regular cyclic loading were determined for conducting laboratory studies of the liquefaction resistance of sandy and clayey soils of the foundations of the construction site under the following calculated characteristics of the scenario earthquake: intensity 7, acceleration A = 143 cm^2/s, fundamental frequency around 2 Hz, fundamental period of oscillations 0.56 s.

To assess the resistance to vibration liquefaction, the following criteria for resistance to destruction were adopted:

– occurrence of axial deformation under triaxial cyclic loading less than 5%;

– the pore pressure coefficient $\beta = p_w/\sigma_m$ should be $\beta \leq 0.6$;

– the hysteresis loop width at the 30th loading cycle should be less than the loop width at the 29th loading cycle, i.e. $\Delta\varepsilon_{30} \leq \Delta\varepsilon_{29}$.

The conducted experimental studies made it possible to establish patterns of deformation growth under equivalent cyclic loading.

After testing for cyclic loading with parameters equivalent to the calculated scenario seismic impact, the soil samples were brought to destruction by a deviatoric static load according to the "crushing" scheme. It was found that, in general, seismic impact on soils did not lead to a decrease in the ultimate deviatoric load compared to the results of static loading.

3 CONCLUSIONS

A calculation model of the stress-strain state of sandy and clayey soils with varying degrees of water saturation during scenario earthquakes has been developed, taking into account the possibility of soil mass acceleration in three directions. Based on the developed calculation model, equivalent parameters of regular cyclic loading have been determined for conducting laboratory studies of the liquefaction resistance of sandy and clayey soils of the foundations of a construction site under the following calculated characteristics of a scenario earthquake: magnitude 7, acceleration A = 140–143 cm^2/s, fundamental frequency around 2 Hz, fundamental oscillation period 0.56 s. Experimental studies of resistance to vibration liquefaction of 143 series of soil samples of undisturbed (134 series) and disturbed (9 series) structure (three twin samples in each series) were conducted in a triaxial compression device (stabilometer) under cyclic loading with parameters equivalent to the parameters of a scenario earthquake with an intensity of 7 points, established for the construction site.

In the course of experimental studies, the main parameters characterizing the state of sandy and clayey soils under cyclic loading were established: longitudinal and radial deformations, pore pressure, effective and average stresses.

The results of experimental studies showed that with cyclic loading parameters equivalent to the main characteristics of a scenario earthquake with an intensity of 7 points, the resistance to liquefaction of sandy and clayey soils of the foundation of the construction site is ensured according to all three accepted criteria.

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Assessment of the stability of bulk soils under dynamic loads caused by seismic effects predicted for the conditions of the construction site of a nuclear power facility

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ABSTRACT

Currently, more and more unique industrial buildings and structures are being built in difficult geotechnical conditions all over the world. One of the most common factors complicating the design of facilities with a high level of responsibility from the point of view of geotechnics is construction in earthquake-prone regions. Seismic impacts lead to a loss of soil stability under dynamic influence and require the determination of nonlinear dynamic properties of soils for the subsequent design of structures on such grounds.
To carry out a qualitative and quantitative assessment of the stability of the base of a nuclear power facility under dynamic loads, a series of special laboratory studies was carried out, including dynamic triaxial compression tests and low-amplitude torsional vibration tests in a resonant column. The base was represented by bulk sandy soils, soil samples were tested at full water saturation and in accordance with the regulatory and technical documentation of the Russian Federation. According to the results of dynamic triaxial compression tests, dilution potentials were determined for each of the soil samples taken from a depth of 0.8 to 9.0 m. As a result of low-amplitude dynamic tests performed using a resonant column, the corresponding resonant frequencies, shear modulus, and damping coefficients were obtained for each of the soil samples taken from a depth of 3.8 to 8.0 m. During the research, a computer program was implemented and tested to speed up the processing of test results, as well as improve the accuracy and reliability of the data obtained because of processing. The use of automated tools for processing laboratory test results significantly reduces the time required to complete research and improves the accuracy of processing. The tests performed made it possible to assess the dynamic stability of the projected structure at different intensities of the predicted seismic impact. Based on the data obtained, a computational model was created in the Plaxis geotechnical software complex, in which the behavior of a sand embankment under dynamic loads was modeled. An analysis of the stress-strain state of the base of the projected structure under seismic influence has been performed, considering the soil parameters determined based on special laboratory studies of the dynamic properties of soils. The assessment of the risk of loss of stability of the foundation soils at various levels of seismic impact is carried out based on the initial data on the predicted seismic impact.

Keywords: dynamic loads, bulk soils, seismic impact, resonant column, geotechnics, soil stability, mathematical model, Plaxis

1 INTRODUCTION

Studying the dynamic characteristics of water-saturated soils is an important task in geotechnics, since these characteristics significantly affect the design and construction of various structures and their stability under potential dynamic influence. The dynamic characteristics of the soil can be determined by dynamic triaxial compression and triaxial compression tests using the resonant column method. In this article, we will focus on determining dynamic characteristics using a dynamic triaxial compression device and a resonant speaker. The mathematical modeling of the base of a nuclear power plant in a general form in the Plaxis geotechnical software package and the evaluation of the results are also considered.

Experimental studies in a resonant column make it possible to obtain soil deformations in the amount of 10^{-6} to 10^{-2}% of the sample height and using a dynamic triaxial compression device - from 10^{-2} to 10%.

Tests in such devices make it possible to fully assess the effect of dynamic effects of various spectra and

frequencies on soils and then quantify the dynamic stability of these soils.

2 METHODS

Laboratory studies of soils using low-amplitude dynamic tests in a resonant column were carried out to determine: the velocity of transverse waves VS (m /s), the dynamic shear modulus G (MPa) and the absorption coefficient (damping) D (%) of soil in the range of shear deformations of the order of 10^{-6}-10^{-2}, as well as to study changes in these indicators in the range of small shear deformations (not more than 0.1%) in order to calculate the vibrations of structures under dynamic influences.

The tests were performed in type "A" chambers designed for samples up to 140 mm high and up to 70 mm in diameter.

The analysis of the dynamic liquefaction process is carried out by comparing the tangential stress caused by the seismic load and the tangential stress required to start soil liquefaction, or the level of shear strain amplitude, which is considered unacceptable for design. The range of measurement of shear deformations is 10^{-2} to 10 %

To achieve this goal, the experimental studies were divided into several stages:

The first stage was aimed at determining the dynamic characteristics of a water-saturated sandy soil.

The second stage focused on compiling a plausible and accurate soil model in the Plaxis geotechnical software package with the specification of characteristics obtained because of laboratory tests conducted at stage 1.

The third stage included a comparison of the results of the first two stages.

The tests were carried out on samples of fully water-saturated sandy soil with a soil skeleton density of ρ_d from 1.38 to 1.49 g/cm^3 at lateral pressures from 14 kPa to 139 kPa to determine the dynamic stability of the soil and from 61 kPa to 123 kPa to determine the degradation of the shear modulus and damping coefficient.

3 FINDINGS

The results of laboratory tests on a resonant column allowed us to obtain the deformation characteristics of fine water-saturated sand in the following values: shear modulus G - from 21.92 to 71.62 MPa; damping coefficient - from 5.89 to 11.05%.

As a result of tests to determine the dynamic stability of soils, the following data were obtained for the same sand: safety coefficient k_s at the groundwater level 2.6 m from the ground surface was 3.12, at a level of 5.3m - 3.72, at a level of 8 m - 3.81, which, according to GOST 25100-2020, means soil stability.

However, with a potential increase in dynamic impact (above the initial target), this soil loses stability and its k_s values at the same groundwater levels take values from 0.92 to 1.45, which on average gives a value of 1.13, which means that the stability conditions are not met.

4 CONCLUSIONS

A series of experimental studies were conducted to determine the potential for soil liquefaction and degradation of the shear modulus and damping coefficient.

The study and determination of dynamic characteristics by laboratory methods is a necessary step towards developing safer and more practical approaches in geotechnical design.

Since experimental studies were conducted on only one type of soil (fine sand), further research is needed to study the influence of granulometric composition, density and degree of water saturation and other factors affecting the change in the deformation characteristics of the soil. In addition, it is necessary to conduct further studies of various types and varieties of soils and consider other types of dynamic tests to clarify their effect on the stability of water-saturated soils.

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Efficacy of fly ash-based water-absorbing polymer in improving unsaturated soil properties and promoting vegetation growth under drying conditions

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ABSTRACT

The fundamental goal of this research is 1) to establish the efficacy of WAP-amended soil on grass species under water stress condition and 2) to estimate the root water uptake (RWU) of soil water movement under the effect of WAP. The experiment was carried out on grass species (i.e., Cynodon dactylon) in a greenhouse. After irrigating vegetation under well-watered condition, it was subjected to water stress condition. The results demonstrate that the WAP improved the water retention behavior due to the alteration of water storage in the soil matrix. The water retention characteristics such as saturated water content (SWC), plant available water content (PWAC), and wilting time (WT) was improved more than 1.5 times, 2 times, and 1.2 times, respectively, as compared to the control soil. Furthermore, it is observed that WAP-amended soil has higher root water uptake from the soil matrix compared to bare soil.

Keywords: Water absorbing polymer (WAP), waste fly ash, water retention behavior, plant available water content(PAWC), root water uptake

1 INTRODUCTION

The frequency and severity of natural disasters such as droughts, heat waves, forest fires and flash floods are increasing, especially arid and semi-arid regions are facing severe water shortages. To manage this problem, alternative water conservation measures such as irrigation scheduling management, water deficit irrigation and drip irrigation systems have been explored. Another promising approach is the use of water absorbing polymers (WAPs), which can act as small reservoirs to store large amounts of water and mitigate the effects of drought. This study investigated the performance of fly ash-based water absorbing polymers (FA-WAPs) in vegetation (grass species) drought tolerance, a topic that has not been explored in existing literature.

WAP alters soil properties and affects vegetation by influencing water movement and root water uptake (RWU). Previous studies have shown that WAPs affect soil hydrology and plant water uptake, remains understudied with WAP amendment. Therefore, this study aims to: 1) Evaluate the efficacy of FA-WAPs under drought stress conditions. 2) Develop RWU profiles of WAP amended soils. By analyzing these factors, the study aims to deepen the understanding of the role of WAP in improving soil water retention and supporting plant survival during water deficits. These findings will contribute to sustainable agricultural practices in drought-prone regions and provide potential solutions for water resource management in arid environments.

2 MATERIALS AND METHODOLOGY

Pot experiments were conducted to evaluate the efficacy of fly ash-based water-absorbent polymers (FA-WAPs) on Cynodon dactylon, a turfgrass widely used in green infrastructure. This study was conducted in a controlled greenhouse environment at Indian Institute of Technology Guwahati using agricultural soil (as per ASTM standard, the soil is classified as sandy soil). FA-WAP was synthesized in-house and applied at concentrations of 0.2% and 0.4%, and a control group (no WAP addition) was set up for experimentation.

Soil water retention curve (SWRC) was monitored using TEROS 21 and EC5 sensors installed in each pot. The data collected were analyzed to determine soil water content (SWC) and plant available water (PAWC). For the analysis of root water uptake (RWU), van Genuchten parameters were derived from the SWRC data using RETC software. These parameters were then used as input to the HYDRUS software to simulate the RWU curves. This experimental setup allowed for a comparative evaluation of the effects of FA-WAP on soil water

retention and plant water uptake under controlled soil.

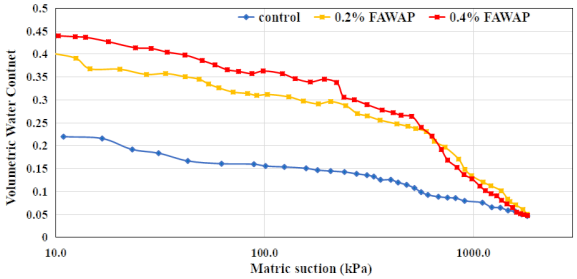


Fig. 1. SWRC of WAP amended soil in comparison with bare soil

3 RESULTS AND DISCUSSION

The Soil Water Characteristic Curve (SWCC) is essential for analyzing unsaturated soil properties, providing insights into hydraulic conductivity, volume changes, solute transport, and thermal diffusivity. Key parameters derived from the SWCC include saturated water content, residual water content, plant available water content (PAWC), and permanent wilting point.

Figure 1 shows the SWRC of the control soil, 0.2% FA-WAP, and 0.4% FA-WAP amended soil. The results show that the WAP-amended soil has improved the water retention capacity compared to the control soil. This improvement is attributed to the expanded WAP filling the pore space, thereby retaining more water. In addition, FA-WAP significantly delayed the time to wilting an indicator of extended water availability. The control soil wilted 34 days, while 0.2% and 0.4% FA-WAP extended this time to 39 and 44 days, respectively. This suggests that higher WAP concentrations can enhance drought resistance by increasing soil water retention. The results suggest that FA-WAP has the potential to become a sustainable solution to alleviate water shortages in arid and semi-arid regions.

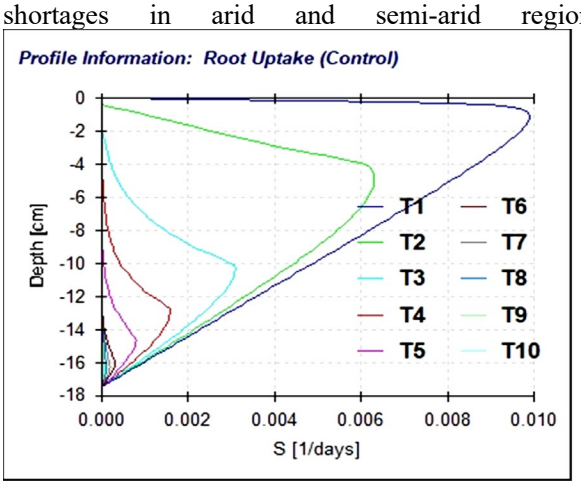


Fig. 2. Root water uptake information with depth for bare soil

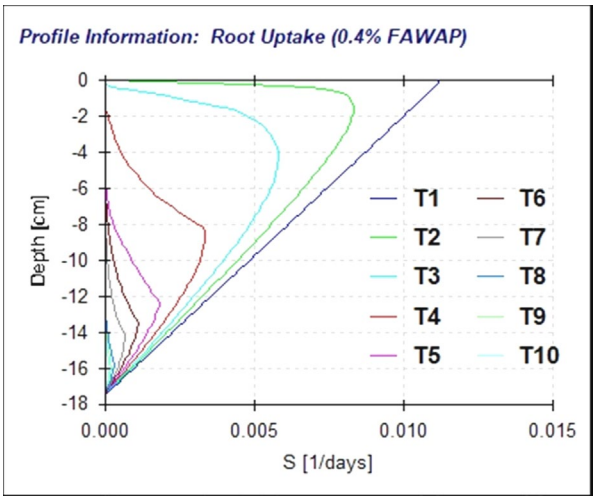


Fig. 3. Root water uptake information with depth for bare soil

Understanding water movement in the vadose zone (partially saturated region between the surface and the water table) is essential for assessing soil-plant-water dynamics. In this study, root water uptake (RWU) curves (Figures 2 and 3) were calculated for treated soil with different concentrations of WAP using HYDRUS-1D, a numerical model for simulating water flow in unsaturated porous media. The model solves the Richards equation governing soil water movement:

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial x}\left[K\left(\frac{\partial h}{\partial x}+1\right)\right]-S$$

Where, h is the hydraulic head (cm), θ is the volumetric water content (cm^3/cm^3), t is time (days), x is the spatial coordinate (cm), and K is the unsaturated hydraulic conductivity function (cm/day). S is the sink term in the flow equation ($cm^3/cm^3/day$) used to calculate root water uptake. RWU was simulated using the Feddes water uptake reduction model with time-varying boundary conditions accounting for precipitation (3 cm/day for the first two days), evaporation (0.4 cm/day), and transpiration (0.1 cm/day).

4 CONCLUSIONS

This study showed that FA-WAP amendment significantly improved the water holding capacity of soil under drought conditions. Increasing WAP concentration increased soil water content (SWC), plant available water (PAWC), and wilting point, prolonging plant survival under water stress. In addition, root water uptake (RWU) was higher in WAP-amended soils compared to control soils, confirming its effectiveness in mitigating the effects of drought. These findings highlight the potential of FA-WAP as a sustainable solution for water conservation in arid and semi-arid regions.

Influence of Deep Foundations on High-Rise Buildings during a Seismic Impact

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ABSTRACT

The seismic performance of high-rise buildings supported by deep foundations, specifically raft-pile systems, is critically influenced by the stratigraphy of the underlying soil. This study investigates the behavior of such buildings founded on a two-layer soil profile consisting of an upper 30-meter layer of loose sand overlying 190 meters of coarse sand. Numerical simulations and experimental research indicate that raft-pile foundations act as an integrated system, distributing structural loads between the raft, piles, and subsoil, thereby improving overall bearing capacity and reducing settlements compared to conventional pile-only foundations. However, the presence of piles embedded in these layered soils introduces distinct dynamic soil-structure interaction effects during seismic events. In particular, the flexibility of the loose sand layer amplifies the deformation transmitted to the superstructure, resulting in increased displacement at the top of the building relative to raft-only or shallow foundations. This behavior is intensified at the pile heads due to kinematic interactions at the interface between the loose and coarse sand strata, where a marked contrast in soil stiffness alters seismic wave propagation. Furthermore, the acceleration profile along the height of the structure changes significantly under seismic loading. The amplification of seismic acceleration is generally observed at upper stories, with peak floor acceleration ratios potentially exceeding values typically recommended by design codes, thereby impacting both structural elements and sensitive nonstructural components. These findings highlight the necessity of careful design consideration of foundation type and soil layering in order to mitigate excessive building deformation and acceleration demands, ensuring the safety and serviceability of high-rise buildings in seismic zones with stratified soils.

Keywords: Deep foundations, Raft-pile foundation, Loose sand, Coarse Sand, kinematic interaction, Acceleration amplification.

1 INTRODUCTION

Seismic design of high-rise buildings critically depends on the interaction between the superstructure, foundation system, and supporting soil. Deep foundations such as piled-raft systems are increasingly employed on soft soil deposits to enhance bearing capacity and control settlements. This study aims to assess the influence of piles on the seismic response of high-rise buildings founded on layered soils, complying with Eurocode 8 requirements. The soil profile consists of an upper loose sand layer approximately 30 m thick, overlying around 190 m of relatively stiff coarse sand.

The research focuses on comparing seismic responses of the building with different foundation types: shallow foundation with jet grouting, piled raft with jet grouting, and piled raft without jet grouting. A detailed finite element method (FEM) analysis using PLAXIS 3D was employed to simulate the nonlinear dynamic soil-structure interaction (SSI) effects during seismic excitation. The study evaluates deformation, acceleration response distribution throughout the structure, and shear forces at the foundation interface, comparing numerical results with simplified design calculations outlined in Eurocode 8 (EN 1998-1:2004) [1][6].

2 OBJECTIVES AND PROBLEM STATEMENT

The core objectives addressed in this study are:

To analyze the soil mass behavior during seismic excitation and validate the results against equivalent linear methods (DEEPSOIL).

To evaluate the seismic response of the high-rise building with three foundation configurations under strong motion.

To calculate the shear forces at the base of the foundation from numerical results and verify compliance with Eurocode 8 lateral force methods.

To investigate the dynamic amplification of acceleration in the superstructure and quantify pile-induced deformation effects.

The analysis assumes the following simplifications:

Soil parameters represent a static condition of the soil mass; liquefaction and groundwater table effects are

excluded.

Kinematic interaction and bending moments in piles are not explicitly analyzed, as per Eurocode assumptions.

Structural elements are modeled with elastic isotropic behavior.

Rayleigh damping is used for soil damping characterization.

3 METHODOLOGIES

3.1 Modeling and Soil Profile

A 3D soil domain with dimensions 270 m × 270 m × 220 m was modeled, comprising a 30 m loose sand layer underlain by 190 m coarse sand, both represented with the nonlinear HSsmall material model to capture hysteretic soil behavior at small strains. Material parameters for both soil layers were defined according to laboratory test data. The building is modeled with a 480 m tall high-rise with a base area of 5929 m², incorporating a two-floor, 10m deep underground structure. Three foundation types are modeled: shallow foundation with jet grouting, piled raft with jet grouting, and piled raft only, with piles 80 m in length embedded into the coarse sand.

3.2 Seismic Loading and Boundary Conditions

Input motion consists of artificially generated accelerograms with peak acceleration 0.4g, frequency content up to 2 Hz, and 5% damping ratio. Fixed boundaries are set at lateral soil limits, with input motion applied at the bedrock level. The mesh was refined near the foundation and structure to capture SSI effects accurately.

3.3 Dynamic Analysis

Time-history nonlinear dynamic analyses were conducted over a 10-second duration in PLAXIS 3D, extracting accelerations and displacements at multiple points including the soil base, pile heads, foundation interface, and selected superstructure levels.

4 RESULTS AND DISCUSSION

4.1 Soil Free-Field Response

PLAXIS 2D nonlinear dissipative soil response shows peak surface acceleration amplification of approximately 2.7 times input bedrock motion, consistent with DEEPSOIL linear analyses, validating the model accuracy for site response.

4.2 Structural Response

Acceleration and displacement time histories indicate significant increases in the superstructure's top acceleration when piles are present. The piled raft system increases peak accelerations at the roof by over 63% compared to shallow foundations. The higher flexibility of the piles interacting with the loose sand layer causes incremental deformation at the building top, highlighting critical SSI influences on seismic performance.

4.3 Shear Forces at Foundation Interface

The PLAXIS 3D output reveals that the pile presence increases shear forces at the base interface by approximately 10% relative to shallow foundation conditions. Eurocode 8 lateral force method estimates maintain close agreement with numerical values for piled raft configurations but slightly overestimate shear forces for shallow foundation models, indicating possible conservatism in simplified design for such soils.

4.4 Implications

The findings emphasize the necessity of advanced SSI consideration in seismic design of high-rise foundations on soft soils. Pile foundations should be carefully designed to accommodate increased seismic shear loads and deformation demands, especially when interfacing layered soil profiles.

5 CONCLUSIONS AND FUTURE WORK

Dynamic soil-structure interaction significantly influences high-rise building seismic response, particularly in layered soils with loose sand overlays.

Piled raft foundations with jet grouting reduce overall deformations but raise base shear demands compared to shallow foundations.

Current design codes provide conservative but generally accurate estimates of base shear forces for pile systems in soft soils.

Further research is required to include kinematic pile bending effects, liquefaction potential, and nonlinear structural behavior to enhance design accuracy.

Experimental validation via shake table and centrifuge tests is planned to complement numerical findings.

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Experimental study of the physico-mechanical properties of bedrock soils

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ABSTRACT

The study of the physico-mechanical properties of bedrock soils plays an important role in the construction and mining industries, as accurate assessment of these properties ensures the stability and durability of structures interacting with such soils. In this work, experimental studies were conducted on samples of granodiorite and granitoid gneiss to determine the correlation between physical and mechanical (strength and deformation) characteristics of bedrock soil. The compressive strength limit under uniaxial compression was determined by the method of failure of tile samples with heights ranging from 10 to 15 mm using co-axial punches. The tensile strength limit under uniaxial tension was determined by the method of failure of samples (at least 10 mm high) using opposing spherical indenters. Based on the results of determining the compressive and tensile strength limits, a strength passport for the bedrock soil was constructed. To determine the deformation characteristics, tests were conducted on rock samples in a triaxial (volumetric) compression device. The results of the study showed that the strength and deformation characteristics of granodiorite and granitoid gneiss are independent of the sampling depth. An analysis of the results was also conducted, allowing for a certain degree of accuracy in determining the mechanical properties of bedrock soil based on its physical characteristics. The obtained results can be used to create more accurate methods for evaluating the properties of bedrock and conducting additional research using other bedrock soils. This, in turn, will improve the reliability of construction structures and contribute to more efficient use of natural resources. This, in turn, will significantly increase the strength and durability of building structures, ensuring their high reliability in various operating conditions. This approach will also contribute to a more rational and efficient use of natural resources, which, in turn, will lead to the optimization of construction processes and lower costs of materials and energy.

Keywords: geotechnical engineering, bedrock soil, physico-mechanical properties, laboratory test, compressive strength, tensile strength, triaxial compression device

1 INTRODUCTION

In this study, granodiorites and granite-gneisses were analyzed to assess their physical and mechanical properties and to explore correlations among different parameter sets. The main objective was to propose a methodology for determining these physical and mechanical parameters based on experimental results.

2 METHODS

Experimental research was conducted on granodiorite and granite-gneiss samples. The studied parameters included both physical and mechanical characteristics. Physical parameters were determined by laboratory methods. Mechanical parameters (strength and deformation) were examined on samples in fully water-saturated and air-dried conditions. The uniaxial compressive strength was measured by testing disk-shaped samples (10–15 mm in height) using flat, coaxial punches until failure. The uniaxial tensile strength was measured by testing samples (with a height of at least 10 mm) using opposing spherical indenters. The deformation characteristics of the rock were determined via triaxial compression tests on specimens sized 42 × 84 mm.

3 RESULTS AND DISCUSSION

After the experimental investigations on granodiorite and granite-gneiss samples were completed, the obtained characteristics were analyzed. A correlation analysis between parameter sets showed the following conclusions: Sampling depth does not affect the physical and mechanical properties of the soil. The shear strength had a strong correlation (86%) with the uniaxial

compressive strength and a moderate correlation (59%) with the uniaxial tensile strength. The deformation modulus under loading had a strong correlation (96%) with the deformation modulus under unloading. No strong correlations could be established between the physical and mechanical parameters. Further research is planned on the characteristics of various rock types to broaden the range of analyzed materials.

4 CONCLUSIONS

Based on the conducted tests, it was possible to establish relationships among different parameter sets. To obtain a complete picture of the correlations among the characteristics of rock soils, similar studies should be carried out for other types of rocks.

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Experimental investigation of side resistance in bored piles embedded in rock

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ABSTRACT

Bored piles embedded in rock or cutting through it are often utilized as foundations for high-rise buildings. As practice shows, the resistance of bored piles is mainly provided by the side surface. The resistance values of the pile shaft surface depend on the type of rock, rock strength, shaft roughness, and the strain level during testing. Most of the methods for assessing bored pile resistance proposed in the literature consider only the unconfined compressive strength of the rock and do not account for the pile strain under load or the roughness of the pile shaft. This article presents the results of laboratory and in-situ tests demonstrating the dependence of shaft resistance on the unconfined stress of the rock and the roughness of the shaft. A semi-analytical solution that takes into account the dilatancy of the rock due to plastic shear strain of the pile shaft is also provided. Recommendations for evaluating the bearing capacity and conducting load tests for bored piles in rocky soils are given.

Keywords: bored pile, side resistance, rock

1 INTRODUCTION

The bearing capacity of bored piles takes into account their side surface and base resistances. As pile testing practice shows, the bored piles resistance is mainly realized by the side surface. The base resistance is realized after significant pile displacement followed by side surface failure. Horvath et al., 1979, 1983 showed that mobilization of the side surface resistance of large diameter piles could be achieved when the displacements equal 5-6 mm. Ng et al., 2001 showed that at S/D≤0.4 % an elastic socket (pile) behavior at working load is observed. Furthermore, load-displacement relationship becomes nonlinear and pile side resistance is fully mobilized at S/D ≈ 1%. However, the pile base resistance starts to mobilize at S/D ≈ 5% (O'Neill et al., 1996). Numerous pile test results show that such displacement values are almost never implemented in rocks. In this regard, the bearing capacity of piles in rock masses is obtained from their side surface resistance.

2 DETERMINATION OF THE SIDE SURFACE RESISTANCE OF BORED PILES

The side surface resistance of bored piles in rock f_s can be found according to the following simple relationship:

$$\frac{f_s}{p_a} = \alpha \left(\frac{R_c}{p_a}\right)^m \quad (1)$$

where R_c is unconfined compressive strength, p_a = 0.1 MPa is atmosphere pressure, α and m are empirical coefficients.

Table 1 summarizes dependencies presented by the authors.

Table 1. Empirical coefficients α and m for equation (1)

Authors	α	m
Rosenberg and Journeaux, 1976	1.19	0.52
Horvath, Kenney, Kozicki, 1983	0.63-0.95	0.5
Rowe и Armitage, 1984	1.42	0.5
Carter and Kulhawy, 1988	0.63	0.5
Reese and O'Neill, 1999	0.65	0.5
Zhang and Einstein, 1998	1.26	0,5
Zhang and Einstein, 1999	0,63	0.5
Ng, Yau, Li, 2001	0.6	0.5
Kulhawy, Prakoso, Akbas, 2005	1	0.5

As analysis shows, most of the existing methods allow estimating side surface resistance of piles depend on unconfined compressive rock strength and do not consider the pile strain. Practice shows that pile tests in rocks are performed at rather low loads which does not allow to estimate their real (maximum) bearing capacity. In this regard, most pile solutions have significant design margins and open up the opportunities for project optimization.

3 GENERAL DESCRIPTION OF INVESTIGATED ROCKS AND PILES

Experimental data included the results of a series of pile tests on compression and pull-out loads collected at

different construction sites in Russia, Turkey, the USA and other countries. The following rocks were carried out in this study: shale, limestone, sandstone, gabbro, granite, porphyrite, etc. The unconfined compressive strength was 0.23-55.0 MPa. The tested bored piles were 0.43-1.83 m of diameter and 0.5-22.2 m of embedded length. Test loads were as high as 33.3 MN. The maximum displacement of piles S was 0.12-130.0 mm which allowed to measure mobilized side surface resistance fsm = 0.01-5.40 MPa.

4 METHODS AND RESULTS

The correlation and regression statistical data analysis technique was employed using MS Excel and IBM SPSS Statistics. The following bearing capacity parameters were analyzed:

- mobilized side surface resistance f_{sm};
- normalized mobilized side surface resistance f_{sm}/p_a;
- ratio of mobilized side surface resistance to unconfined compressive strength $f_{sm}/R_{c;}$
- ratio of pile displacement to pile embedment in rock (relative pile strain) S/L_r;
- the ratio of pile displacement to pile diameter S/D.

The analysis revealed that unconfined compressive rock strength R_c and R_c/p_a (Fig. 1) as well as the length of embedment in rocky soils L_r/D greatly influence mobilized pile side surface resistance f_{sm} and f_{sm}/p_a.

The relationships between mobilized side surface resistance and pile displacements S and S/D are weak and not statistically insignificant. At the same time, the value of S/L_r affects the mobilized resistance f_{sm} and f_{sm}/p_a, and is also inversely proportional to its embedment in the rock L_r/D.

In general, the following conclusion can be drawn based on the performed analysis of the total sample:

- mobilized side surface resistance of piles depends on soil strength, the length of pile embedment in rock and the value of S/L_r ratio;

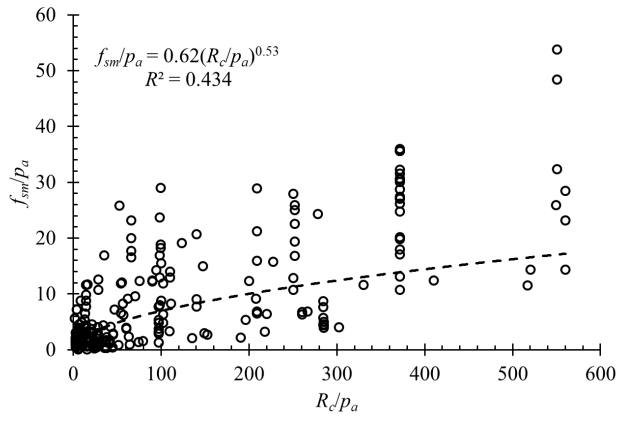


Fig. 1. Relationship between normalized mobilized side surface resistance f_{sm}/p_a and normalized unconfined compressive strength R_c/p_a

The correlation analysis showed that S/D ratio does not assess an significant influence on f_{sm}. In addition, the pile displacement and the measured f_{sm} resistance significantly depend on the length of pile embedment in the rock mass. According to the authors, the relative pile strain S/L_r is more suitable for categorizing the strain level. It is applied in classical definition of strain of a pile since the width of the interaction zone of a single pile with the soil is comparable to the length of its embedment (Randolph, 1994, Fleming K. et al., 2009). In this regard, the following pile strain classification (strain index SI) is proposed: I – $S/Lr \geq 1\%$; II – $0.5\ \% \leq S/Lr < 1\ \%$; III – remaining results. Such classification will enable one to assess pile bearing capacity more accurately and exclude data with insignificant strains.

Figure 2 shows the results of the joint effect of R_c/p_a and SI on f_{sm}/p_a values. The higher is S/L_r, the higher is the resistance f_{sm}/p_a; lower values of f_{sm}/p_a generally correspond to minimum values of strains. It indicates that the relationships based on these data have excessive safety reserves.

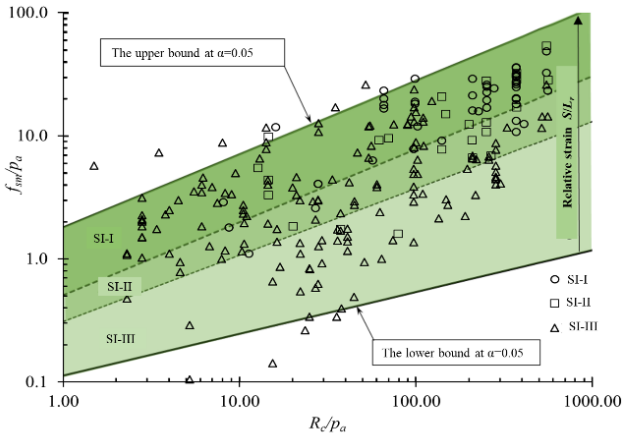


Fig. 2. Influence of normalized unconfined compressive strength R_c/p_a on normalized mobilized side surface resistance f_{sm}/p_a as a function of strain index

Figure 2 provides quantitative and qualitative assessment of the effect of R_c/p_a on the f_{sm}/p_a, depending on deformation level. The equation obtained may be useful for practical applications:

$$\frac{f_{sm}}{p_a} = \alpha \left(\frac{R_c}{p_a}\right)^m \quad (2)$$

where α and m are empirical coefficients. The values of α and m depend on strain index, their values are respectively 0.77 and 0.6 for SI-I (R^2=0.68); 0.75 and 0.54 for SI-II (R^2=0.51) and 0.93 and 0.34 for SI-III (R^2=0.22).

The obtained empirical coefficients do not fundamentally differ from those obtained by other authors (Table 1). However, the empirical coefficients and strain level increase simultaneously. Considering the processed data of large number of pile load tests performed in rocks, the following semi-empirical dependence was proposed:

$$\frac{f_{sm}}{p_a} = a \left(\frac{S}{L_r}\right)^b \left(\frac{R_c}{p_a}\right)^c \quad (3)$$

Empirical coefficients a, b and c depend on rock genesis and are given in Table 4.

Equation (3) provides side surface resistance of a pile

depending on the expected pile strain level that can be calculated using established solutions (Fedorovsky V., 1974; Randolph M. and Wroth C., 1978). However, pile load testing is still necessary even with the above approach. Moreover, the dependence enables to reduce bearing capacity excessive reserves of piles that were used up in empirical dependencies obtained from underloaded pile testing results.

5 CONCLUSIONS

Based on correlation analysis it is established that mobilized side surface resistance of piles depends on soil strength, pile embedment length and differential pile displacement value S/L_r. At the same time, absolute displacement values S and displacement-to-diameter ratio S/D have little influence on the value of mobilized pile resistance.

A classification of bored piles by strain index depending on differential pile displacement is proposed: SI-I - $S/L_r \geq 1\%$; SI-II - $0.5\ \% \leq S/L_r < 1\ \%$; SI-III - other results. Such classification enables to assess accurately the bearing capacity of piles in rocks and exclude underestimated values of f_{sm} with insignificant strains. The ranges of mobilized side surface resistance of piles for various strain indices are established.

The combined dependence to estimate mobilized side surface resistance considering rock strength and pile strain is proposed. The dependence allows to get a better understanding of load transfer through borehole lateral walls in consideration of pile displacement and the embedment depth in rock

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TC-305 Geotechnical Infrastructure for Megacities and New Capitals

Geosynthetic-reinforced retaining walls for flooding scour protection

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ABSTRACT

In 2024, massive floods swept across Eurasia, overwhelming many aging hydraulic structures that were not designed to withstand such extreme conditions. Geosynthetic reinforcement can significantly enhance scour resistance while providing a rapid and cost-effective solution for slope restoration. This paper reviews industry research on the topic and presents case studies demonstrating the performance of geosynthetic-reinforced soil embankments under scour conditions.

Keywords: scour resistance, reinforced soil wall, geosynthetics

1 INTRODUCTION

The increasing frequency and intensity of extreme weather events, exemplified by the catastrophic 2024 Eurasian floods, have exposed critical vulnerabilities in aging hydraulic infrastructure. Traditional retaining structures, often designed for historical hydrological conditions, frequently fail under unprecedented scour forces, leading to costly repairs and prolonged downtime. Geosynthetic-reinforced soil (GRS) retaining walls offer a resilient alternative, combining flexibility, rapid construction, and enhanced scour resistance as presented by Tatsuoka (2012). By integrating geosynthetic layers into soil matrices, these structures distribute stresses more effectively and mitigate erosion-driven failures. This paper examines the role of GRS technology in flood-prone environments, with a focus on its application in post-flood recovery and scour protection. Three case studies from diverse geographical and hydrological contexts are analyzed to demonstrate the adaptability and performance of GRS systems under real-world conditions.

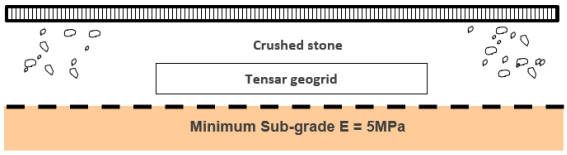
2 CASE STUDIES

Section 3 presents three case studies highlighting the performance of geosynthetic-reinforced soil structures under scour conditions.

2.1 Ust-Luga Port

Between 2006 and 2008, a new port was constructed in the Leningrad Region on the Baltic Sea. Today, Ust-Luga ranks as Russia's second-largest port by cargo volume, but its initial development involved hydraulic filling of fine sand to create roads and platforms at sea level. A polypropylene stabilizing geogrid was installed at the base of these structures (Figure 1).

a) roads



b) container areas

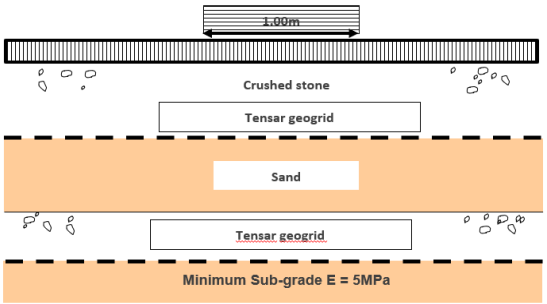


Fig. 1. Cross-sections of roads and container areas.

During hydraulic filling in 2006, an overflow event washed away unstabilized fill. However, structures reinforced with geogrids remained intact and required only minor repairs (Figure 2).



Fig. 2. Overflow impact.

2.2 Dibba Road UAE

The Dubai–Fujairah Freeway Project (2003) included 45,000 m² of reinforced soil retaining walls as described by Wills (2011). A section of the road traversed a mountainous area prone to mudflows (Figure 3).



Fig. 3. Mudflow impact on a reinforced soil wall.

Gabion mattresses were installed as scour protection in front of most walls, ensuring their stability. However, one unprotected section experienced undermining of the facing leveling strip, causing localized deformation. Despite this, the reinforced soil structure remained intact, and the facing was successfully restored.

2.3 Moscow City Approach Road, Russia

In 2023, an 85-meter-long reinforced soil retaining wall (average height: 5 m) was constructed near Moscow in under two weeks. However, due to rushed construction, the transition zone between the reinforced soil structure and a traditional embankment lacked proper shoulder protection. Consequently, rainwater runoff from the pavement slab eroded the embankment (Figure 4).



Fig. 4. Scour damage at the road shoulder.

The geogrid reinforcement prevented catastrophic failure, allowing timely repairs. A conventional reinforced concrete wall would not have provided the same resilience.

3 CONCLUSIONS

Geosynthetic-reinforced soil retaining walls have proven to be a cost-effective and resilient solution for scour protection in flood-prone regions. Key findings include:

- Scour Resistance: GRS systems effectively redistribute hydraulic and gravitational stresses, preventing catastrophic failure even when localized erosion occurs.
- Rapid Deployment: The modularity of GRS walls enables swift construction and post-disaster repairs, critical for maintaining infrastructure functionality during emergencies.
- Design Imperatives: Success hinges on holistic design, including transitional zone protection (e.g., gabions, vegetation) and adherence to construction protocols.

The 2024 floods underscore the urgent need to modernize infrastructure using adaptive materials like geosynthetics. Future research should prioritize long-term monitoring of GRS performance under cyclic flooding and standardized protocols for integrating these systems into existing flood defense networks. As climate variability intensifies, GRS technology represents a pragmatic pathway toward sustainable, disaster-resilient infrastructure.

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Experimental study of the influence of geosynthetic materials on the strength characteristics of dispersed soils

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ABSTRACT

The use of geosynthetic materials is a widespread method of reinforcing various types of dispersed soils on construction projects. Due to the large number of different geotechnical projects where this solution is applied, it is necessary to investigate more thoroughly the stress-strain state of the reinforced soil mass to analyse its strength characteristics. For this purpose, it is necessary to evaluate the impact of geosynthetic materials on the strength parameters of dispersed soils. Such parameters include the angle of internal friction and cohesion, as well as the strength reduction factor, which allows estimating the reduction of strength parameters at the interface between the dispersed soil and geosynthetic material. The aim of this experimental study was to investigate the influence of the type of geosynthetic material on the strength characteristics of the dispersed soil, which is important for assessing the stability of soils and their reinforcement. To realise this objective, a series of tests were carried out using a large-scale direct shear apparatus. Gravelly sand was selected as the test specimens. The tests were carried out using a consolidated-drained scheme at three different normal stresses with a constant shear rate. Special attention in this study was paid to the different positions of the geosynthetic material in the direct shear apparatus. Two positions of the reinforcing material were considered: longitudinal and transverse. These two various options are important to consider because the geosynthetic material can interact differently with the soil depending on the orientation of the fibres or filaments. This approach allows a more accurate assessment of how geosynthetic materials affect the strength characteristics and behaviour of dispersed soil under different conditions. The results of the study were used to plot ‘tangential stresses - shear strains’ and ‘ultimate tangential stresses - normal stresses’ graphs, and to determine the strength reduction factors resulting from the interaction of sandy soil with geosynthetic material.

Keywords: geogrid, sandy soil, geosynthetics, strength reduction coefficient, laboratory experiments, direct shear apparatus.

1 INTRODUCTION

The study of the strength characteristics of various soils is an important task in geotechnics, as these characteristics significantly influence the design and construction of various structures. The strength characteristics of soil can be determined using a triaxial compression test, a simple shear test, or a direct shear test apparatus. In this paper, we will focus on the determination of strength characteristics using the direct shear test apparatus. Experimental studies with the direct shear method allow to acquire important data of the ultimate states of soils and their behavior under different loads. It should also be noted that the design of the direct shear apparatus allows testing soils with the use of various geosynthetics, such as geogrids, geotextiles, and other modern composites. The application of geosynthetics helps to increase the reliability of structures by improving the mechanical properties of the soil. These innovative materials have become an integral part of modern engineering practice, enabling the creation of safer and more durable structures.

2 METHODOLOGY

In order to determine the strength characteristics of sandy soil, as well as the strength reduction factors (Rint) at the "sand-geogrid" interface, experimental studies were conducted using a large-scale direct shear apparatus manufactured by NPP "Geotek" with sample dimensions of 300x300x150 mm. This setup is designed for testing sandy and coarse-grained soil samples under vertical and shear loads up to 100 kN.

To achieve the set goal, the experimental studies

were divided into several stages.

The first stage was aimed at determining the strength characteristics of the sandy soil.

The second stage focused on determining the strength reduction factors (Rint) at the "sand-geogrid" interface, as well as the strength characteristics resulting from this interaction.

The third stage of the experimental research similarly involved determining the strength reduction factors (Rint) at the "sand-geogrid" interface and the strength characteristics, but with the geogrid placed at a 90-degree angle to its original position.

The direct shear tests were conducted on sandy soil samples using a consolidated-drained scheme with a moisture content of 7.8% and a maximum dry unit weight of $\rho d = 2.19$ g/cm³.

3 FINDINGS

The results of the first series of experimental studies provided the strength characteristics of gravelly sand, with the following values: cohesive strength c = 0.0 kPa; internal friction angle $\varphi = 43.155°$.

The results of the second series of experimental studies provided the strength characteristics of gravelly sand, as well as the values of the strength reduction factors (Rint) depending on the applied vertical stress. The test results yielded the following strength characteristics: cohesive strength c = 0.0 kPa; internal friction angle $\varphi = 34.598°$. The values of the strength reduction factor at peak values ranged from 0.760 to 0.893; at residual values, they ranged from 0.916 to 0.971.

The results of the third series of experimental studies provided the strength characteristics of gravelly sand, as well as the values of the strength reduction factors (Rint) depending on the applied vertical stress. The test results yielded the following strength characteristics: cohesive strength c = 0.0 kPa; internal friction angle $\varphi = 42{,}384°$. The values of the strength reduction factor at peak values ranged from 0.986 to 1.369.

The analysis of the results showed that the highest internal friction angle was obtained during tests on gravelly sand samples without a geogrid. The lowest value of the internal friction angle was obtained during tests with the geogrid fixed in the large-scale direct shear transversely, as presented in the text of this study.

It should be noted that when the geogrid was fixed transversely in the direct shear apparatus, the strength reduction factors (Rint) increased compared to the strength reduction factors (Rint) obtained when the geogrid was fixed longitudinally.

4 CONCLUSIONS

Three series of experimental studies were conducted in a large-scale direct shear apparatus to determine the strength characteristics of gravelly sand, as well as to determine the strength reduction factors (Rint) at the "sand-geogrid" interface.

Since the experimental studies were conducted only on one type of soil (gravelly sand), further research is required to investigate the influence of geosynthetic type, applied normal stresses, and other factors affecting the change in soil strength characteristics. Additionally, it is necessary to conduct further studies on various types and kinds of soils and geosynthetics to refine the actual values of the strength reduction factors.

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Deformation characteristics of sand with liquefaction history considering sand particle orientation

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ABSTRACT

It has been widely reported that reliquefaction engenders under smaller seismic events than previous liquefaction. Once a sand undergoes liquefaction, the initial sand's fabric (skeleton structure) formed during sedimentation process will be completely destroyed and changed during liquefaction and re-sedimentation process. Therefore, reliquefaction resistance is influenced by not only density but also structure change. To fully understand the mechanism of reliquefaction, this study focused on effects of sand particle orientation on deformation characteristics of sand in small to large strain range. The specimens were prepared using Toyoura sand with three initial depositional angles of 0°, 45°, and 90°. Then, two kinds of mechanical tests, those are bender element (BE) tests to measure initial shear modulus (G_0) and local small-strain (LSS) tests to obtain secant shear modulus at various strain range were conducted using a triaxial apparatus. Two types of specimens, which are before liquefaction (without liquefaction history) and after liquefaction (with liquefaction history), were used in those tests. The particle orientations before and after liquefaction were measured using an image microscopy. The results obtained are follows. The BE tests showed increased G_0 with increase of initial orientation angles (0° to 90°) before liquefaction. However, this trend became indistinct after liquefaction. Microscopic analysis for the particle orientation revealed that particle orientation tended to concentrate on 90° orientation in all cases after liquefaction. LSS test results additionally indicate degradation of secant G (G_{sec}) with shear strain is greater in the specimens with larger orientation angles. Those deformation characteristics had close relations with sand particle orientation.

Keywords: liquefaction, bender element, local small strain, triaxial test, sand, anisotropy

1 INTRODUCTION

Reliquefaction defined as repeated liquefaction, which is sometimes triggered by smaller subsequent seismic events, has been reported in past earthquakes (Ishihara, 1996). During liquefaction and re-sedimentation, the particle orientation (i.e., soil structure) is significantly altered and is affecting the initial shear modulus (G_0) and deformation behavior represented by secant shear modulus (G_{sec}). Although these structural changes influence reliquefaction resistance, studies on liquefaction history considering particle orientation are limited. In this study, a series of triaxial tests, incorporating bender element (BE) and local small strain (LSS) measurements, was conducted to investigate the effects of liquefaction on deformation characteristics within a strain range between about 0.0001% and 1%. Two-dimensional optical digital microscopy was also performed before and after liquefaction to observe fabric rearrangement.

2 TESTING MATERIAL, APPARTUS, AND PROCEDUDE

Specimens of Toyoura sand were prepared by air pluviation method at depositional angles of 0°, 45°, and 90° to achieve a relative density (D_r) of approximately 60%. A series of monotonic drained (CD) triaxial tests was conducted using an automated apparatus equipped with LSS and BE measurement systems, described by Toyota et al. (2017) and Pavel et al. (2025), respectively, to investigate the deformation characteristics of sand.

Specimens were saturated and isotropically consolidated under an effective confining pressure (σ_c') of 50 kPa (before liquefaction). Cyclic undrained loading was applied until the excess pore water pressure ratio reached 95%. Then, the specimens were isotropically consolidated again at $\sigma_c' = 50$ kPa (after liquefaction). BE tests, LSS tests in compression, and microscopic observations were conducted in before and after specimens. LSS tests cannot performed in the same

specimens involving microscopy observation after liquefaction because of removal of membrane.

3 TESTING RESULTS AND DISCUSSIONS

3.1 BE and LSS tests results

Figure 1 shows that, before liquefaction, the G_0 obtained from the BE test increases with increasing depositional angle (α) from 0° to 90°. However, this trend becomes indistinct after liquefaction, indicating a loss of the initial fabric anisotropy.

Figures 2 and 3 present the degradation of G_{sec} obtained from LSS tests, including BE results, before and after liquefaction, respectively. Before liquefaction, in the small-strain region ($\varepsilon_s < 0.001\%$), G_{sec} is largest at 90° and smallest at 0°. However, when shear strain is larger than 0.1%, this trend is reversed. After liquefaction, this trend changes: In the small strain region ($\varepsilon_s < 0.001\%$), G_0 and degradation of G_{sec} become almost independent of the depositional angle, while G_{sec} of 0° is slightly greater than that result of 90° at larger strains ($\varepsilon_s > 0.001\%$).

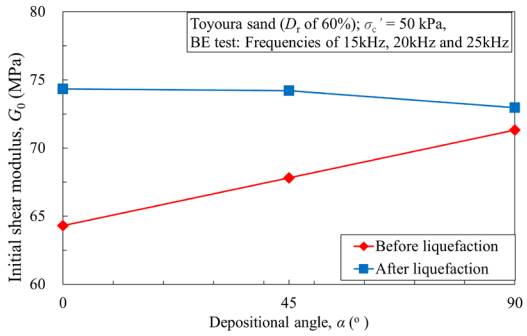


Fig. 1. Initial shear modulus of sand in BE test.

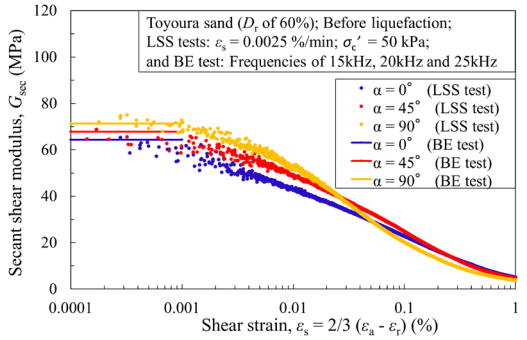


Fig. 2. Secant shear modulus of sand (Before Liquefaction).

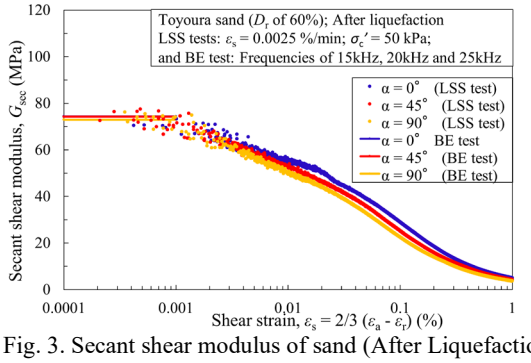


Fig. 3. Secant shear modulus of sand (After Liquefaction).

3.2 Microscope results

Figures 4 and 5 respectively show the distribution of inclined angles θ, which indicate direction of major axis of sand particles, before and after liquefaction in the specimens with α of 0°, 45°, and 90°. Before liquefaction, the concentration of θ matches with α, indicating that initial structure was formed in the specimens as expected. After liquefaction, however, particle orientations converge toward the same trend: most particles aligning near 90°, suggesting a homogenized vertical fabric. This result suggests that liquefaction induces significant changes in particle orientation, which induce change of properties in G_0 or G_{sec}.

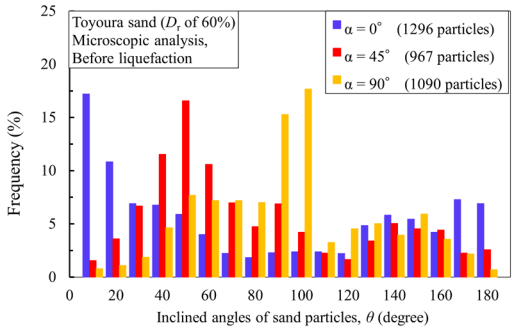


Fig. 4. Particle inclination distribution (Before Liquefaction).

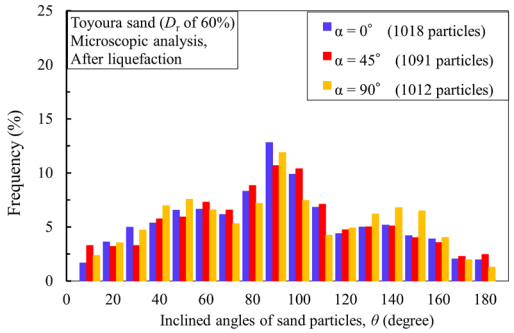


Fig. 5. Particle inclination distribution (After Liquefaction).

4 CONCLUSIONS

This study investigated the effects of α and liquefaction history on the deformation characteristics of Toyoura sand. Before liquefaction, specimens with higher α exhibited greater G_0 and greater degradation of G_{sec}. After liquefaction, particle orientations converged toward a mainly vertical alignment, and the differences in mechanical behavior among specimens diminished, particularly in the small strain range.

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Consideration of input data influence for geotechnical parameter identification of rock dynamics

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Keywords: parameter identification, input data magnitude, seismic wave data, dynamic response analysis

1 INTRODUCTION

The prevention and control of deep engineering disasters urgently require precise dynamic parameters of rock masses. However, traditional methods are limited by sample representativeness and static parameter identification (Hansen et al. 2024; Miranda et al. 2011; Hashemi et al. 2010), making it difficult to obtain key energy dissipation parameters (such as Rayleigh damping coefficient) under dynamic loads. Most of the existing identification methods are based on small impact loads in the laboratory (5.5kN) (Huang R. et al.) and their input magnitude is significantly different from that of real ground shakes (such as the 2024 Ibaraki M5.3 earthquake), with questionable parameter scalability (Hashemi et al. 2010; Cai et al. 2011; Yi et al. 2025), and there is a lack of systematic research under real seismic wave input. This study extends the method and utilizes the Ibaraki seismic wave data-driven finite element iterative algorithm to collaboratively identify the elastic modulus and Rayleigh damping coefficient, with a focus on exploring the sensitivity of the input data magnitude to the identification of parameters (especially the damping coefficient).

2 MODEL EXTENSION AND EXPERIMENTAL DESIGN

2.1 Review of parameter identification framework

Based on the method proposed in previous studies, the simultaneously identification of elastic modulus E and Rayleigh damping coefficients α and β is achieved by eigenvalue analysis with dynamic response sensitivity. The algorithm simultaneously updates the elastic modulus and damping coefficient instead of identifying them step by step. This coupling treatment is more in line with physical reality because the change in material stiffness inevitably affects the damping characteristics. First, it is necessary to construct the basic dynamic framework of the mass matrix and stiffness matrix, and then obtain the natural frequency of the system through eigenvalue analysis. Then solve the motion equation using the Newmark-β method. To consider numerical stability, the sensitivity matrix is processed by SVD decomposition in the algorithm design, and finally the convergence condition is met by setting the relative error threshold.

2.2 Input data

A controlled impact test was conducted on the flat surface of Oya tuff using a 5.5 kg hammer (effective radius 0.038 m, duration 0.02 s). Acceleration responses were recorded at five points (0.3, 0.6, 1.2, 2.4, 4.8 m from impact) via 24-mm-diameter accelerometers. The generated Rayleigh waves exhibited peak accelerations up to 10 m/s². Data were filtered with a 500Hz low-pass filter to align with vibration propagation characteristics, limiting the effective frequency range to 0–500 Hz.

2.3 Seismic wave data

Field data from the March 21, 2024, Ibaraki earthquake (M5.3, depth 50 km) were obtained from the Oya Observation Center. Original velocity time histories (90s duration, Δt=0.002s) were converted to acceleration using the central difference method (Eq(1), Fig. 1). The North-South (Y) component showed the highest amplitude (±0.06 m/s²), with distinct P-wave (7s) and S-wave (15s) arrivals. Frequency spectra revealed dominant bands below 20 Hz in all directions (X/Y/Z). For FLAC3D simulations, data were interpolated to Δt=0.0005s to satisfy the CFL stability criterion.

$$a(t) = \frac{v(t+\Delta t) - v(t-\Delta t)}{2\Delta t} \quad (1)$$

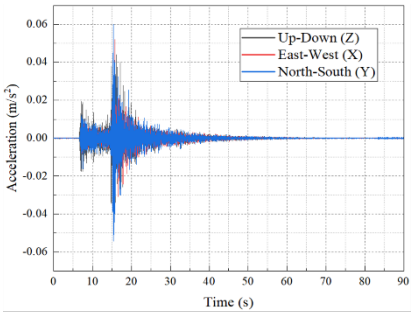


Figure 1 Ground acceleration time history (Ibaraki earthquake).

2.4 Key contrast

The datasets exhibit three orders of magnitude difference in peak acceleration and divergent frequency content (Lab: 0–500 Hz; Seismic: 0–20 Hz), highlighting fundamentally distinct loading regimes (Table 1).

Table 1. Input data characteristics

Parameter	Laboratory Impact	Ibaraki Earthquake
Peak Acceleration	10 m/s²	0.06 m/s² (Y-dir.)
Frequency Range	0–500 Hz	0–20 Hz
Duration	0.02 s	90 s
Source Mechanism	Point impact (Hammer)	Seismic waves (M5.3)

3. RESULTS AND ANALYSIS

Using the coupled inversion framework, laboratory impact data (5.5 kN hammer) yielded an elastic modulus $E_{lab} = 4.30$ GPa, and Rayleigh damping coefficients $\alpha_{lab} = 29.254$, $\beta_{lab} = 3.576 \times 10^{-4}$ Preliminary analysis of seismic wave input (Ibaraki M5.3 earthquake) indicates distinct parameters: $E_{seis} = 3.92$ GPa, $\alpha_{seis} = 21.36$, $\beta_{seis} = 8.94 \times 10^{-4}$.

The seismic-derived β coefficient is 2.5× larger than the laboratory value, while α decreases by 27%. revealing a fundamental divergence driven by input dynamics: High-amplitude/short-duration laboratory impact (>100 Hz) prioritizes mass-proportional damping (α), whereas low-amplitude/prolonged seismic waves (<20 Hz) amplify stiffness-proportional damping (β) through cyclic straining. This frequency-dependent shift underscores how input magnitude dictates "equivalent" damping behavior in linear models, with elevated β under seismic loading potentially indicating latent nonlinearities (e.g., microcracking) at field scales.

Even with linear elastic modeling, the identified parameters represent equivalent damping behavior. The elevated β_{seis} suggests stronger energy dissipation at low frequencies, potentially reflecting unresolved nonlinearities (e.g., joint sliding/microcracking) under field-scale loading. This challenges direct scalability of lab-derived damping parameters to seismic conditions.

4. DISCUSSION

This study establishes that input data magnitude exerts a deterministic influence on identified parameters, particularly Rayleigh damping coefficients. Identical numerical models and inversion algorithms yield significantly divergent results when processing high-magnitude laboratory impacts versus low-magnitude seismic waves. Three interconnected mechanisms explain this phenomenon. First, strain-level dependency emerges as seismic loading induces prolonged cyclic straining at lower amplitudes, activating latent nonlinearities such as microcrack propagation or joint slippage. These mechanisms remain unaccounted for in linear elastic constitutive models. Second, frequency-energy decoupling occurs because hammer impacts primarily excite high-frequency modes, overemphasizing mass-proportional damping α.

Conversely, seismic inputs amplify stiffness-proportional damping β through sustained low-frequency oscillation. Third, inherent limitations of the Rayleigh damping model constrain its ability to characterize vibration attenuation characteristics across different excitation spectra. Consequently, for earthquake engineering applications, hammer-test parameters may underestimate damping. Therefore, employing seismically representative input remains critical for reliable stability evaluations.

5. CONCLUSIONS AND FUTURE WORK

This study extends laboratory-based parameter identification methods to seismic wave input, demonstrating that input data magnitude influences rock dynamic parameter identification, particularly for Rayleigh damping coefficients. These findings highlight the necessity of matching input data characteristics with target engineering scenarios in geotechnical dynamic analysis. This study discusses the sensitivity of input parameters. Future work will investigate parameter identification sensitivity under seismic waves of varying intensities and spectral properties. Implementing seismically representative inputs remains essential for reliable stability assessments in earthquake engineering applications.

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Changes in geotechnical properties of embankments made of crushed solidified soils over time

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ABSTRACT

One method for effectively utilizing the unsuitable soil that is generated at construction sites is to improve it by solidification. One of the solidification improvements is the improvement to crushed solidified soil. Crushed solidified soil is a material created by crushing solidified improved materials after they have undergone a certain period of strength development. This material is compactable and exhibits only a small strength increase after compaction. Previous studies have investigated the geotechnical properties of crushed solidified soil. In this current study, to understand the strength characteristics of crushed solidified soil, a one-year investigation was conducted on compacted lab specimens and embankments constructed in the field using this material to examine the relationship between elapsed time and strength. These results show that embankments made of crushed solidified soil exhibit low strength development, making such soil a suitable material for constructing embankments on soft ground.

Keywords: crushed solidified soil, strength characteristics, solidification

1 INTRODUCTION

In the Ishikari River basin of Hokkaido, Japan, the excavated soil is being used for the construction of river levees. When the excavated soil is soft, it is improved into crushed solidified soil before use (Nakano 2023).

Previous studies on crushed solidified soil prepared in the lab have demonstrated that, while its strength is lower than that of solidified soil, its unconfined compressive strength can increase significantly over time, depending on the solidified mixing amount and standby time (Sato 2024). It is unclear what the strength of embankments made of crushed solidified soil will be. So we investigated the strength of crushed solidified soil embankments was monitored for a one-year period post-construction and the results of tests on embankments were compared to those of tests on lab specimens. This paper summarizes the results.

2 CRUSHED SOLIDIFIED SOIL

Crushed solidified soil is created by mixing the original soil with a stabilizer, allowing it to solidify over a certain period, then crushing it to make it compactable. The terms used in this paper are defined as shown in Figure 1.

- Solidified soil: Soil mixed with a stabilizer
- Crushed solidified soil: Solidified soil that has undergone sufficient solidification and has then been crushed
- Standby time: The period between the mixing of the soil with stabilizer and the crushing of the solidified soil.
- Curing time: For solidified soil and crushed solidified soil, the period from specimen preparation to testing.

3 TESTING METHODS

The original soil used in this study is a silty clay with a high fine fraction, and its cone index is extremely low, at 29.6 kN/m², rendering it unsuitable for supporting construction machinery operation without improvement (Japan Road Association 2009).

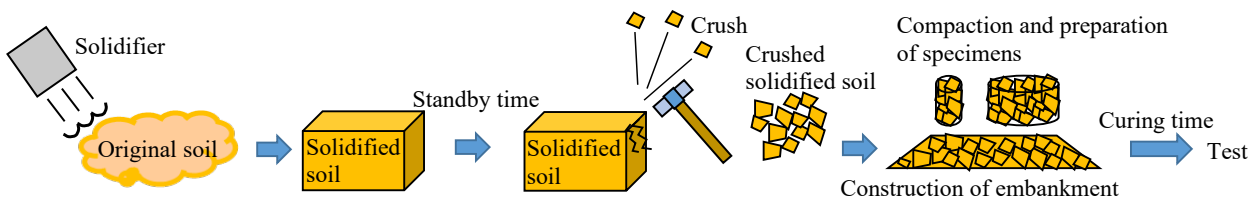


Fig. 1. Definition of the terms.

A cement-based stabilizer was used to produce the crushed solidified soil. The standby time was 7 days. The original soil was mixed with a stabilizer to produce solidified soil. The mixing ratios of the stabilizer were 90 kg/m³, 100 kg/m³, 110 kg/m³, 120 kg/m³, and 140 kg/m³. This mixture of original soil and stabilizer was temporarily stored in the field yard. After a standby time of 7 days, the solidified soil was crushed using a backhoe, producing crushed solidified soil. That soil was then leveled to achieve embankments with a height of 0.3 m and was compacted using a 4-ton-class combined vibratory roller with 2, 4, 6, and 8 times. Approximately one year after construction, block sampling was conducted, and unconfined compression tests were performed.

Lab specimen tests were conducted using two types of soil samples: solidified soil, which was the original soil mixed with a stabilizer at the time of embankment construction, and crushed solidified soil, which was the original soil mixed with the stabilizer, temporarily stored, and then crushed. For each specimen, the cone index was measured immediately after preparation. After a specified period of curing, the unconfined compressive strength was remeasured.

4 TEST RESULTS

Figure 2 shows the cone index both of embankment by crushed solidified soil and specimens of solidified soil and crushed solidified soil from laboratory tests. The target cone index was set as 400 kN/m² to ensure the proper operation of the roller machine. The solidified soil exhibits a low cone index, while the crushed solidified soil successfully meets the 400 kN/m² cone index required for construction. Additionally, the cone index of the embankment's crushed solidified soil increased with increase in the number of compaction times, confirming that crushed solidified soil responds well to compaction.

The relationship between curing time and unconfined compressive strength was investigated for lab-prepared crushed solidified soil specimens with solidified mixing amount of 140 kg/m³ (Figure 3). In both cases, the unconfined compressive strength of the crushed solidified soil was lower than that of solidified soil. From the 150th day of curing, the strength increase of the crushed solidified soil significantly slowed.

The unconfined compressive strength of the embankment one year after construction is shown in Figure 4. Excluding the 2-tim condition, it appears that the unconfined compressive strength generally increases with increases in the number of compaction times. A comparison was made for different solidified mixing amount for each compaction condition.

The strength slightly decreased with increase in solidified mixing amount. The unconfined compressive strengths of the embankment are significantly lower-approximately half-than those of the lab specimens.

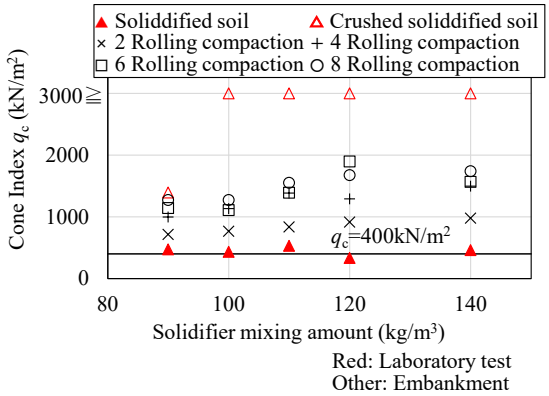


Fig.2 Cone index of immediately after rolling compaction of solidified soil and crushed solidified soil

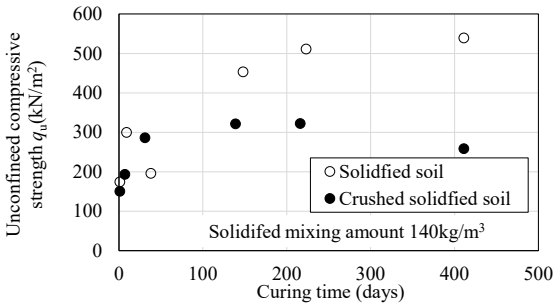


Fig.3 Relationship between curing time and unconfined compressive strength of lab-prepared specimens

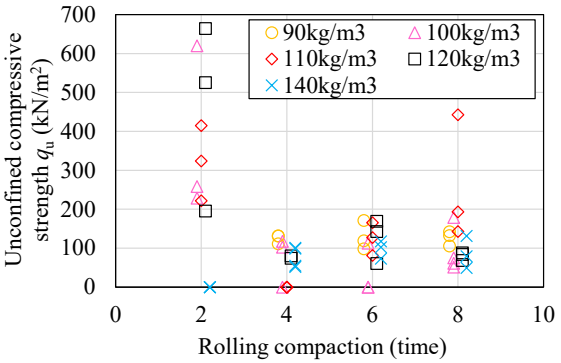


Fig.4 Unconfined compressive strength of test embankment

Higher solidified mixing amount, many soil clumps (approx. 5 mm in diameter) were present, making it difficult to form standard cylindrical specimens (5 cm diameter, 10 cm height). Additionally, the adhesion between the soil clumps was weak.

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Horizontal Prestressing Effects of Stone Columns: Numerical Insights into Installation and Analysis Methods

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ABSTRACT

This study investigates the horizontal prestressing effects induced by stone column installation, with particular focus on displacement-based methods performed without or with minimal predrilling. Using advanced three-dimensional numerical modelling and critical evaluation of existing literature and analytical procedures, this study examines how installation-induced lateral stresses influence the geotechnical performance of soils.
The research focuses on three key aspects of ground improvement: settlement reduction, bearing capacity enhancement, and liquefaction resistance. Analysis highlights the significant differences in stress transfer mechanisms and soil-structure interaction between Low-Confinement Stone Columns (LCSC) and displacement-based installations. Displacement methods such as vibro-displacement were shown to substantially increase horizontal stresses in the surrounding soil, resulting in markedly improved ground performance.
In addition, we review and assess the limitations of current design approaches used for settlement analysis, stability checks, and liquefaction evaluation. The findings underscore the need to incorporate horizontal prestressing effects into geotechnical design practices to avoid underestimating and overestimating performance, and to develop more accurate, installation-informed design strategies for stone column-improved ground.

Keywords: stone columns, horizontal prestressing, ground improvement, numerical modeling, settlement reduction, liquefaction mitigation, low-confinement stone columns (LCSC)

1 INTRODUCTION

Stone column ground improvement, commonly referred to as vibro-replacement or vibro-displacement, is widely used to improve soils by installing columns of compacted crushed stone or gravel using a horizontally or vertically vibrating probe. These columns enhance the composite stiffness of the soil, reduce settlement, and increase liquefaction resistance. While stone column systems such as Geopier®, Ampier®, and other methods may appear similar, their effectiveness depends strongly on the installation method and the resulting interaction with the surrounding soil.

A key mechanism in the performance of stone columns is horizontal prestressing, defined as the increase in lateral stresses within the surrounding soil caused by column expansion during installation. Displacement-based installation techniques, such as vibro-displacement, induce this effect by laterally expanding the column during construction, thereby generating significant horizontal confinement, in particular if the vibrating probe vibrates horizontally. This process enhances load transfer efficiency and promotes a more favourable stress state than for earth pressure at rest (K_0-state), resulting in a stiffer, more resilient, and liquefaction-resistant ground system. In contrast, certain installation methods involving predrilling or washing out of substantial soil volumes (vibro-replacement) and minimal lateral soil displacement result in limited confinement. To distinguish this category of installation from displacement-based techniques, the present study introduces the term Low-Confinement Stone Columns (LCSC), referring to methods that do not actively mobilize lateral stresses during installation and therefore offer reduced improvement performance.

This study aims to explore the geotechnical significance of installation-induced horizontal prestress through a combination of numerical simulations and literature synthesis. It examines how different installation strategies affect soil-structure interaction, and how these effects influence three key performance indicators: settlement behaviour, bearing capacity, and liquefaction response.

3 ANALYSIS RESULTS

Settlement Performance:

A pad foundation (3 × 3 m) under a contact pressure of 240 kN/m² (5000 psf) was analyzed using different

stone column improvement scenarios.

The results confirm that stone columns installed using displacement-based methods, which induce horizontal prestressing through radial expansion during construction, achieve significantly greater settlement reduction compared to Low-Confinement Stone Columns (LCSC). While LCSC installations involve predrilling and minimal soil displacement, resulting in limited lateral confinement, displacement-based techniques generate substantial horizontal stress in the surrounding soil. This leads to improved load transfer efficiency and increased composite ground stiffness, enabling settlements to be reduced to within commonly accepted limits (~2.54 cm or 1 inch).

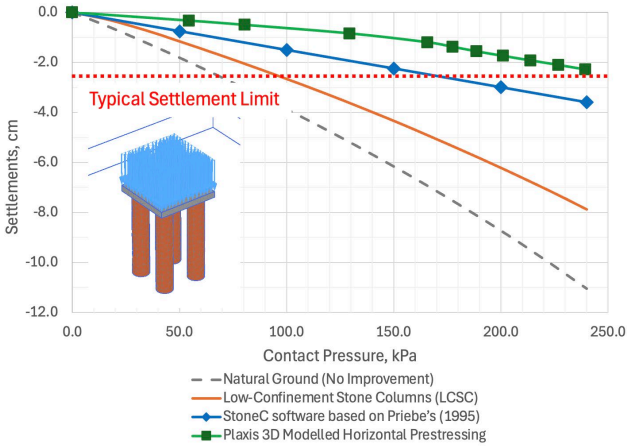


Fig. 1. Foundation settlement curves under different stone column installation scenarios.

Bearing Capacity Enhancement:

Stone columns substantially improved ultimate bearing capacities compared to untreated soils. The analysis emphasized that methods simulating horizontal prestressing provided notably higher capacity improvements, reinforcing the critical role of installation-induced lateral stresses.

Liquefaction Mitigation:

Horizontal prestressing significantly increased the cyclic resistance ratio (CRR), enhancing liquefaction resistance. By explicitly considering the increased lateral stresses in the design, the safety factor against liquefaction (SF) notably improved, quantified as:

$$SF_{liq} = \frac{CRR_0 + \Delta CRR_{h,0}}{CSR \times R_{rd}} \qquad (1)$$

where:

- CRR_0: Initial CRR at original confinement
- $\Delta CRR_{h,0}$: CRR increase due to horizontal prestressing
- CSR: Cyclic Stress Ratio
- R_{rd}: Shear stress reduction factor

These results underscore the necessity of incorporating horizontal confinement into liquefaction analyses and support displacement-based installation techniques as highly beneficial for ground improvement applications.

4 CONCLUSIONS

This study highlights the critical role of horizontal prestressing caused by installation in enhancing the effectiveness of stone column–reinforced ground. Methods focused on displacement that create lateral soil confinement were shown to significantly exceed drill-and-fill approaches in minimizing settlement, boosting bearing capacity, and improving liquefaction resistance.

The findings underscore the importance of incorporating horizontal prestress effects into geotechnical design. Future work should focus on validating these insights through field instrumentation and expanding the analysis to a broader range of soil types, including cohesive layers.

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Unlocking the Potential of Construction Waste Reuse in Slovenia: Challenges and Solutions

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ABSTRACT

Construction activities in Slovenia produce substantial volumes of waste, predominantly excavated soil and demolition debris. Despite the recognized environmental and economic advantages of reuse, implementation remains limited due to legal, organizational, logistical, and technical challenges. Excavated materials, though often uncontaminated, are classified as waste and subject to strict environmental permitting procedures. The lack of clear guidelines and the complexity of compliance hinder reuse efforts.

This paper investigates the feasibility of using various types of construction waste—such as soil, sediments, and industrial by-products—in pavement structures. It presents a methodological framework that integrates material classification, mechanical characterization, and parametric optimization for structural design. Based on laboratory experiments conducted within a bilateral research project between the University of Maribor (Slovenia) and Afyon Kocatepe University (Turkey), the study explores the mechanical properties of materials such as waste rubber, lignin, glass powder, and nylon fibers. These materials were incorporated into subgrade, base, and asphalt layers.

The results demonstrate that optimized pavement designs incorporating secondary materials can reduce both construction costs and CO_2 emissions. The use of geosynthetics was shown to be economically and environmentally beneficial when the subgrade CBR is below 5–6%. A systematic reuse approach can significantly improve resource efficiency in road construction. However, broader adoption requires regulatory clarity, standardized testing protocols, and effective coordination among stakeholders. This study highlights the need for integrated policies and technical frameworks to unlock the potential of construction waste in achieving sustainable infrastructure development.

Keywords: construction waste, excavated soil, sustainable construction, reuse of materials, implementation challenges

1 INTRODUCTION

Slovenia, with 2.2 million residents and an area of 20,271 km² (50% forested), faces growing demand for construction materials—particularly for road infrastructure. Each year, millions of cubic meters of soil are excavated, much of which is treated as waste and landfilled.

International agencies (e.g., UN, WHO) have emphasized sustainable soil management, advocating reuse as a solution to reduce environmental degradation. Numerous studies (e.g., Huang et al., 2007; Bamigboye et al., 2021) confirm that secondary materials can effectively replace virgin aggregates. However, the mechanical performance of these materials depends on composition, with excessive plastic or organic content reducing load-bearing capacity (Abukhettala & Fall, 2021). Some additives, like fly ash, have proven beneficial for improving subgrade performance (Khan et al., 2022). This research evaluates the integration of such materials into pavement structures using parametric analysis and optimization models.

2 KEY ASPECTS OF REUSE IN CONSTRUCTION.

2.1 Regulatory Framework

In Slovenia, excavated soil is considered waste under national law, regardless of contamination. Reuse is permitted only if environmental standards are met and the soil is used on property owned by the generator. EU legislation allows temporary storage (up to one year) and recycling within three years. However, the approval process is lengthy and administratively complex, discouraging investors.

Liability is a major deterrent; while landfill disposal shifts responsibility to waste management operators, reuse places long-term environmental accountability on the project owner.

2.2 Technical Considerations

Technical limitations stem from the lack of standardized testing methods to assess geotechnical and

geochemical suitability. As a result, designers often exclude reuse in early project planning. Most projects default to primary materials with known properties, minimizing perceived risks.

2.3 Economic and Logistical Barriers

Reuse is often costlier due to transportation, testing, and temporary storage. Space constraints on construction sites and low landfill fees further diminish financial incentives. Without economic support mechanisms, the use of secondary materials remains marginal.

2.4 Environmental Implications

Reuse can significantly reduce the extraction of natural resources and associated emissions. However, contamination risks must be rigorously evaluated. Standardized risk assessments are essential to ensure that reused materials do not pose long-term threats to soil, groundwater, or human health.

3 EXPERIMENTAL APPLICATIONS AND CASE STUDIES

Several European and bilateral research projects have explored the feasibility of integrating waste materials into construction:

- DRA-MUR-CI focused on sediment reuse for flood control in the Drava-Mura basin.
- ELGIP addressed urban soil reuse in member countries, identifying legal and logistical gaps.
- CINDERELA and URGE developed circular economy models for urban construction.
- GEOLAB and GEOTECHNOLOGY focused on geotechnical innovations using recycled materials.
- A bilateral project (2023–2025) between Slovenia and Turkey tested the use of industrial waste (rubber, glass, lignin, nylon) in road construction.

In the latter, laboratory tests and large-scale triaxial apparatus were used to evaluate load-bearing capacity. Glass powder and nylon bags were also incorporated into asphalt binders. The results informed optimization models to minimize cost and CO_2 output while ensuring long-term structural performance.

4 PAVEMENT STRUCTURE OPTIMIZATION: MODEL AND RESULTS

A parametric analysis was conducted using an optimization model (Fig. 1) with the following variables:

- Traffic load (ESALs): 10^4 to 10^7
- Subgrade CBR: 3% to 7%
- Asphalt stability: 2–10 kN
- CBR of base and subbase layers: 30%, 60%, 100%

The model was governed by structural, economic, and environmental constraints, including minimum layer thickness, frost resistance, and Marshall stability. Cost and emissions were calculated using unit material prices and CO_2 indices.

Results showed:

- Geosynthetics significantly reduced cost and emissions when subgrade CBR < 5–6%.
- Waste rubber and lignin performed well in lower layers.
- Glass powder enhanced the stiffness of the asphalt binder.
- Optimization reduced layer thickness while meeting performance standards over 20 years.

5. CONCLUSIONS

This study demonstrates the technical feasibility and environmental benefits of using construction and industrial waste in pavement structures.

Key findings include:

- Waste rubber, lignin, and glass powder are viable for use in subgrade and base layers.
- Glass powder and nylon can enhance bituminous layers.
- Optimized designs incorporating geosynthetics and waste materials outperform traditional structures in cost-effectiveness and sustainability.

Future research should focus on in-situ pilot projects and developing national guidelines to support broader implementation.

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AsRTC10 «Urban Geo-informatics»

Boring Data Base to Characterize Hidden Active Fault for Urban Safety in Osaka and Almaty

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ABSTRACT

Osaka and Almaty are found to have some common geological conditions. These common conditions are 1 deep sediment geology 2 dip slip fault type and 3 hidden active faults under the surface. The base rock distributes at 600 to 1,500 meters in depth in Osaka and at 1,000 to 4,000 meters in Almaty. Both areas have been continuously sinking since the recent young geological period in a few million years. Faults in both areas may be recognizable by surface topology. A hidden active fault has been found in Osaka by a seismic reflection survey. Geotechnical boring data in Osaka was found very useful to identify the detailed characteristics of the fault deformation for preparing structural design against possible displacements of structures crossing the fault.

Keywords: hidden active fault, fault displacement, disaster design for underground structure, Osaka, Astana

1 INTRODUCTION

Active faults are found in both areas in Osaka and Almaty. Recent research based upon reflection seismic surveys in Osaka have revealed clear evidence of the existence of active faults which show no surface fault deformation. Topological features of active fault are discussed in Almaty. In addition to the above discussion, a boring data system is utilized to understand the detailed characteristics of the hidden faults in Osaka.

2 ACTIVE FAULTS IN OSAKA

Recent studies have revealed the active fault in Osaka as shown in Figure-1. These faults have been studied by geological, topological, geophysical, and geotechnical methods.

2.1 Uemachi-fault

Uemachi-fault is located at the center of Osaka-city from north to south direction shown in Figure 1. Section along west to east of RL2 in Figure 1 is shown in Figure 2. Figure 2 shows the Uemachi fault defined by topology and by geological section by boring data. The topological feature of the fault by a continuous step feature is some 500 m east of the real fault as shown by boring section. The original position of the edge of folding by the Uemachi fault moved to eastwards by weathering of the ground surface.

3 HIDDEN ACTIVE FAULTS IN OSAKA

After the Kobe Earthquake in 1995, a series of geophysical surveys were carried out with borings in Osaka.

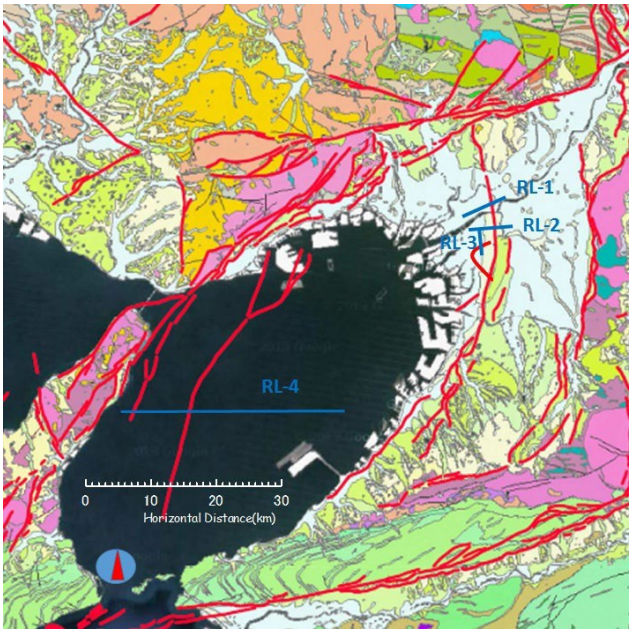


Figure 1 Active Faults in and around Osaka

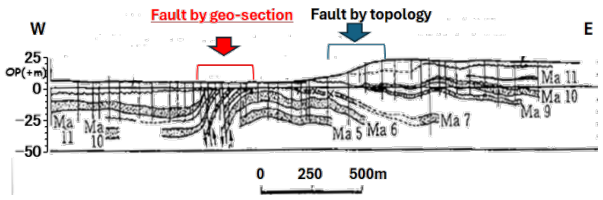


Figure 2 E-W section crossing Uemachi-fault

One of the seismic reflection surveys was conducted as shown in Figure 3. A folding structure by a reverse dip slip fault was found and named as "Sakuragawa-Fault." The result shows in Figure 5 and clear dip slip fault at a depth around 1,200m and a folding deformation at the shallower surface. The faulting structure near the surface was further studied by available geotechnical borings and shown in Figure 4. The uppermost layer named Ma13 is an alluvial marine clay layer which is the recent deposit after about 19 thousand years ago. The lower diluvium marine clay layer of Ma12, Ma11, and Ma10 are found folding deformation due to fault movements.

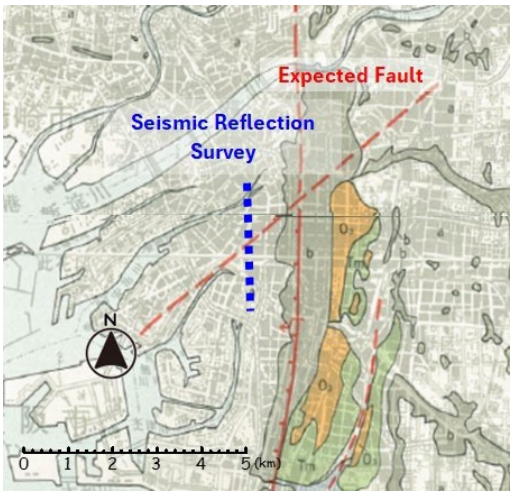


Figure 3 Faults in Osaka (Fujita, 1987)

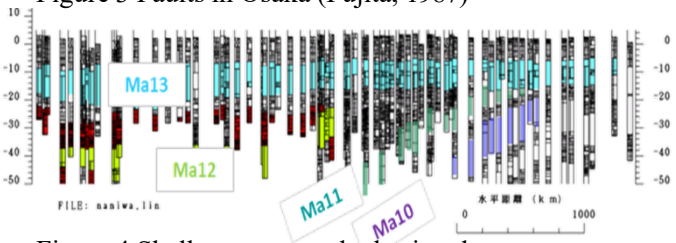


Figure 4 Shallow structure by boring data

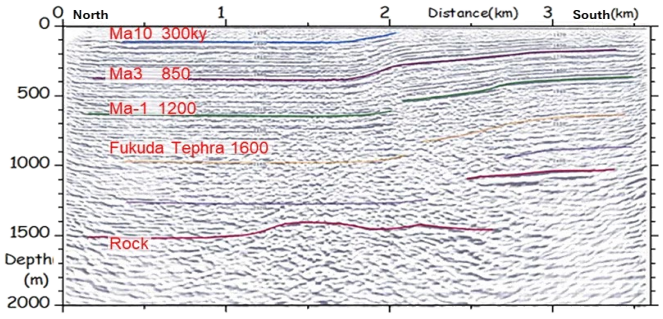


Figure-5 Folding Structure by Reflection Survey

The results of boringdata in Osaka were studied and the boring points of the bending part were colored red with down and up sides in black and blue in Figure-6. Geotechnical borings are very useful to understand the hidden faults and used for designing subway structures.

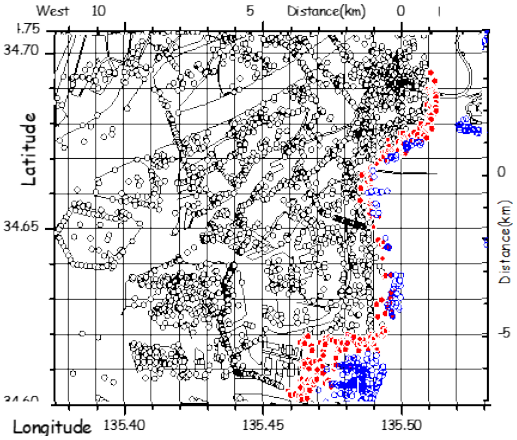


Figure 6 Hidden fault in Osaka

3 HIDDEN ACTIVE FAULTS IN ALMATY

A topological study of faults in Almaty was shown in Figure 7. The result indicates the estimated hidden fault is found very near to the subway line in Almay.

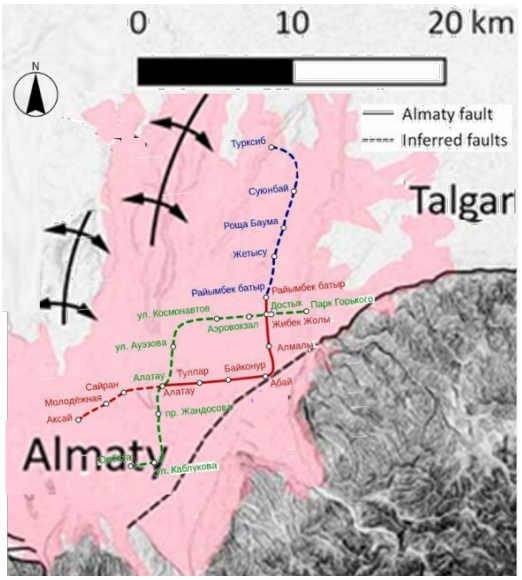


Figure 7 Hidden fault in Almaty (Grützner, 2017)

4 CONCLUSIONS

Hidden fault in Osaka was identified by a reflection survey below flat ground surface condition. The detailed active hidden fault was further identified by boring databases in Osaka. In Almaty, geomorphological survey suggests the hidden fault near the metro line in Almaty. Boring database of Almaty may be one of the best approaches to clarify the hidden active fault in Almaty.

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Insights into liquefaction risk associated with microtopography of Tokyo lowland

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ABSTRACT

This study constructed a three-dimensional (3D) ground model and assessed the liquefaction risk in Katsushika Ward, Tokyo. A model based on 870 borehole logs was developed using spatial interpolation. The liquefaction risk was evaluated by calculating the liquefaction potential index (P_L) under different seismic scenarios. The results indicated high liquefaction susceptibility across the region, particularly along natural levees. However, the P_L values in the Old Sumida rivers former channel were lower, despite historical liquefaction. This discrepancy can be attributed to thin liquefiable soil layers. These findings highlight the need for geological correction in P_L-based assessments to improve the accuracy of complex terrain.

Keywords: liquefaction, microtopography, former riverbed, voxel model, liquefaction potential index

1 INTRODUCTION

In this study, a voxel model with a 100 m × 100 m grid was constructed using high-density borehole log data from the Tokyo lowlands, and a simplified liquefaction assessment was conducted. The study area includes regions where liquefaction was confirmed during the 1923 Great Kanto Earthquake and is characterized by the microtopographic features of coastal plains, such as former river channels, natural levees, and deltas. Therefore, the liquefaction potential index was calculated based on the liquefaction assessment results, and the distribution characteristics of the liquefaction potential index in relation to different microtopographies were analyzed.

2 CONSTRUCTION OF THE GROUND MODEL FOR THE STUDY AREA

2.1 Overview of the topography of the study area

Katsushika Ward, Tokyo, was selected as the study area. The microtopographic classification of this area is shown in Figure 1. This region is categorized as part of the Tokyo Lowland, with the Old Sumida River located in the northern part, the Nakagawa River extending from east to south, and the Arakawa River flowing along the western boundary. Natural levees developed along these riverbanks, forming a slightly elevated terrain, whereas the central part of the area consists of coastal plains and deltaic formations. The Old Sumida River represents an ancient river channel.

2.2 Ground modeling using geotechnical information

To construct the ground model, a total of 1,084 borehole logs were compiled, consisting of 424 public borehole data records and 660 private borehole data records. Boreholes with drilling depths less than 30 m were excluded, resulting in 870 borehole logs. The borehole locations are shown in Figure 1. Private borehole data were obtained from subsurface investigations conducted on individual building construction projects, which resulted in a relatively uniform distribution within the study area.

The ground model was developed as a three-dimensional (3D) grid model by Kimura et al. (2013) using the following procedures: (a) compilation and preprocessing of borehole data for ground model construction, (b) determination of the basement depth of the alluvial layer from borehole data, (c) generation of a geological boundary surface model, (d) construction and integration of grid models for both the alluvial layer and underlying foundation, and (e) quality assessment and validation of the 3D grid model.

3 APPLICATION OF THE GROUND MODEL FOR LIQUEFACTION RISK ASSESSMENT

3.1 Overview of liquefaction risk assessment

Liquefaction risk assessment was conducted by calculating the liquefaction safety factor (F_L) based on the specifications for highway bridges (2017). The liquefaction potential index (P_L) was derived using the

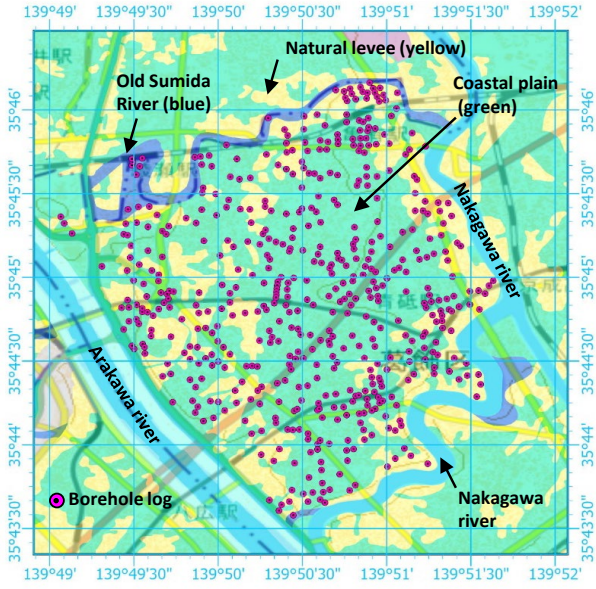


Fig. 1. Microtopography of the study area

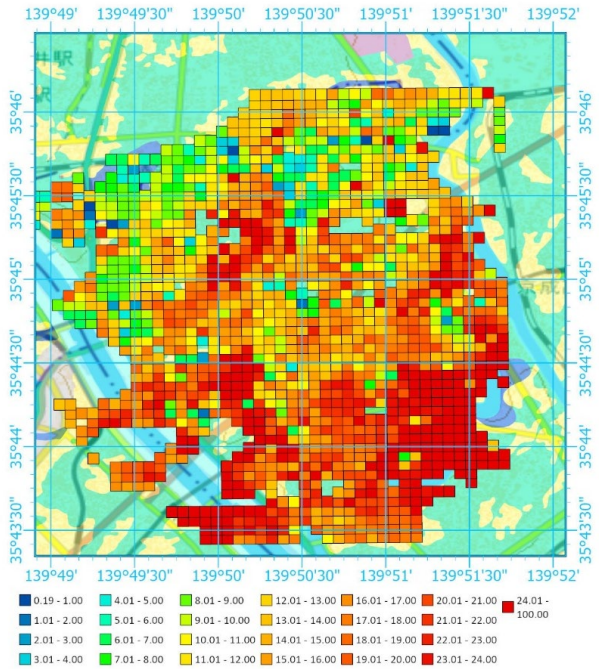


Fig. 2. Liquefaction potential index distribution

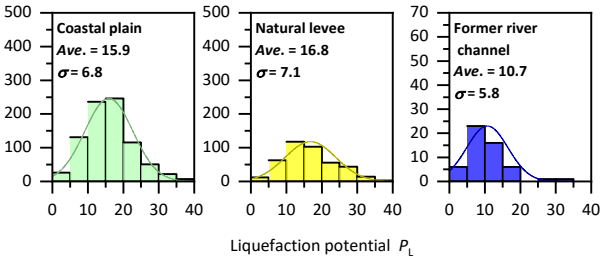


Fig. 3. Frequency distribution of P_L values

F_L distribution proposed by Iwasaki et al. (1978). The fine content was estimated using the proposed by Kamei et al. (2002) for the Tokyo lowland. The seismic loading was considered in terms of the maximum surface acceleration, assuming four levels of acceleration: 0.18, 0.30, 0.40, and 0.60 m/s^2.

3.2 Relationship between liquefaction risk assessment and microtopography

Figure 2 shows the spatial distribution of the P_L values under a maximum surface acceleration of 0.30 m/s^2. Figure 3 shows the frequency distribution of P_L values for different microtopographic classifications within the study area. The results showed that the mean P_L values in the delta plains and natural levees were approximately 16–17 with a standard deviation of 7, indicating similar distribution characteristics. In contrast, the mean P_L value in the former channel of the Old Sumida River was 11, which was lower than those of the other microtopographies.

Therefore, despite the documented history of liquefaction in the former channel of the Old Sumida River during the Great Kanto Earthquake, no significant increase in P_L values relative to the surrounding areas was observed. Specifically, the top 3 m consists of fill deposits, whereas the deeper layers contain a high proportion of fine-grained soil. Consequently, the liquefiable soil layers were relatively thin in this area, leading to lower P_L values compared with the surrounding regions. These findings suggest that liquefaction risk assessments based on P_L values may not adequately capture the potential for liquefaction in geologically distinctive microtopographies, such as former river channels.

4 CONCLUSIONS

In this study, we successfully constructed a detailed ground model and conducted a liquefaction risk assessment. Although the P_L-based assessment effectively identifies the liquefaction risk in natural levees and deltaic plains, it may underestimate the risk in former river channels with distinct geological characteristics. Therefore, incorporating geological corrections into P_L-based assessments is necessary to improve the accuracy.

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Seismic Response Study in Krishna-Godavari Offshore Region of India

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ABSTRACT

A detailed seismic study was carried out at the proposed oil and gas production site in the Krishna-Godavari offshore region, located on the eastern coast of India. Initially, a seismicity and seismo-tectonic map for a 300 km radius from the center of the site was developed using the GIS tool ArcMap 10.1. Subsequently, a probabilistic seismic hazard analysis (PSHA) was performed using the computer program R-Crisis. Further, one-dimensional ground response analysis was conducted using equivalent linear method in SHAKE2000. The soil profile at the site is composed of very soft to hard clay, followed by medium dense silty sand and weathered shale or sandstone, extending up to a depth of 300 m. The shear wave velocity profile of the soils was derived from static cone penetration resistance tests conducted at the site. The analysis revealed that under extreme level earthquake (ELE) conditions, significant amplification of ground motion occurred at the proposed site.

Keywords: seismic response study, seismic hazard analysis, site effects, Krishna-Godavari offshore region of India

1 INTRODUCTION

It was intended to explore a block for the oil and gas production in the Krishna Godavari offshore region of the east coast of India. The area is approximately 800 km southwest of the Ganges River Delta, and more than 1100 km south of the Himalayan front which are characterized by low rates of seismic activity. However, during the last decade (1993-2003), three major earthquakes (M 6.0 ~ 7.7) occurred in peninsular India. Therefore, a detailed site specific seismic study was carried out at the site to arrive ground motion parameters required for the design of critical oil and gas production facilities.

2 SEISMOCITY & SEISMOTEXTONIC MAPS

A total of 105 earthquake events were recorded within a 300 km aerial radius from the proposed project site, located in the vicinity of the Krishna-Godavari offshore basin. Among the seismic events, approximately 54 earthquakes had magnitudes in the range of 3.0–4.0, 37 events were in the range of 4.0–5.0, and 18 events were in the range of 5.0–6.0. The number of low-magnitude earthquake was found to be considerably higher than that of moderate to high-magnitude earthquakes, indicating a region of frequent but relatively low-intensity seismic activity.

The seismotectonic map, a critical component of the regional seismic hazard assessment, was developed using the Geographic Information System (GIS) platform ArcMap 10.1, which enabled accurate spatial representation and analysis, and is presented in Fig. 1. The earthquake details were mapped onto the faults based on the location and depth of the earthquake and the length of the fault, thus the faults capable of producing strong motion were identified.

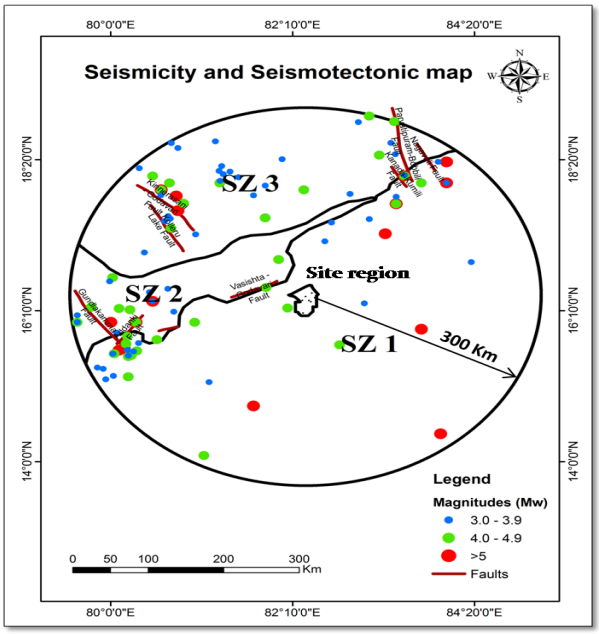


Fig. 1. Seismicity and Seismotectonics of the study region

3 PROBABILITIC SEISMIC HAZARD ANALYSIS (PSHA)

PSHA represents a scientific framework within which uncertainties related to the size, location, and recurrence rate of earthquakes can be considered to develop a holistic picture of the hazard. The purpose of PSHA is to assess the likelihood or the probability of exceedance of the level of earthquake induced vibratory ground motions at a site in a specified time period. In this study the probabilistic computations for the hazard analysis were carried out using R-CRISIS version 18.4.2 software. From the PSHA analysis Peak Ground acceleration (PGA) values for different periods were obtained. The horizontal PGA contour map for an Extreme Level Earthquake (ELE) corresponding to a return period of 200 years (25% probability of exceedance in 50 years) was developed. It was found that PGA at rock level varies from 0.023 g to 0.3 g.

4 SITE RESPONSE ANALYSIS

Local site effects profoundly influence the amplitude, frequency content, and duration of strong ground motion. In the present study one dimensional ground response analysis was carried out by equivalent linear approach (Kramer 1996) using the software SHAKE2000.

4.1 Design earthquake

The site-specific ground response analysis was carried out for the design magnitude of 5.7 M_w with PGA of 0.03g. Two recorded acceleration time history of earthquake of similar magnitude (Imperial Valley M_w 5.5 and Whittier Narrows M_w 5.9) were chosen and scaled accordingly, ensuring that they closely reflect potential ground motions at the site.

4.2 Input soil properties

The soil profile shows that the site is composed of clays ranging from very soft to hard clay extending up to a depth of 148 m below the seafloor and medium dense silty sand from 148 m to 180 m. Weathered shale or sandstone is present at the depths of 180 m – 300 m. The G_{max} profile was calculated based on the correlation with total cone resistance (q_t) proposed by Mayne and Rix (1995) for clay and Robertson (2009) for sand layers. The input soil profile used in the site response analysis is presented in Fig. 2, providing a detailed understanding of the subsurface conditions.

4.3 Surface Horizontal PGA

The mean surface time history for the Imperial Valley and Whittier Narrows Earthquakes are calculated and presented in Fig.3. The horizontal surface acceleration time history corresponding to ELE, presented in Fig.3 shows that the Peak Horizontal Acceleration (PHA) is amplified to 0.055 g from the input motion PHA of 0.03 g.

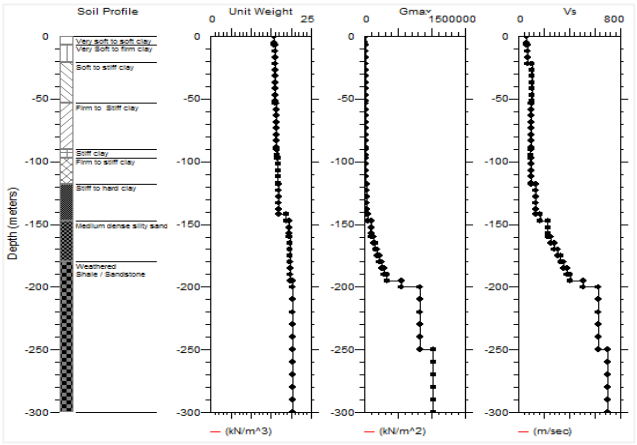


Fig. 2. Soil profile and dynamic soil properties

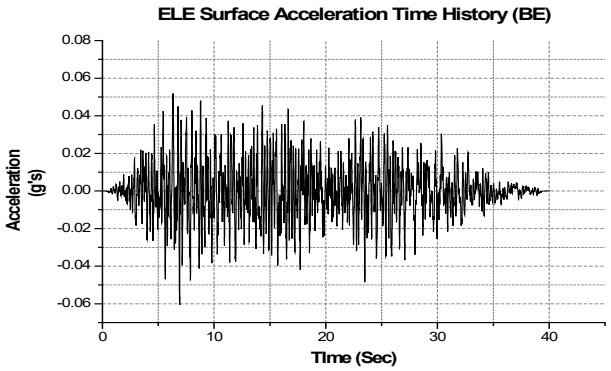


Fig. 3. Acceleration time history obtained at the surface

5 CONCLUSIONS

Seismic study carried out at the proposed oil and gas production site in the Krishna-Godavari offshore region of India shows the occurrence of significant amplification of ground motion due to the presence of thick soil deposits present at the project site. These subsurface conditions notably affect seismic wave propagation, underscoring the necessity of incorporating local site effects into the seismic design of structures in this area.

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Development of GIS-based mapping for ground shaking susceptibility using microtremor in densely populated areas of Sukumo city, Japan

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ABSTRACT

Microtremor observations play a vital role in assessing localized seismic risks in Sukumo City, Kochi Prefecture—an area highly susceptible to strong ground shaking and tsunami hazards due to its proximity to the Nankai Trough earthquake source region. This study conducted high-resolution microtremor surveys using dense mesh grids (125 × 125 m and 62.5 × 62.5 m) and analyzed H/V spectral ratios through GIS-based interpolation techniques. Validation was performed using actual damage data from the 2024 Mj 6.6 Bungo Channel Earthquake. The results indicate that finer mesh resolutions significantly enhance the accuracy of estimating ground shaking susceptibility. When Ground Shaking Risk maps were overlaid with damage records, a strong spatial correlation emerged—particularly in Risk Level 4 zones, which accounted for 64% to 71% of all recorded damage. These included house/building damage, fallen objects, piping system failures, and external facility impacts. Set intersection analysis also revealed recurring multi-component damage patterns, most notably involving the overlap of structural and utility-related failures. Piping systems were consistently present in all three- and four-component damage clusters, emphasizing the need to treat infrastructure as an interconnected system. Overall, this study demonstrates the effectiveness of integrating microtremor observations with high-resolution GIS-based risk mapping. The proposed approach provides a practical framework for disaster preparedness, seismic risk evaluation, and urban resilience planning in earthquake-prone regions.

Keywords: GIS, ground shaking susceptibility, microtremor, mesh mapping, densely populated areas.

1 INTRODUCTION

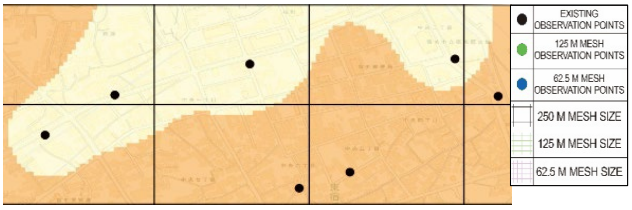
Microtremor observations are critical for assessing seismic risks through the identification of site amplification and localized ground shaking characteristics[1)2)]. Sukumo City, located in Kochi Prefecture, Japan, faces significant seismic hazards due to its proximity to the Nankai Trough, where megathrust earthquakes can trigger tsunamis reaching heights of up to 25 meters. The recent M6.6 Bungo Channel Earthquake (April 2024)[2)] highlighted the need for higher-resolution seismic assessments to better address localized vulnerabilities, especially in densely populated urban areas[2)]. This study enhances seismic risk assessment by applying high-resolution microtremor observations integrated with Geographic Information System (GIS) methodologies, specifically targeting central Sukumo City. Improved spatial resolution in microtremor surveys enables accurate identification of ground shaking susceptibility, significantly supporting disaster preparedness and mitigation strategies in regions prone to severe seismic impacts.

2 METHODOLOGY

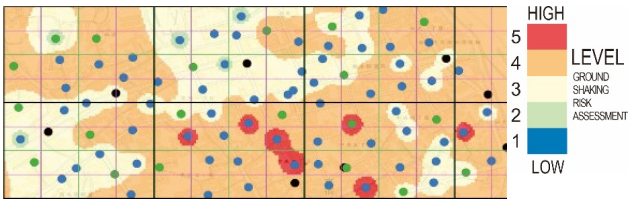
Microtremor surveys were conducted using high-density mesh grids of 125 × 125 m and 62.5 × 62.5 m, supplemented by a general 250 × 250 m grid (JIS X 0410/AMENDMENT1:2002). At each location, measurements were taken using a JU210 seismometer, recording three ground acceleration components (NS, EW, UD) continuously for about 11 minutes. Horizontal-to-vertical spectral ratios (H/V) were calculated from five low-noise intervals, each lasting 40.96 seconds, to determine dominant periods and peak amplitudes. These ratios were used to infer subsurface conditions and classify ground risk into five levels based on road bridge design standards. A Ground Shaking Risk map was generated using QGIS with Inverse Distance Weighting (IDW) interpolation. Validation was performed by comparing the risk map with existing seismic assessments and damage reports from the recent earthquake, using spatial overlay and intersection analysis to confirm consistency between predicted risk and observed damage.

3 RESULTS AND DISCUSSION

Ground shaking risk assessments become significantly more accurate with higher-resolution mesh mapping. Results from microtremor surveys demonstrated that smaller mesh sizes (125 m and 62.5 m) provided a more precise dominant period compared to previous assessments using larger mesh grids (250 m) (Figure 1). Specifically, (Figure 1b) high-resolution mapping clearly identified areas of ground shaking risk which is shown by red color (level 5).



(a) Previous Studies (250 m)



(b) Current Studies

Fig. 1. Comparison of Ground Shaking Risk Map

Validation through overlaying Ground Shaking Risk map with actual residential damage data from the magnitude 6.6 Bungo Channel Earthquake indicated strong correlations between predicted risk zones and observed damage (Table 1). Damage caused by the earthquake was concentrated in areas classified as high-risk zones (risk level 4), with damage rates for most components ranging from 64.29% to 71.05%.

Table 1. Percentage Distribution of Earthquake Risk Levels by Damage.

Risk Level	House/Building Damage (A)	Dropped & fell (B)	Piping Systems(C)	Parking Area Ext. Building (D)
Level 1	0.00%	0.00%	0.00%	0.00%
Level 2	0.00%	0.00%	0.00%	0.00%
Level 3	32.14%	27.42%	23.68%	32.14%
Level 4	64.29%	66.13%	71.05%	67.86%
Level 5	3.57%	6.45%	5.26%	0.00%

Furthermore, Based on Table 2, which is derived from the Set Intersection Matrix (A, B, C, D), the total number of reported cases for each component were 112 for house /building damage (A), 62 for dropped and fallen objects (B), 38 for piping systems (C), and 28 for parking and external buildings (D). The deepest overlap, (A ∩ B ∩ C ∩ D), includes 4 cases, all of which occurred at risk level 4. This indicates that risk level 4 has the potential to cause damage across all four components simultaneously. Notably, piping systems (C) show high vulnerability, with frequent overlaps: 36 cases with house/building damage (A) and 14 with external structures (D). House/building damage (A) appears as the most interconnected component, overlapping with (B), (C), and (D) in 22, 36, and 23 cases, respectively. These patterns indicate that damage often occurs across multiple components simultaneously, highlighting the need for integrated mitigation rather than isolated measures. This is further supported by the 13 cases where (A), (B), and (D) co-occur, pointing to potential cascading failures.

Table 2. Set Intersection Matrix (A, B, C, D)

∩	A	B	C	D	A∩B	A∩C	A∩D	B∩C	B∩D	C∩D	A∩B∩C
A	-	22	36	23	-	-	-	9	7	13	-
B	22	-	9	7	-	9	7	-	-	4	-
C	36	9	-	14	9	-	13	-	4	-	-
D	23	7	14	-	7	13	-	4	-	-	4

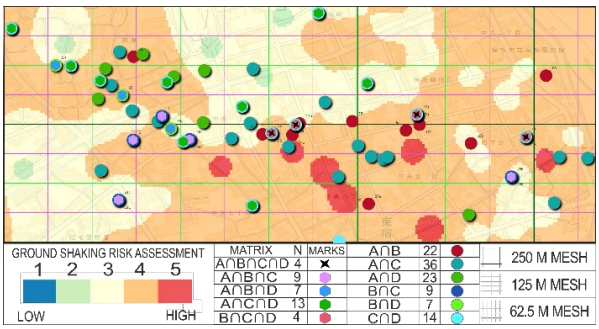


Fig. 2. Comparison of Ground Shaking Risk Map with Set Intersection Matrix of Damage Components

4 CONCLUSIONS

High-resolution microtremor observations combined with GIS-based H/V spectral analysis proved effective in identifying localized seismic risks in Sukumo City. Validation using data from the 2024 Bungo Channel Earthquake showed strong correlation between predicted high-risk zones and actual damage, particularly at-risk level 4. The intersection analysis further revealed that structural damage often occurred across multiple components, especially involving house / building damage and piping systems. These results support the use of fine-mesh Ground Shaking Risk mapping as a practical tool for seismic risk assessment and disaster mitigation in urban areas exposed to megathrust earthquakes.

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Glacier Lake Outburst Flood (GLOF) Hazard Assessment and Modelling in parts of Himalaya

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ABSTRACT

Glacial Lake Outburst Floods (GLOFs) occur when the moraine dam of a glacial lake fails, releasing large volumes of water downstream with little or no forewarning. The Indus River Basin in Himalaya is increasingly threatened by Glacial Lake Outburst Floods (GLOFs) due to accelerated glacial melt caused by climate change. This study aims to identify potentially dangerous glacial lakes throughout the basin and to simulate possible GLOF situations to evaluate downstream risk. This research uses hydrodynamic modelling and high-resolution remote sensing data to evaluate the hazard potential of glacier lakes. The evaluation includes 100%, 75%, and 50% of moraine failure breach scenarios across diverse meteorological circumstances to replicate maximum releases and its downstream impacts. The study conforms to recognized risk assessment frameworks, offering insights into the dynamics of glacier lakes amid climate change. The findings highlight the imperative for continuous monitoring and aggressive interventions to combat the escalating danger of GLOFs, hence enhancing disaster preparedness and risk mitigation methods for downstream communities in the Indus Basin. Findings suggests Nubra sub-basins are under considerable GLOF risk. GLOFs simulations indicate that projected peak discharges may vary from 400 m^3/s to exceeding 500 m^3/s in extreme circumstances, with flood waves potentially arriving in downstream locales such as Sumur village within hours. A Glacial Lake has been evaluated to potentially release up to 500 m^3/s in the event of total moraine failure and piping failure, affecting regions like Sumur village and nearby area within roughly hours. The potential outflow from this lake leads to serious risks to people and infrastructure, emphasizing the need for early warning systems and disaster management in this area.

Keywords: Himalaya, GLOF, glacier retreat, hydrodynamic modelling, hazard assessment

1 INTRODUCTION

Glacial Lake Outburst Floods (GLOFs) have emerged as one of the most pressing environmental hazards in the Himalayas, a region acutely sensitive to climate change due to its geological youth and dynamic cryosphere. Over the past several decades, Himalayan glaciers have been retreating at unprecedented rates, with regional warming estimated at 0.15 to 0.60°C per decade—significantly outpacing the global average (Veh et al., 2020; Ahmed, R. et al., 2021). This rapid warming has driven the formation and expansion of over 5,000 glacial lakes across the Himalayas, many of which are dammed by loose, unconsolidated moraines. These natural dams are inherently unstable, and their failure can unleash sudden, powerful floods that threaten lives, infrastructure, and livelihoods far downstream (Veh et al., 2019).

The risk posed by GLOFs is not evenly distributed across the Himalayas. The Eastern Himalayas have been identified as a hotspot, with hazard levels estimated to be several times higher than in other parts of the range (Ahmed, R. et al., 2021). The Indus watershed, including its northern tributaries such as the Nubra, Hunza, and Shyok rivers, is especially vulnerable due to the rapid proliferation of glacial lakes held back by fragile moraines (Ives et al., 2010). When these dams fail, the resulting floods can rival or exceed the peak discharges of monsoonal rivers, causing catastrophic destruction to agricultural land, roads, bridges, hydropower facilities, and entire communities (Carrivick & Tweed, 2016; Veh et al., 2019).

Despite growing recognition of GLOF hazards, comprehensive, site-specific assessments remain limited, particularly at the sub-basin or watershed scale (Ahmed, R. et al., 2021). This study addresses this gap by methodically locating a potentially dangerous glacial lakes within the Nubra sub-basin and modeling flood events to assess water flow and socio-economic hazards downstream. With advances in remote sensing, geospatial analysis, and hydrodynamic modeling now offer powerful tools for mapping glacial lake evolution, simulating potential outburst scenarios, and predicting inundation extents with greater accuracy (Ahmed, R. et al., 2021).

Given the increasing exposure of populations and infrastructure to GLOF hazards, there is an urgent need for localized studies that identify potentially dangerous lakes and assess downstream socio-economic risks. Such research is critical for informing early warning systems, disaster-resilient infrastructure planning, and targeted risk-reduction strategies to safeguard vulnerable Himalayan communities (Veh et al., 2020).

2 METHODOLOGY

This study integrates remote sensing, geospatial analysis, and hydrodynamic modeling to assess GLOF risks in the Nubra sub-basin. High-resolution Landsat 8 imagery and COP30 DEM data was used to map glacial lake area. the depth and volume of lake is important for modelling the GLOF assessment. Due the unavailability of bathymetry data we rely on empirical equation by (Huggel et al., 2002) for depth and volume of lake.

The dam breach Hydrodynamic model using HEC-RAS 2D used for 3 scenarios considering (100%, 75%, 50%) for hazard assessment in the downstream area. The breach parameter was estimated by Froehlich, 1995 such as time of failure(t_f), average bottom elevation (B_{ave}) and bottom breach width (B_{bw}).

The output of the HEC-RAS dam breach model outputs is flood depth, flood velocity and breach hydrograph where the breach is validated with verify with Froelich equation of peak discharge (Q_p).

4 RESULTS & DISCUSSION

The model results indicate a 100% water release from the lake, resulting in a peak discharge of 871.1 m^3/s for overtopping failure and 782.57 m^3/s of piping failure. This generates a peak flood depth of 12.5 meters and a maximum flood velocity of 16.1 m/s.flood wave is projected to reach the nearby settlement within 45 minutes with a overtopping failure and 52 min with a piping failure. The total inundated area with over topping failure is 42.52 km^2 and 40.76 $km^2$for a piping failure.

Upon arrival, it is expected to impact critical infrastructure, including the monastery and the hydropower station. Additionally, large areas are likely to experience significant inundation, as depicted in
In the case of a 75% water discharge from the lake, the model forecasts a maximum outflow of 697.28 m^3/s in an overtopping failure situation and 600.85 m^3/s during a piping failure scenario. Such incidents would lead to a total flooded area of roughly 32 km^2.

The highest anticipated flood depth is about 11.78 meters, with a peak flow speed of 15.15 m/s. It is estimated that the flood wave will reach the area near the settlement in approximately 53 minutes for overtopping and around 57 minutes for a piping failure. Key infrastructure and nearby regions are expected to endure significant flooding, to the effects.

Under a 50% water release scenario from the glacial lake, the model simulations indicate a peak outflow of 697.28 m^3/s in the case of an overtopping failure, and 600.85 m^3/s for a piping failure. The resulting flood is projected to inundate an area of approximately 19.52

$km^{2.}$ The flood wave is expected to reach the hydropower station in about 1 hour and 6 minutes, and the nearby settlement in approximately 1 hour and 13 minutes

The maximum flood depth is estimated at 10.82 meters, with a peak flow velocity of 12.45 m/s.

5 CONCLUSIONS

This study demonstrates that combining remote sensing and hydrodynamic modeling offers an effective framework for assessing GLOF hazards in the Nubra sub-basin. The results highlight those rapidly expanding lakes near tectonic faults, such as the lake in this study, presents significant risks to downstream communities and infrastructure. Simulated flood scenarios reveal that critical infrastructure could be impacted within hours of a breach, underscoring the urgency for targeted adaptation. The findings emphasize the need for predictive modeling and land-use planning tailored to the region's unique geomorphic and tectonic context. Proactive measures, including early warning systems and engineered interventions, are essential to reduce vulnerability and build resilience against future GLOF events in the Himalayan. Combining hazard assessments with infrastructure mapping can yield essential insights for disaster preparedness and reduce the effects of future GLOF episodes.

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Landslide Dam Hazard Zonation in the Alaknanda Catchment, Uttarakhand, India Using Frequency Ratio Modelling

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1. INTRODUCTION

Landslide dams are obstructions caused by landslides on the river channel that partially or entirely block the channel flow. The impoundment inundates the region upstream and poses a significant threat of flash flood downstream, called landslide lake outburst flood (LLOF) (Ermini & Casagli, 2003). Uttarakhand has a history of landslide dams, and the majority of the flash floods in Uttarakhand are caused by the landslide dam breach rather than the breaching of glacial lakes (Joshi & Siva Subramanian, 2025; Wasson et al., 2013). The Alaknanda catchment in Uttarakhand, India, is particularly susceptible to landslides due to its steep topography, complex geological structure, and high rainfall, which collectively contribute to slope instability (Kothyari et al., 2024).

Various GIS and machine learning methods for landslide zonation have been developed during the last few decades, including analytic hierarchy process (AHP), frequency ratio (FR), support vector machine (SVM), logistic regression (LR), fuzzy logic and many more (Dai et al., 2002; Lee, 2019; Sarkar & Kanungo, 2004; Siddle et al., 1991; Singh et al., 2024; Zhu et al., 2014). The study's objective is to prepare a landslide dam susceptibility map for the Alaknanda catchment, Uttarakhand, India, using the frequency ratio method. By integrating spatial data on past landslide dams with key controlling factors, the research aims to provide valuable insights for hazard mitigation in one of the most landslide-dam-prone river basins in the Uttarakhand Himalayas.

2. METHODOLOGY

To determine the susceptibility of the landslide dam hazard, the landslide dam inventory for the Alaknanda catchment is prepared using the databases of the Geological Survey of India (GSI) and the National Remote Sensing Centre (NRSC), ISRO. The landslide polygons that intersect the river are manually validated using Google Earth Pro. This inventory is used to determine the susceptibility of landslide dams in the catchment.
To assess susceptibility, eight thematic factors influencing landslide dam formation were considered: slope, aspect, elevation, curvature, Topographic Wetness Index (TWI), Stream Power Index (SPI), distance to streams, and distance to faults. All layers were prepared using a 30-meter resolution SRTM DEM, and the geological information was gathered from the Geological Survey of India's Bhukosh portal. Landslide dam inventory data were compiled from published landslide inventories, satellite image interpretation, Google Earth Pro, and scientific literature. The Frequency Ratio (FR) technique was applied to quantify the relationship between historical landslide dam locations and each class of the controlling factors. After computing individual FR values,

a cumulative susceptibility index was generated and reclassified into five risk zones: very low, low, moderate, high, and very high.

3. RESULTS

The final susceptibility map of the Alaknanda catchment shows the following spatial distribution across the risk categories: Approximately 22.70% of the area falls under the very low susceptibility zone, indicating minimal likelihood of landslide dam formation. The low susceptibility zone covers the largest portion, accounting for 28.09% of the catchment, followed closely by the moderate susceptibility zone at 26.37%. Areas classified as high susceptibility constitute 16.40%, while the very high susceptibility zone, representing the regions with the greatest potential for landslide dam formation, comprises 6.42% of the total area. The high and very high susceptibility zones are typically located near major faults, steep valley slopes, and river confluences, indicating a strong geomorphic control over potential damming sites.

4. CONCLUSION

The study provides a first landslide dam susceptibility zonation for the Alaknanda catchment using the Frequency Ratio method. The resulting map serves as a valuable tool for regional planning, risk assessment, and early warning system development, especially for infrastructure and communities situated downstream. The approach can be further strengthened through improved landslide dam inventory and integrated into broader disaster risk reduction frameworks in the Indian Himalayas.

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Preliminary note on utilization of Geo−informatics database
— Spatial visualization and clustering —

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ABSTRACT

Kansai Geo-informatics Network (KG-NET) has constructed and maintained the Geo-informatics Database for a long time in the Kansai and Shikoku regions. Currently, the number of boreholes registered in the database has reached approximately 70,000, excluding Shikoku (https://www.kg-net2005.jp/index/db03.html). In order to support the utilization of the boreholes database increasing amount of data, the visualization, which provides an overview of a certain depth range by simplifying the strata, has been developed. A cluster analysis was performed using the original data for visualization of the Yodo River area in Osaka. Boreholes of one cluster are predominantly sand and gravel, and the other boreholes of the other cluster are predominantly clay. Comparing the cluster distribution obtained from the analysis with the flood control classification map, the sand bars and dunes areas were consistent with the cluster distribution, whose boreholes were predominantly sand and gravel. The procedure of the combination of visualization and cluster analysis is expected to contribute effective utilization of the borehole database, which consists of a huge amount of data.

Keywords: borehole data, visualization, cluster analysis

1 INTRODUCTION

Many borehole databases, such as the national land information search site 'Kunijiban' (e.g., MLIT), have been constructed and are open to the public. The Kansai Geotechnical Information Network updates the borehole database by adding about 700 to 1,000 borehole data provided by its members every year. Currently number of boreholes is more than 90,000 for both Shikoku and Kansai (KG-NET, not open to the public). The higher the density of the data in the database, the more reliably the subsurface models can be constructed (e.g., Kimura et al., 2011). The Kansai Geotechnical Information Network has regularly published books that summarize the soil characteristics of specific regions using borehole data (e.g., Geo-Database Information Committee of Kansai, 1998 and Geo-Database Information Committee of Kansai, 2007). However, the map function of the Kansai Geotechnical Information Network database is specialized for selecting borehole data, and it is not possible to directly refer to other map information. Thus, we have used a trial-and-error approach to select and create cross sections from a huge amount of borehole data referring to topographic classification, surface geology. Therefore, we have been developing the display of simple visualized information on a GIS to support the use of drilling databases (Inoue et al., 2024). In this study, I present the results of an attempt to classify regional characteristics by applying cluster analysis to the data used for visualization.

2 DATA AND METHOD

The data used for the analysis were classified into clayey soil (clay and silt), sand, gravel (including sand and gravel), and other materials (e.g., embankments) based on the facies information in the boring data, and the thickness of each facies composition in a certain section was used (Inoue et al., 2023). In this study, data from the Kansai Area Ground Information Network database was extracted from the Yodo River basin in Osaka City with a drilling length of 20 m or more, and the data was used for visualization of the above-mentioned facies changes (up to a depth of 16 m, with a section of 2 m). Fig.1 shows a distribution diagram in which each facies is graphed from shallow to deep every 2 m. The analysis method applied to the classification was the k-means method, which classifies data groups into k clusters. It is a type of unsupervised machine learning and a representative method of non-hierarchical cluster analysis. The only parameter required for the analysis is the number of clusters.

3 RESULTS

The cluster that is most included in each mesh is shown in Fig.2. Comparing with the visualized facies change (Fig.1), it appears that the predominant soil type in Cluster 1 is gravel, and in Cluster 2, clay. The classification results around the Uemachi Plateau, which is approximately in the center of the figure, were consistent with the distribution of sand-bars and sand dunes shown in yellow on the flood control topography classification map in the background map. The distribution of Cluster 1, which corresponds to gravel, appears to extend to the east side of the plateau to some extent.

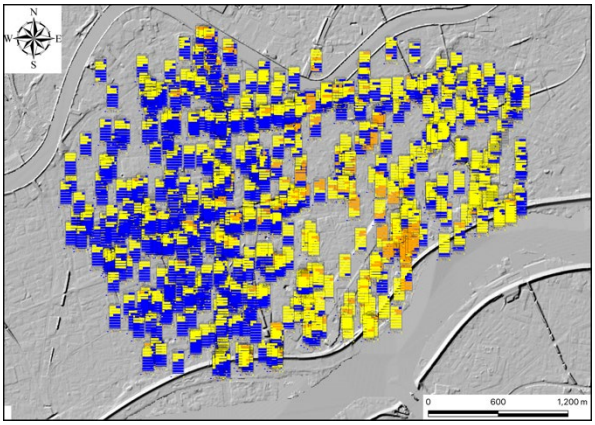


Fig. 1. Visualization of the borehole data used in the analysis (each layer is graphed from shallow (top) to deep (bottom) every 2m; orange indicates gravel layer, yellow indicates sand layer, and blue indicates clayey soil).

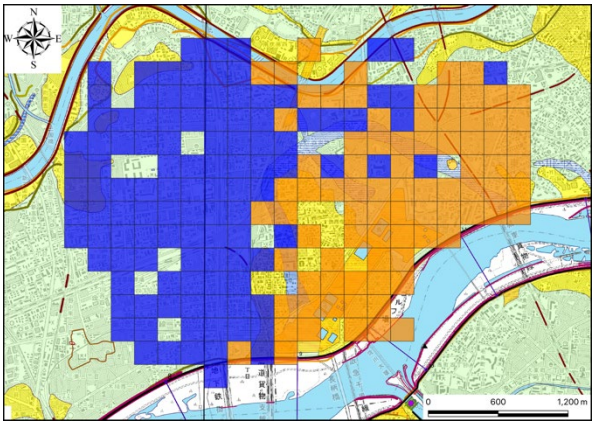


Fig. 2. Cluster classification results for each mesh (orange: cluster 1, blue: cluster 2). The background map is the updated version of the flood control topography classification map (GSI).

4 CONCLUSIONS

I attempted to classify regional characteristics using the k-means method, using shallow soil information from borehole data. When the number of clusters was 2, borehole data were divided into those dominated by gravel and those dominated by clayey soil, and the meshed cluster classification results showed a tendency to be consistent with the flood control topography classification map to a certain extent. In addition, although only facies information was used this time, an analysis incorporating N values would also be effective. In this example, an example with 2 clusters was shown, but I would like to conduct an analysis with an increased number of clusters.

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TC-305 Geotechnical Infrastructure for Megacities and New Capitals

Damage measures of low-rise building foundations on layered sands due to earthquake impacts

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ABSTRACT

The potential damages of low-rise building foundations on liquefiable sites were studied using the Midas GTS-NX analysis. Both free-field and foundation responses for single raft and piled raft foundations were monitored on layered sands using the UBCSand model. It was found that a single raft can be easily uplifted by surrounding groundwater and soil liquefaction occurred. The piled raft foundation has better seismic resistance compared to the single raft. The foundation load will enlarge the ground settlements but reduce the horizontal ground displacements underneath the foundation. The 2D analysis can provide rational predictions of the piled raft responses despite the foundation geometry not being fully simulated. Using a thicker raft can eliminate the angle variable of the foundation. Considerable amounts of foundation settlements were found for single rafts and piled rafts according to the design specification. Longer piles will help to reduce the foundation settlement.

Keywords: low-rise building foundation, raft, piled raft, damage measures, layered sand, soil liquefactionю

1 INTRODUCTION

In recent years, a few devastating earthquakes occurred in Taiwan were found causing various levels of damages to low-rise buildings located in the suburban areas. It was noticeable that the soil liquefactions were exhibiting at the sites. The low-rise buildings built on three- to five-story reinforced concrete structures are mainly used for residential houses. Typical foundation of these structures is the single footings and/or the combined footings. Lu *et al.* (2017) examined several such building structure damages with a preliminary analysis to conclude that these foundations are vulnerable to the soil liquefaction during the earthquake. To understand the applicability of the mat foundation and/or the piled raft foundation for such building structures, the aimed foundation was investigated with a series of numerical studies. The damage measures of these foundations were comparable with the local design specifications (MOI, 2024) to verify their features.

2 METHODS AND RESULTS

The seismic behaviors of light weight building foundations, including both raft foundation and piled raft foundation, on liquefiable sites subjected to earthquake excitations were studied and presented in this paper. Two-dimensional (2D) and three- dimensional (3D) Midas GTS-NX analyses (Midas, 2020) were conducted to simulate the time-dependent responses of the foundation site. A layered sand profile whose thickness is 50 meters was assumed with the Gibson soil hypothesis, in which the shear wave velocity (Vs) of the soil at ground surface is assumed at 130m/s and the Power number of the function is assumed to be 0.33. Poisson's ratio of the soil was given to yield the elastic modulus (G and E) of the soils. Emprical formula of Vs and SPT-N values following the local design code was adopted. The values of corresponding (N1)60 values were found for the sands. Table 1 lists the material properties and strength parameters used in this study. The UBCSand model (Byrne *et al.*, 2004) was adopted to simulate the liquefaction behaviors of the sands. Table 2 summarizes the major formulas and material parameters. The seismic responses of free-field site and foundation site were analyzed for earthquake impacts with target PGA at 0.07g, 0.11g, 0.195g and 0.26g in relation to respective intensity levels. The seismic records of a few seismic stations in Taipei for Chi Chi earthquake (1999.09.21) and Hwalien earthquake (2002.03.31) were used as the input motions. Th scaling method, the Kaul method and the uniform stress-cycle method were studied to monitor their influences.

The time-histories of the shear stresses, shear strains, excess pore water pressures (EPWP) and the ground displacements were monitored. Colorful contours and vectorized displacements diagrams were made to show the variations of the mechanism. Corresponding damage measures of the foundations, in terms of the settlements

and tilts, were analyzed and compared to the design specific limits. The significance of the near-fault effects was also reported. It was found that the raft foundation can enlarge the ground settlement underneath the foundation load. The ground surface horizontal displacement asides the foundation was found increased. To increase the embedment depth and the width of raft can reduce the foundation responses due to the seismic impacts. The 3D analysis is especially important to the piled raft foundation analysis. 2D analysis can overestimate the EPWP influences to lift the foundation should the groundwater level is nearby the raft. The full soil-structure interaction effects with the inclusion of the building in the 2D and 3D analysis were compared to the kinematic interactions of the alone foundation model. The inertial soil-structure interaction became significant when the building height increased. Increasing the rigidity of mat is helpful to eliminate the potential damage of the building foundation. Liquefaction potential analysis made by various empirical methods can also provide rational agreements with the results from the ground response analysis of the free-field site.

Table 1. Fundamental properties and parameters of the soils used in this study.

Soils	Geometry, fundamental properties and parameters
Sand	GWT= EL-2m, H1=EL0m~EL-2m, H2= EL-2m~EL-10m, H3=EL-10m~EL-20m, H4=EL-20~EL-30m, H5=EL-30~EL-50m; γ_d= 18 kN/m^3, γ_{sat}=20kN/m^3; ν_s (Poisson's ratio) =0.3 (dry) and 0.4 (saturated); V_s=80(N)$^{0.33}$ where N is SPT-N value measured in the field; E_s (Young's Modulus) =2V $^2\rho_s(1+\nu_s)$ where ρ_s is the mass density of soil; $_s$ E_s= 99~264 MPa for sandy layers; *c'*=0 kPa; ϕ*'* =28°~35°; *Dr* =39~51 (%)

Table 2. UBCSAND model (Byrne et al., 2004) and model parameters in use.

Formulas	Definition and value of model parameter	Formulas	Definition and value of model parameter
G^e or $G^e_{new} = K^e_G Pa(\frac{\sigma m'}{Pa})^{ne}$	G^e: elastic shear modulus; G^e_{new}: updated shear modulus; K^e: elastic modulus constant (500-2000)*; Pa: atmospheric pressure; σ_m': mean effective stress; ne: model parameter (can be taken as 0.5)	$\Delta\varepsilon_v^p = (sin\phi_{cv} - \frac{\tau'}{\sigma_m})\Delta\gamma^p$	$\Delta\varepsilon_v^p$: plastic volumetric strain; ϕ_{CV}: constant volumetric friction angle (between 30°-34° average is used); $\Delta\gamma^p$: plastic shear strain increment
$G_t^p \cong 3.7(Dr)^4 G^e_{new} + Pa$	G^p: initial plastic shear modulus; *Dr*: relative density (33%, 45% and 54% were used)	$M = K_m Pa(\frac{\sigma_v'}{Pa})^m$	M: constraint tangent modulus; K_m: material constant on constraint tangent modulus (taken as 1600); σ_V': effective vertical stress; m: model parameter (suggested as 0.5)
$G^p = G_t^p(1 - \frac{\eta}{nf}R_f)^2$	G^p: plastic shear modulus; η_f: shear stress ratio at failure; Rf: calibration factor between 0.7-0.98 (averaged value is used)	*Note: K_G^e is estimated using the formula $434 \times (N_1)_{60}^{0.33}$ suggested by Beaty and Byrne (1998)	

10 CONCLUSIONS

The applicability of the raft and the piled raft foundation for the low-rise building on liquefiable soil profile was investigated numerically in this study. For local intensity levels of V+ (PGAT at 0.195g) and VI- (PGAT at 0.26g), large foundation settlements (>30cm) were observed for raft foundations. The 1.5m thick slab can produce a rigid foundation rotation in which the angle variable can be restraint according to the design code. Such amount of rotation can still damage the building and need to be cautioned.

The ground motion input investigation indicates that the seismic motion in use must reflect the natural site characteristics. Rational predictions can be obtained by using either one or several seismic records. The Kaul method (in which the design spectrum is considered) was found more conservative in comparison with the Scaling method (in which only PGAT is considered). When applying the uniform stress-cycle method, the optimum time period was found as 1sec for the cases where Nc is less than 8.

ACKNOWLEDGEMENTS

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Dynamic analyses of a fill dam by elasto-plastic constitutive models

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ABSTRACT

The settlement of the top of Ushirotani Dam during the main shock of the Chuetsu Offshore Earthquake (2007) was approximately 30cm. The dam just been completed before the earthquake and was not storing water. So, the materials of dam are in unsaturated condition. The acceleration measured at the KIK-NET Kawanishi observation point was used to the dam foundation. Since the epicentral distance from the Chuetsu offshore earthquake is approximately 10 km, the input earthquake magnitude was adjusted. The elastoplastic models were applied and the calculated deformations are compared with the settlement measured at the top of the dam body after the earthquake. The constitutive models used were 1) Perfectly elastoplastic model, 2) Isotropic hardening/softening model, and 3) Anisotropic hardening/softening model. The results by calculations showed settlements of 12cm (Model 1) ,18cm (Model 2) and 43 cm (Model 3). The actual measured settlement of the dam crest is around 30 cm.

Keywords: fill dam, dynamic analyses**,** elasto-plastic models, finite element method, settlement

1 INTRODUCTION

The settlement of the top of Ushirotani Dam during the main shock of the Chuetsu Offshore Earthquake in Japan (occurrence time 2007/7/16-10:13:00) was approximately 30cm. The dam just been completed before the earthquake and was not storing water. So, the materials of dam are in unsaturated conditions

A simple perfectly plastic material model (Model 1) for soil is used with the features of non-associated flow characteristics. Then a kinematic hardening model considering the cumulative deformation by cyclic loading is developed based on the soil model of isotropic strain-hardening-softening property. It is shown that the dynamic relaxation method (Tanaka & Kawamoto, 1988) combined with the generalized return mapping algorithm (Ortiz & Simo 1986) is effective in static analyses (Tanaka 1992) and finite element simulations of tri-axial tests are carried out. In the case of kinematic hardening elastoplastic constitutive model (Model 3), cyclic behaviors of unsaturated material deformation are taken into accounts based on modified and extended soil model of isotropic strain-hardening-softening property (Model 2).

2 CROSS SECTION OF USHIROTANI DAM

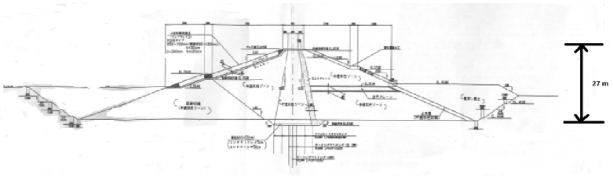


Fig. 1. Cross section of Ushirotani Dam (height is 27m).

The acceleration measured 200m deep underground at the KIK-NET Kawanishi observation point was input to the dam foundation. The height of this dam is 27m and the finite element mesh is shown in

Fig. 2.

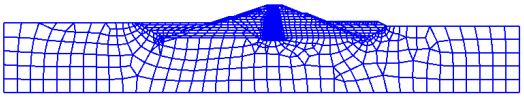


Fig. 2. Finite element mesh of Ushirotani Dam.

3 ELASTO PLASTIC DYNAMIC ANALYSES

Since the epicentral distance from the Chuetsu offshore earthquake is approximately 10 km, the input earthquake magnitude was adjusted and the results are shown next figure.

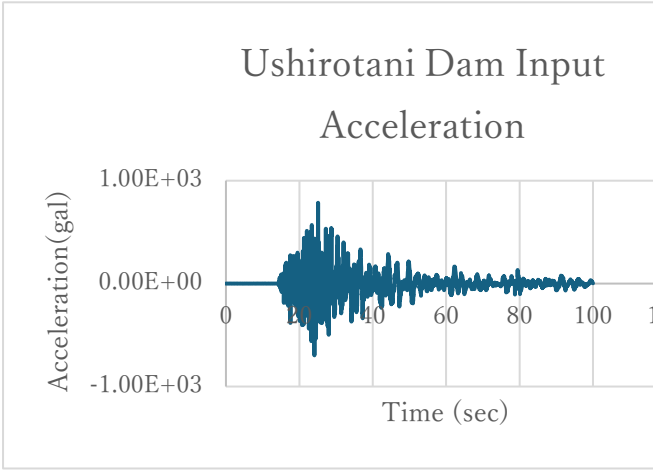


Fig. 3. Input acceleration to Ushirotani Dam.

The elasto-plastic constitutive models were applied and the calculated accelerations and deformations are obtained. The constitutive models used were 1) elasto-perfectly plastic model (Model 1), 2) isotropic hardening/softening model (Model 2), and 3) kinematic hardening/softening model (Model3).
The results by calculations assuming the same elastic properties for all models showed settlements of

12cm (Model 1) ,18cm (Model 2) and 43 cm (Model 3). The actual measured settlement of the dam crest is around 30 cm.

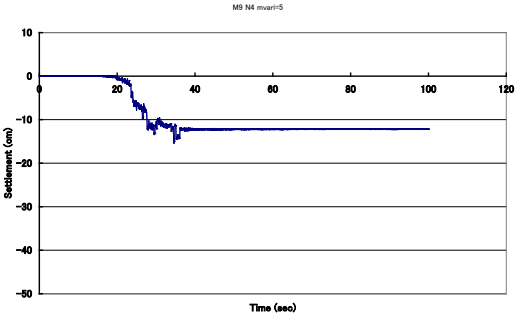


Fig. 4. Computed settlement at the top of dam by perfectly plastic constitutive model.

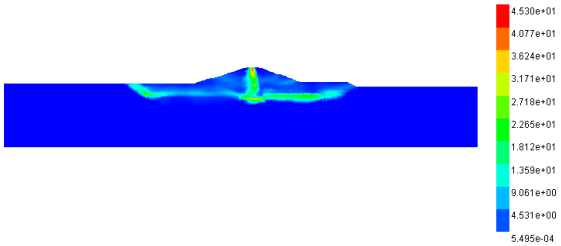


Fig. 5. Computed maximum shear strain by perfectly plastic constitutive model (%).

4 CONCLUSIONS

The constitutive models of perfectly elastoplastic model, isotropic hardening/softening model, and anisotropic hardening/softening model are applied to the prediction of real fill dam settlement. The elasto-plastic analyses are promising for the prediction of fill dam deformations by an earthquake.

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Effect of strength reduction factor on settlement of a single pile

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ABSTRACT

Nowadays, due to the increase in the number of storeys of structures under construction, the number of buildings erected on pile foundations has increased. A large number of theoretical and experimental studies of pile operation under load have allowed the development of various methods for calculating pile settlement. However, most of them do not take into account the technology of pile construction, which affects the change in the properties of the contact zone soils. In addition, most of the methods assume that the settlement of the pile is equal to the settlement of the soil, but in reality, after reaching the ultimate strength, the pile has the possibility of slipping through the soil.
The main purpose of this paper is to investigate the influence of the interface friction coefficient (strength reduction factor R_{inter}) on the results of pile settlement calculations performed by numerical method in the geotechnical software package Plaxis. In this paper, the interface friction coefficient was determined from the results of laboratory shear tests of sandy and clayey soils over concrete, in accordance with the current regulatory documentation (SP 22.133330.2016 «Soil bases of buildings and structures») and back calculation based on field tests. Based on the results of the calculations, graphs of load-dependent settlement were plotted and a comparative analysis of the results obtained with the results of known field tests of bored piles by static loading was performed. Based on the comparative analysis, conclusions were drawn about the influence of the method of determining the interface friction coefficient on the pile settlement calculation results, the influence of the pile construction technology on the interface friction coefficient value and recommendations on how to take this coefficient into account in the calculations.

Keywords: pile-soil interaction, direct shear, the interface friction coefficient, numerical simulation, pile settlement, field test.

1 INTRODUCTION

It is well known that at the contact of structures with the soil there is a change in mechanical characteristics depending on various factors, which can be accounted for by such a parameter as the interface friction coefficient (strength reduction factor R_{inter}). When designing, the consideration of this coefficient and its value have a significant impact on the forces in the structures and their displacements. The greatest influence on the properties of the contact zone, i.e. the value of the interface friction coefficient, which is characterized by the change in the properties of the contact zone, is exerted by the technology of constructing structures in the soil. In this case, mechanical characteristics at the contact can both decrease and increase.

In addition to the pile-in-soil technology, the change in mechanical properties is also affected by the type of soil and the pile material, as contact friction is characterized by the meshing of soil particles with the roughness of the construction material. Therefore, the rougher the particles and rougher the surface of the construction material, the higher the friction will be and vice versa. The interface friction coefficient (strength reduction factor R_{inter}) can be determined in the following three ways:

1. In accordance with the current Russian regulatory documentation (SP 22.133330.2016 «Soil bases of buildings and structures») [6]. In this case, this coefficient will depend on the construction technology, soil type and moisture content, as well as on the construction material. However, it is not always possible to use the coefficients proposed in the normative documentation in analytical and numerical calculations, for example, when it is equal to 0, because it immediately indicates the lack of contact and failure strength. In addition, the proposed coefficients are given very conditionally, without reliance on experimental studies, i.e. often these values are far from reality.

2. Based on laboratory shear tests in direct shear apparatus, torsional shear apparatus or simple shear apparatus. In this case, the tests shall be performed in two patterns: «soil-soil» and «soil-solid». Based on the laboratory test results, the interface friction coefficient will be defined as the ratio of the ultimate resistance between the soil and the solid material to the ultimate shear resistance obtained from the soil-on-soil test. This takes into account the type of soil, its moisture content and the material from which the proposed structure will be constructed, but does not take into account the construction technology, which is one of the main factors affecting the value of this coefficient.

3 The third method for determining this coefficient is the most accurate, as it is based on the results of field tests of piles at the construction site. Based on the results of the field tests, a graph of the load-dependent settlement or shear stress variation over depth is plotted, and then, by inverse calculation using the numerical method, such interface friction coefficients are selected so that the graphs of the load-dependent settlement and tangential stress variation over depth obtained by the numerical method coincide with similar graphs obtained on the basis of field tests. Thus, this method not only takes into account the type of soil and material, but also takes into account the pile technology.

2 MATERIALS AND METHODS

In this paper the interface friction coefficient (strength reduction factor R_{inter}), which was determined in three different ways/

- in accordance with the current Russian normative documentation SP 22.133330.2016 «Soil bases of buildings and structures»;
- on the basis of laboratory shear tests in a direct shear apparatus, presented in Figure 1;
- by inverse calculation by numerical method in the geotechnical program *Plaxis* on the basis of static tests of bored piles.



Fig. 1. The device of direct shear.

3 RESULTS AND DISCUSSION

Based on the results of the static test of the bored pile and the results of the numerical modeling, the graphs of settlement versus load were plotted as shown in Figure 2. The accepted values of the interface friction coefficient R_{inter} for each soil layer depending on the method of its determination are given in Table 1.

Table 1. Interface friction coefficient values R_{inter}.

№ EGE	Interface friction coefficient values R_{inter}		
	SP 22.133330.2016 [6]	Direct shear test	Inverse calculation based on static test results
4	0.50	0.45	0.80
14	0.33	0.62	0.90
16	0.50	0.45	0.80

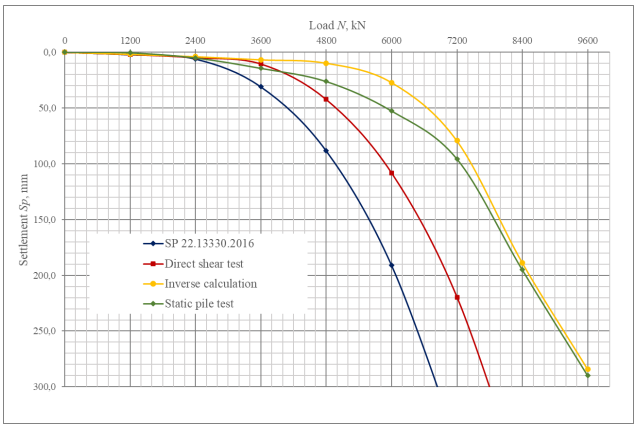


Fig. 2. Graphs of settlement-load dependence obtained from static pile test results and from numerical calculation results depending on the method of determining R_{inter}.

4 CONCLUSIONS

1. This paper investigates the influence of the method of determining the interface friction coefficient (strength reduction factor R_{inter}) on the convergence of the results of pile settlement calculations performed numerically in *Plaxis* and obtained from the results of known field tests of the pile by static loading.

2. In this work, the interface friction coefficients R_{inter} were determined in three ways. The worst convergence of the graphs of settlement-load dependence with the results of static pile test is observed when determining R_{inter} based on SP 22.13330.2016, and the best convergence was obtained when determining R_{inter} by inverse calculation.

4 The third method for determining the interface friction coefficients R_{inter} is the most accurate, because is based on the results of field tests of piles at the construction site. Thus, this method allows to take into account the type of soil and material, and also takes into account the technology of pile construction, which is one of the most important factors affecting the mechanical characteristics of the soil at the «pile – soil» contact.

Numerical Analysis on Response of Battered Pile under Combined Lateral and Uplift Loading

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ABSTRACT

This study presents a comprehensive numerical investigation into the behaviour of batter piles subjected to combined loading conditions using plaxis-2D. A finite element-based geotechnical analysis software. The primary aim is to understand the influence of both batter angle and pile slenderness (expressed in terms of the length-to-diameter ratio, L/D) on the response of single pile under combined loading.

To assess the geometric impact, batter angles of 0° (vertical), 10°, 20°, 30°, and 35° were selected, covering a wide range of inclinations. Furthermore, L/D ratios of 10 and 17 are considered. The pile response under uplift loading with and without the presence of lateral loading was analysed. Different magnitudes of constant lateral load were considered for the analysis.

1 INTRODUCTION

Batter piles are inclined foundation elements used to resist both vertical and horizontal loads in structures. Unlike vertical piles, they are driven into the ground at an angle to provide greater stability against lateral forces such as wind, waves, and earthquakes. This study focuses on the behaviour and performance of batter piles under different load conditions. Using numerical modelling and analysis, the research aims to understand how pile angle affects load-bearing capacity. The findings can help improve design strategies for structures in challenging environments.

Most existing studies focus on lateral or axial (uplift) loading of batter piles independently, with limited research on their combined effect, particularly using numerical models like PLAXIS 2D. Additionally, the impact of varying batter angles under combined loading in dense sand remains underexplored, leaving a gap in optimizing design parameters for such conditions.

2 METHODOLOGY

The pile was modelled as a linear elastic beam element, and the surrounding soil is modeled as Mohr-Coulomb constitutive model, which provides a simplified but effective approximation of soil strength parameters for static conditions. The simulations were carried out under plane strain conditions, and refined meshing was implemented around the pile to ensure accurate capture of displacement gradients and stress redistribution during loading. The soil properties Modulus of elasticity (E) is 50Mpa, Unit Weight of soil is taken as γ =20kN/m^3,Φ=37^0,C=4.88kN/m^2 and Poisson's ratio as 0.3.

The pile was modelled as a linear elastic beam element, and the surrounding soil was represented by the Mohr-Coulomb constitutive model, which provides a simplified but effective approximation of soil strength parameters for static conditions. The simulations were carried out under plane strain conditions, and refined meshing was implemented around the pile to ensure accurate capture of displacement gradients and stress redistribution during loading.

3 NUMERICAL ANALYSIS

To assess the geometric impact, batter angles of 0° (vertical), 10°, 20°, 30°, and 35° were selected, covering a wide range of inclinations. Furthermore, L/D ratios of 10 and 17 are considered. The pile response under uplift loading with the presence of incremental lateral loading was analyzed. Different magnitudes of constant lateral load as 25%,50%,75% and 100% were considered for the analysis.

Battered piles embedded in dense granular cohesionless soil has been considered, since the tall buildings generally would demand pile foundation of high bearing capacity even in dense sand conditions. Battered piles installed at different batter angles: 0°, 10°, 20°, 30°, and 35° are analyzed. Piles having 1m diameter

and two lengths (L = 17 m and L = 10 m) were considered. The analysis of results indicate that pile inclination influences the resistance and bending behavior of piles under combined loading. The results indicate that batter angles enhance lateral capacity by decreasing the lateral displacement of pile head as shown in fig.1 and fig.2. Under combined loading, it is noticed that the response of battered pile is governed by the magnitude of lateral and axial uplift force components, and batter angle.

Table 1 Displacement data for Combined Uplift and Lateral Load for L/D=17

Uplift (U) with % Lateral (L)	L/D = 17					
	Cases	0^0	10^0	20^0	30^0	35^0
		Displacement (mm)				
361.67kN (U) , 167.5 kN (L)	Case1	7.3	4.43	3.15	2.11	2.54
361.67 kN (U) , 335 kN (L)	Case2	17.84	13.6	11.46	5.94	4.85
361.67 kN (U) ,502.5 kN (L)	Case3	30.91	24.75	21.84	13.75	11.71
361.67 kN (U) , 670 kN (L)	Case4	46.22	37.93	33.92	22.8	19.4

Table 1 Displacement data for Combined Uplift and Lateral Load for L/D=10

Uplift (U) with % Lateral (L)	L/D = 10					
	Cases	0^0	10^0	20^0	30^0	35^0
		Displacement (mm)				
169 kN (U) , 167.5 kN (L)	Case 1	7.48	6.139	5.0	3.9	3.7
169 kN (U) , 335 kN (L)	Case 2	18.93	16.35	13.8	11.22	10.22
169 kN (U) , 502.5 kN (L)	Case 3	34.31	29.4	24.85	20.66	18.65
169 kN (U) , 670 kN (L)	Case 4	53.79	46.76	37.62	32.29	29.08

4. ANALYSIS

Results from the combined load with varied lateral load simulations indicated that vertical piles (0°) experienced the highest lateral displacements, particularly at full loading. As the batter angle increased to 10°,20°,30° and 35° the lateral displacement of batter piles gradually decreases. This is because the pile aligned with the direction of the applied load, allowing it to resist forces more effectively. On the other hand, under uplift-only loading, displacement initially reduces as the batter angle increases, reaching its lowest value around 20°. However, at steeper angles like 30° and 35°, the displacement starts to increase again. This shift is likely due to reduced shaft friction and possible separation between the pile and surrounding soil, which limits the pile's ability to resist uplift. These findings suggest there is an optimal batter angle that minimizes uplift displacement.

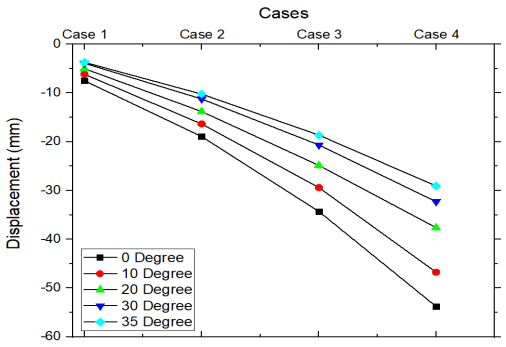


Fig. 1. Load vs Displacement L/D =10

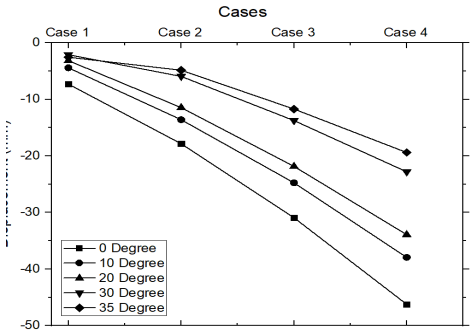


Fig. 1. Load vs Displacement L/D =17

5. RESULTS

The results for piles with an L/D ratio of 10 show that lateral displacement decreases consistently as the batter angle increases from 0° to 35°. This trend is observed across all loading cases, indicating that steeper batter piles offer improved resistance under combined loading conditions. The pile inclined at 35° exhibited the least displacement, while the vertical pile (0°) experienced the highest. This behavior highlights the enhanced efficiency of inclined piles in mobilizing soil resistance and reducing lateral deflection.

A similar decreasing trend in displacement with increasing batter angle is observed for both L/D = 10 and L/D = 17. However, the longer pile (L/D = 17) shows slightly lower overall displacement, indicating improved stiffness and deeper soil interaction with increased slenderness.

Studying the mixing of problematic soils

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ABSTRACT

Problematic soils are founded in many regions in the world. It's necessary to explain problematic soils properties due the damage caused by that soils. The results of experimental tests on the engineering properties of problematic soils are presents in this paper, and the effects of mixing that soils together on that properties are showed. Two types from the problematic soils : Gypseous soil, laterite soil are used in this study. Seven percentage of gypseous soil (5%,10%,15%,50%,85%,90% and 95%) by weight of the dry laterite soil were mixed. The tests includes evaluation of properties : consistency limits, compaction, shear strength , and compressibility. A series of tests using maximum dry density and optimum moisture content were performed and the shear strength (τ) and compression index (Cc) evaluated according to percentages for the Gypseous soil added to laterite soil. Results of the study showed that gypseous soil increased the maximum dry density (γ_d). The increase in laterite soil (SM) content decreased the plasticity index of gypseous soil (SP).

Keywords: problematic soils ,gypseous soil, laterite soil, shear strength, collapsibility, compressibility**.**

1 INTRODUCTION

Problematic soils are founded in many regions in world. Many types of soils classify as problematic in field of geotechnical engineering because collapse, undergo excessive settlement , expand and have a distinct lack of strength [1]. Problematic soils are characterized by giving rise to major problems and causing loss of life and financial loss [2]. There are essentially six main types of problem soils: Expansive Soils, cause of this soil is the presence of swelling clay minerals. Soil's volume change is determined by its initial moisture content, void ratio, vertical stress as well as the number and kind of clay minerals [3].

The aim of this study was effects of mixing soils on that properties, two types of problematic soils investigated: Tropical soils and Collapsible soil. Also observed the influence of mixing that soils together on engineering properties.

2 MATERIALS USED

The soils samples investigate in this study were relatively disturbed samples gypseous soil (SP =poor sand), laterite soil (SM = silty sand), gypseous soil obtained from IRAQ and laterite soil from MALAYSIA.

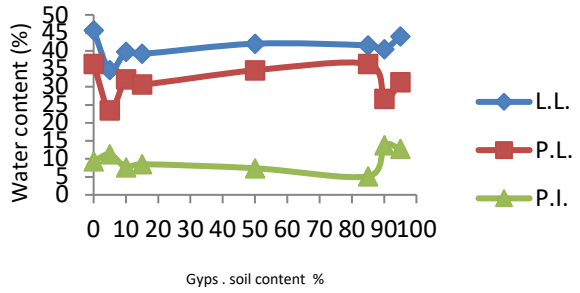
3 EXPERIMENTAL TESTS

Laboratory investigation has been done at the mixing of various concentrations of Gypseous soil was (5, 10, 15, 50, 85, 90 and 95%) by dry weight of laterite soil. After obtain maximum dry density and optimum moisture content from compaction test for each percent mixed, tests under that density and moisture content were performed and the consistency limits**,** shear strength , and compression index evaluated.

4 RESULTS AND DISCUSSIONS

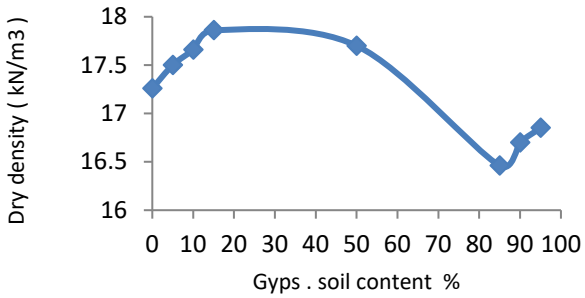
4.1 Consistency limits

Variation of liquid and plastic limits for different Gypseous soil content added to laterite soil is shown in (Fig. 1).



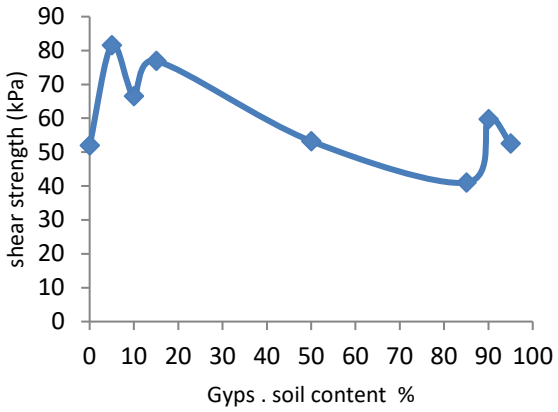
4.2 Compaction characteristics

(Fig. 2) shows the relationship between the maximum dry density (γ_d) and soil additive contents added to laterite soil.



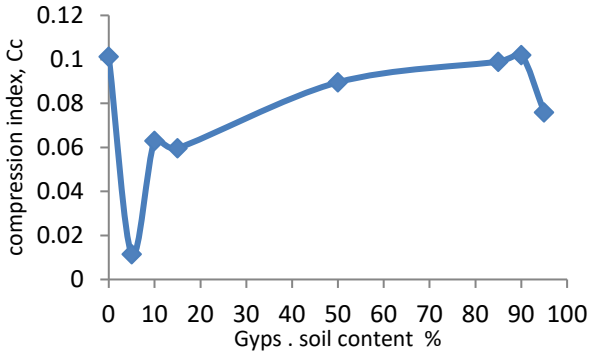
4.3 Strength characteristics

The effect of Gypseous soil on the shear strength of laterite soil specimens compacted at different dry densities with Gypseous soil percentages present in (Fig. 3).



4.4 Compressibility Test

From (Fig. 4) shows the relationship between compression index (Cc) for laterite soil and the percentage of Gypseous soil added to that soil.



5 CONCLUSIONS

The following conclusions may be drawn from the study:

1. Decreased the plasticity index for (SM) soil when increase in content of gypseous soil.
2. low percentage of gypseous soil added to laterite soil increase (γ_d) and after 50 percent for gypseous soil the max. dry density decreased
3. Decrease in strength with increase gypseous soil in laterite soil
4. Increase Gypseous soil percent increased the compressibility of laterite soil.
5. The little content of gyseous soil was observed to be the optimum content.

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Application of Electrical Tomography for Studying the Lithological Structure of Foundation Soils for the Construction of a Unique High-Rise Building in Astana

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ABSTRACT

The article discusses the application of the modern geophysical method of well electrical tomography to determine the geophysical properties and lithologies of a site for construction. Geophysical investigations of the foundation soils were carried out using a vertical geophone BHG-2 with a frequency of 10 Hz. As a result, resistivity profiles were obtained, which varied up to depths of 10-15 meters within the range of 1-100 Ω·m. At depths greater than 15 meters, the resistivity varied between 100-1000 Ω·m. The interpretation of geophysical measurements was performed using the software packages REZ2DINV and Surfer 9. The geophysical results allowed for the identification of lithological layers based on different resistivity levels. Coarse-grained soils and rocks containing sedimentary deposits showed higher resistivity compared to fine-grained soils. The article discusses the application of the modern geophysical method of well electrical tomography to determine the geophysical properties and lithologies of a site for construction. Geophysical investigations of the foundation soils were carried out using a vertical geophone BHG-2 with a frequency of 10 Hz. As a result, resistivity profiles were obtained, which varied up to depths of 10-15 meters within the range of 1-100 Ω·m. At depths greater than 15 meters, the resistivity varied between 100-1000 Ω·m. The interpretation of geophysical measurements was performed using the software packages REZ2DINV and Surfer 9. The geophysical results allowed for the identification of lithological layers based on different resistivity levels. Coarse-grained soils and rocks containing sedimentary deposits showed higher resistivity compared to fine-grained soils.

Keywords: resistivity, soil, electrotomography, lithological structure

1 INTRODUCTION

The territory of Astana is characterized by a folded zone with significantly varying rock geology. The relief is flat, reflecting the semi-desert steppe area. The construction of a unique high-rise complex was carried out in the capital of the Republic of Kazakhstan, Astana, located on the left bank of the Esil River at the intersection of Sagynak and Turkestan streets. The total area is 130,000 m² (see Figure 1), which includes: 36,000 m² of four-level underground parking, 15,000 m² of two-level podium, 60,000 m² of office towers with heights ranging from 39 to 75 floors (including the basement and podium), and a 17,000 m² residential tower with 18 floors.



Fig. 1 The territory of the site and the general view of the Abu Dhabi Plaza complex.

2 MATERIALS AND METHODS

The soil conditions at the construction site include a thin layer of embankment soil, lying above Quaternary alluvial deposits at a depth of 10 meters below ground level, above solid deposits, which consist of highly weathered rocks located above the rocky foundation. The upper layer of alluvial deposits consists of a fine-grained loam layer at a depth of 5 meters, while the layer below is made up of coarse-grained soil, consisting of sand and gravel. The lower alluvial deposits contain rocks formed by the leaching of substances from the soil and include coarse-grained deposits of varying thickness, consisting of sand layers. There are also thin, lens-shaped layers of fine-grained soil (loam and clay).

The fully weathered rocks, which have undergone chemical weathering (leaching), exhibit the disappearance and movement of soluble soil particles. This layer is diverse, primarily composed of binding materials such as fine-grained substances like loam and clay with a plasticity index of 155, with possible inclusions of aleurolite making up approximately 20-40%. The thickness of this layer varies from 1 to 14

meters. The highly weathered rocks include fragmentary materials formed from the weathering of intact rocks (depicted as gravel in the logging diagrams). Inside this layer, there are zones of completely weathered rocks in the form of sandstones and aleurolites.

The rocky foundation consists of layered rock belonging to the Ordovician period. The rock is composed of thin interbedded layers of dark gray aleurolite and gray-green sandstone.

The application of the electrical tomography method was conducted on two boreholes with a depth of 55 meters. A geophone was installed in one of the boreholes, and a shear wave hammer was placed in the neighboring borehole at the same depth. The shear wave hammer was lowered using a cable. Data recording was carried out at each geological layer depth, and calculations were made based on the distance between the boreholes and the wave travel time.

During the geophysical studies, a cutting frequency of 10 Hz was used. Two horizontal and one vertical geophone sets were employed. Vertical and horizontal impulses were generated at a distance of 1 meter from the borehole mouth, and recordings were made up to 256 nanoseconds.

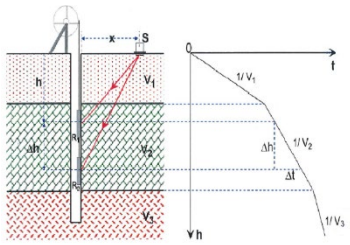


Fig. 3 schematic representation of the electrotomography

The method involves determining seismic velocities and depths. These testing methods are limited to determining interval velocities based on the arrival time and relative arrival time of compression (P) waves and vertically (SV) and horizontally (SH) polarized shear (S) seismic waves, which are generated near the surface and propagate downward to a set of vertically installed seismic sensors (see Figure 4). Data is collected from the study site in a format compatible for processing with the REZ2DINV software. After the data is interpreted using the software, their 2D resistivity sections are created using Surfer 9. Thus, ERT sections are obtained and then lithologically interpreted.

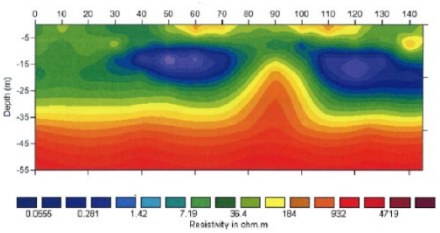


Fig. 5 - The results of the electrical tomography profiles ERT1/ERT2.

CONCLUSION

The tests conducted at the site were carried out in all boreholes to determine geotechnical parameters for assessing lation's engineering conditions. The geophysi l of electrical resistivity tomography allowed f ermination of geophysical properties and the l l structure of the investigated area. Due to the current site conditions, weather conditions, and the geological environment in the study area, low resistivity values were calculated at depths up to 10-15 meters, ranging from 1 to 100 Ohm·m. These soil layers consist of clay, sand, and silt. Physical properties, such as resistivity values, complement the information for soil identification and classification.

In all ERT profiles, resistivity values increase with depth in direct proportion. This increase is due to the transition of the soil medium to a rock medium. Resistivity values in these units range from 100 to 1000 Ohm·m. As the depth increases, the resistivity of the rock units becomes higher, and the fractures within the units decrease. According to all ERT studies, no contact zones or faults were identified within the depth of the investigation in the study area.

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TC-305 Geotechnical Infrastructure for Megacities and New Capitals

Case Study of Retaining Wall Reinforcement on Slope Stability

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ABSTRACT

The slope stability on reinforce retaining wall is influenced by natural factors such as rainfall, topography, and soil conditions, as well as human-induced factors like excavation and embankment construction. Various slope failure factors are closely related to each other. In particular, the stability of existing slope structures is significantly affected by factors such as discontinuities, groundwater levels, rainfall, and soil properties. This study presents a case where the risk of collapse due to discontinuities was identified during the excavation of the slope at the top of a retaining wall, leading to the reinforcement of both the structure and the slope. The length is approximately 193m, and the height is about 15.5 m. The main geological layers consist of weathered soil, weathered rock, and soft rock. The investigation revealed a high possibility of collapse in the existing slope. The retaining wall was reinforced with permanent anchors (CTC=4.0m), while the slope section was reinforced with soil nailing (CTC=1.5*1.5). As a result, for the stability of slope structures, excavation and slope reinforcement are necessary based on the soil and rock conditions, along with continuous maintenance. Furthermore, thorough drainage management is essential to prevent subsidence at both the upper and lower parts of the cut slope and to avoid the occurrence of seepage pressure during operation.

Keywords: slope stability, retaining wall, collapse, rock condition, seismic, seepage pressure

1 INTRODUCTION

The collapse of structure and slopes is influenced by both natural factors, such as rainfall, topography, and soil conditions, as well as artificial factors, including excavation and embankment construction. The causes and forms of structural and slope failures are diverse, and these factors are related each other, therefore it is impossible to attribute the collapse of retaining walls and slopes to a single cause. In particular, the stability of existing structures and slopes is determined by factors such as discontinuities, groundwater levels, rainfall, and soil characteristics.

Slope failure cases in Korea are mostly caused by rainfall, soil characteristics, and artificial factors such as road construction and housing site development. Notable collapse incidents include the slope collapse in Yongin road construction site (2002), slope collapse in Sangdo-dong housing site construction, Seoul (2018), soil collapse in Hongcheon road construction site for intersection improvement (2020), most of which were triggered by soil runoff during heavy summer rainfall. Even recently, slope collapse accidents are still occurring frequently.

In this study, investigations and analyses were conducted to prepare for the reinforcement of existing retaining walls and the prevention of upper slope collapses at the road construction site Gurye-gun, Jeollanam-do, Korea. Construction site with road network in this research is shown in Fig 1.

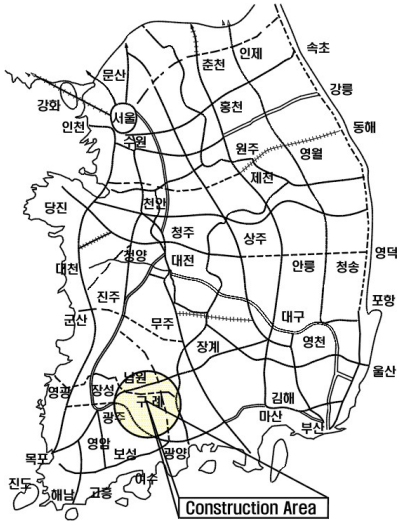


Fig. 1 Construction site with road network in this research

Study of rutting on roads of Shymkent city

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ABSTRACT

The Shymkent city, located in the south of Kazakhstan, has become a megalopolis. The population and traffic intensity on roads of the city are growing, which outpace the rate of increase in road strength. The traffic flow contains public transport with a large capacity and heavy trucks. The climate of the region is characterized by a long hot period. The above factors caused the appearance of rutting on asphalt concrete road surfaces in the city. This paper studies rutting on city roads with asphalt concrete pavement. In field conditions, the characteristics of rutting and influencing factors were determined.

Keywords: city roads, asphalt concrete pavement, rutting, temperature, transport flow.

1 INTRODUCTION

One of the most common types of destruction of flexible road pavements of roads is rutting. Research shows (Teltayev et al., 2024; Teltayev et al., 2025) that the cause of rutting on the roads of large cities is the combined effect of factors (high temperature, heavy load, weak road pavement, insufficient shear resistance of materials and soils, etc.).

This paper examines rutting on a section of a road in the Shymkent city, located in the south of Kazakhstan.

2 EXPERIMENTAL SECTION

To study rutting in asphalt concrete layers of road pavement, a section of the A-2 main road in the city of Shymkent was selected.

By opening the road pavement structure on the experimental section was installed:

Layer 1: stone-mastic asphalt concrete (SMA-20), thickness
h_1 = 5 cm;
Layer 2: Coarse-grained porous asphalt concrete, h_2 = 7 cm;
Layer 3: Crushed stone treated with bitumen, h_3 = 5 cm;
Layer 4: Sand and gravel mixture, h_4 = 80 cm;
Subgrade soil: loam.

The general view of the section is presented in Fig. 1.



Fig. 1 General view of the experimental section

The location of the opening of the road pavement structure is presented in Fig. 2

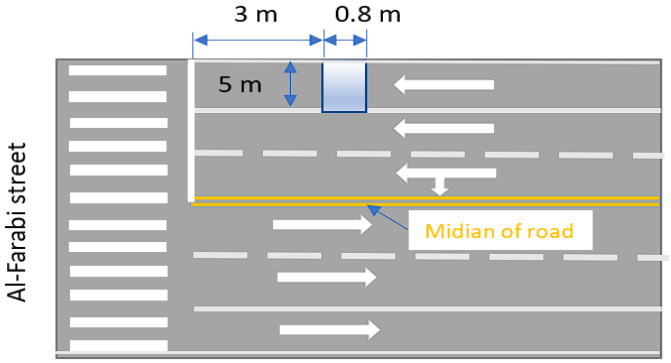


Fig. 2 Place of opening of the road pavement structure at the experimental section

3 GEOMETRIC AND STATISTICAL CHARACTERISTICS OF RUT

On the experimental section, the measurement of the rut depth on the asphalt concrete surface was performed on seven lines located at different distances from the stop line: 0; 5; 10; 15; 20; 25; 30 m.

For a visual representation, graphs of the distribution of the rut depth on the asphalt concrete surface by the width of the selected traffic lane were constructed (Fig. 3). As can be seen from Figure 3 that:

1) the maximum values of the rut depth on different lines are different and range from 40 mm to 55 mm; the maximum values equal to 55 mm are noted on the 20 m and 30 m sections;

2) On all sections, “crests” are observed, formed as a result of shifts of asphalt concrete from the wheel paths to the adjacent (“free” from loads) sides of the traffic lane.

The histogram shows (Fig. 4) that almost half of all measured rut depth values (40%) are within the range of 0 to 10 mm; 16% and 18% of them fall within the intervals of 10-20 mm and 20-30 mm, respectively; 12% are within the interval of 30-40 mm; 10% are within the intervals of 40-50 mm; in the remaining intervals, the number of rut depths is less than 3%. Thus, (74%) of all (179) measured rut depth values are within the intervals of 0-10 mm, 20-30 mm, 30-40 mm and 40-50 mm.

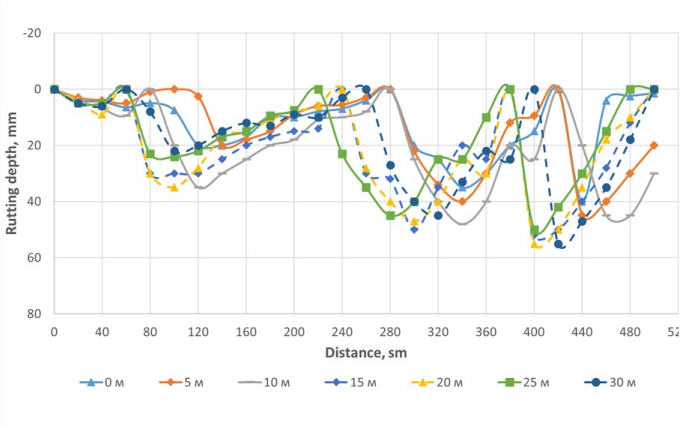


Fig. 3 Graphs of rut depth distribution on the asphalt concrete pavement surface at different distances

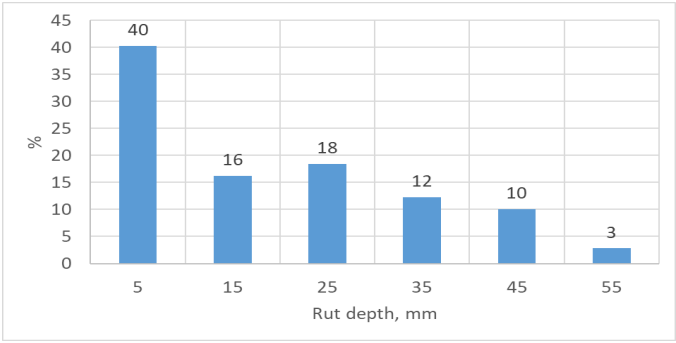


Fig. 4 Histogram of rut depth distribution on the asphalt concrete pavement surface

From the cumulative curve (Fig. 5) we determine the following two important values of the rut depth on the test section:

1) the average weighted value of the rut depth corresponding to 50% of the accumulated frequency, which is equal to 10 mm;

2) the calculated value of the rut depth corresponding to 95% cumulative frequency, which is equal to 42 mm.

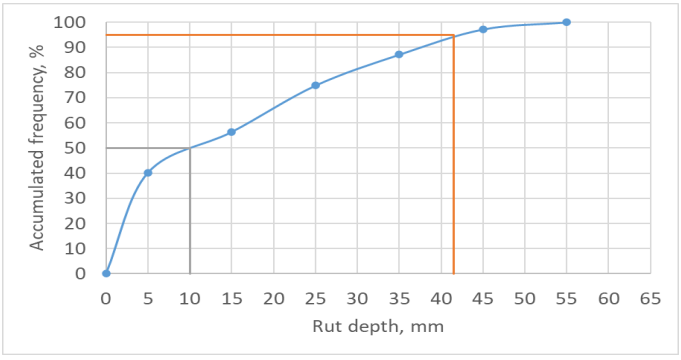


Fig. 5 Cumulative curve of rut depth

4 TRAFFIC INTENSITY AND COMPOSITION

Information on the intensity and composition of the traffic flow on the section is presented in Table 1. In the traffic flow, vehicles causing significant rutting are 17.9% (trucks, buses and tractors).

Table 1. Traffic intensity and composition

Type of transport facilities	Traffic intensity in eastern direction, units/day
Cars	11 829
Light trucks (≤10 tons)	1250
Heavy trucks (>10 tons)	1074
Buses	253
Tractors	20
Motorcycles	107
Total	**14 533**

5 CONCLUSIONS

1. The maximum rut depth on the section is 55 mm, which exceeds the permissible value of 35 mm.

2. In the east direction, the traffic intensity of vehicles causing rutting is 2597 units/day, which is excessively higher.

3. The road pavement structure, which includes two 12 cm thick asphalt concrete layers, is weak, i.e. does not correspond to the actual traffic intensity and composition.

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Fast-moving seismically induced hazardous geological phenomena in South-Eastern Kazakhstan as a factor in assessing natural hazards and ways of their systematic mitigation

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Abstract: in Kazakhstan, in the areas of development of mining structures, transient dangerous geological phenomena of a wide range, which are genetically caused by seismic events, are widely developed. They were studied by the authors in all South-Eastern regions of Kazakhstan and are represented by such dangerous natural disasters as earthquakes, seismic dislocations, floods, seismic ruptures, seismic landslides, landslides, mudflows, glacier movements and the emergence of high-mountain tectonic lakes, as well as unusual atmospheric and many other phenomena that are very significant in scale. The work provides their genetic classification, development conditions and systemic approaches for their mitigation or elimination.

The main natural factors influencing the development of hazardous geological phenomena include: physical and geographical (relief, hydrography, climate, soil and vegetation cover), geological and tectonic, seismic, geomorphological, hydrogeological and engineering and geological conditions.

Tectonic structures, along with climate, are determining factors in the development of the main relief forms and directly related exogenous processes. The pattern is that tectonic (endogenous) processes and structures organize the composition, spatial distribution, time course and intensity of exogenous processes in a certain way. The main effect of endogenous (tectonic-magmatic) processes is the deformation of the earth's surface and the creation of uplifts and depressions of various ranks, distributed in a certain way in space and connected to each other.

Tectonic movements, which intensified at the end of the Neogene and Quaternary, formed a mountainous country, dissected into systems of uplifts and depressions. The presence of a complex branched system of faults in the Almaty region largely determined the distribution of seismicity in our time - most earthquakes are genetically related to movements in fault zones.

These structures, being deeply permeable zones of the earth's crust, create conditions for active circulation of underground waters and gases in them and contribute to the formation of linear weathering zones that go to great depths.

In the history of the Tien Shan, Zhetysu Alatau, Tarbagatai and Altai-Sayan mountain regions over the past 150 years, catastrophic earthquakes have repeatedly occurred, which led to a sharp increase in seismically induced dangerous geological phenomena: seismic dislocations, seismic collapses, landslides, rock slides, mudflows, large snow avalanches and others.

The most seismic zones are the largest marginal faults that border the Ile-Alatau Mountains from the south and north, the zones of the Chilik-Kemin, Main and North Dzungarian deep faults, where the most destructive earthquakes known in the past in the Northern Tien Shan were localized: Vernenskoye (1887), Chilik (1989), Kemin (1911) and many others.

Noting the widespread development of seismically induced OGNA in mountainous regions, it can be noted that their distribution and formation are subject to the paragenetic principle, i.e., the development of one process leads to the formation or activation of another or other processes.

Large seismic collapses and landslides, avalanches in mountainous and high-mountainous regions lead to the formation of large tectonic lakes, which, in the event of a breakthrough, can cause catastrophic mudflows (Big Almaty Lake, Issyk, Akkol in Ile Alatau, ChonUryukty, Kolsay (2 lakes) and Kaindy in Kungey Alatau, two Zhasykol lakes in the Lepsy River basin in Zhetysu Alatau, etc.).

Active seismogenic faults play a key role in controlling the seismicity of the study area, its susceptibility to earthquakes of a certain strength. Seismicity is a constant factor in the

development of dangerous geological phenomena. The described territory is located in the zone of influence of numerous seismogenic zones, which characterize the seismicity of the territory.

The scale of seismic hazard in Kazakhstan can be judged by the seismic zoning map included in the Construction Norms of the Republic of Kazakhstan: "Construction in Seismic Regions" (SNiP RK 2.03-30-2006). According to this map, about 25% of the republic's territory is classified as a seismic hazard zone, where earthquakes with an intensity of 7 or more points are possible.

A significant factor in assessing the seismic hazard of mountainous areas may be the paleoseismic dislocations that are ubiquitous in Dzungaria and Tien Shan, which serve as an indicator of the seismotectonic activity of the region in the Holocene era and evidence that similar seismic events are possible here at the present time and in the near future.

It should be noted that paleoseismodislocations contain information about earthquakes for several thousand years. Therefore, they are more reliable sources of information about earthquakes of rare recurrence. This pattern should be taken into account when assessing the maximum intensity of possible earthquakes in the mapped areas.

Results of demercurisation works in Central Kazakhstan and monitoring of ecological state of the environment

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Keywords: Central Kazakhstan, mercury pollution, natural-technogenic disasters, industrial plants pollutants, monitoring, purification, bottom sediments, groundwater.

After the collapse of the Soviet Union to Kazakhstan were inherited the sources of technogenic mercury pollution that located within two large industrial centers. The mercury pollution in Pavlodar was associated with the production activities between 1975-1993 years of the chemical plant Production Association (PA) "Khimprom" which included chlorine-alkaline production by the electrolysis method with a mercury cathode with a capacity of 100,000 tons of chlorine per year [1]. The activity of acetaldehyde production of the "Carbide" JSC plant was a source of extensive mercury pollution within 1950-1993 years in the area of Temirtau city [2,3]. The main environmental risks in the industrial zone of Pavlodar city were due to the emissions of mercury in to the atmosphere from mercury-contaminated dilapidated buildings and topsoil, as well as the possibility of the spread of mercury-contaminated groundwater to the floodplain of the "Irtysh" river in 5 km west from the plant. The risks to health for continue working staff and people living near the plant were contaminated by mercury vapor and mercury poisoned fish that fishermen were caught from the storage of sewage. The groundwater pollution could also reach the water wells of a large settlement located 4 km from a chemical plant between the plant and the "Irtysh" river. The long-term releases of mercury to the environment of Temirtau city were carried out during accidents, as well as due to imperfection of technologies in the thermal regeneration of mercury from sludge, discharge of mercury-containing wastewater into the "Nura" river and the accumulation of mercury wastes in the enterprise and beyond. As a result the soil was polluted on industrial site and around, the riverbed, banks and floodplains of the river for about 30 km downstream from the city to the "Intumak" reservoir, in the bottom sediments of which were also found traces of mercury pollution.

Despite to the closure of the hazardous industries, environmental pollution grew and continued to have a negative impact on public health, increasing the possibility of mercury emission by groundwater into the "Irtysh" and "Nura" rivers, which could lead to an environmental disaster. Taking into account the negative effects of mercury pollution by the Government of the Republic of Kazakhstan were carried out long-term research of technogenic pollutions by the scientists of the country with their foreign colleagues in a frame fo the grants such organizations as ISTC, INTAS-Kazakhstan, INTAS, INCO-Copernicus. Then the complex of demercurization activities was implemented.

In Pavlodar first phase included the removal and disposal of all process equipment chlorine-alkaline production, the manual collection of metallic mercury, disassembly mercury contaminated industrial buildings, partial removal of the surface layer of heavily soils mercury isolation from the atmosphere and groundwater main underground elemental mercury sources and mercury waste, construction of a repository for mercury-containing building elements and equipment parts. In the second phase of the work four underground sources with heavily contaminated of mercury were isolated around the perimeter from underground water (There

were areas: under the electrolysis case, a local treatment plant for the treatment of mercury-containing wastewater and the wastewater pumping station, as well as at the storage site of mercury wastes and sludge) using the antifiltration barrier type - "wall in the ground". The concrete bases left over from buildings of chlorine-alkaline production were isolated from the atmosphere impact by laying and compacting clay screens, and at the site of the storage of mercury waste and sludge by installing a multi-layer screen (ash, gravel, fertile layer, turf). The repository for mercury-containing building structures and equipment parts from chlorine alkaline production buildings was located on 50 meters from the electrolysis manufactory and consisted of a pit no more than 3 meters deep with a 0.5 m thick layer of compacted clay and filled with a variety of materials filled with a solution of soil concrete.

As a result of the implemented work the carbide plant was completely dismantled in Temirtau city. The construction waste was transported to a hazardous waste landfill. The contaminated soil of the industrial area was dug up to two meters in depth and also transported to the landfill. All metal mercury was mothballed in concrete and plastic containers, and the sewage ponds of treatment facilities were isolated. Contaminated bottom sediment from sewage treatment plants, "Zhaur" swamps and old ash dumps at FPS were also cleaned up. The giant landfill was covered with soil.

Before, during and after completion of work at two sites, in accordance with the projects, monitoring was carried out for the mercury content in the surface atmosphere, groundwater, surface soil layers in the contaminated area around industrial sites, as well as in bottom sediments, water and river fish.

The list of literature:

1. *Mercury pollution in Pavlodar.* http://Hg-Pavlodar.narod.ru
2. *Concept, conditions and problems of implementation in Kazakhstan of the Project "Purification of the "Nura" river from mercury" / Abstracts of the 9th international conference on the mercury problem. China, 2009 y. (Kamberov I., , Schwaiger F., Nussbaumer M.).*
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Day2

AsRTC10 «Urban Geo-informatics»

Regional modeling for flood simulation using MIKE SHE

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ABSTRACT

This study aims to identify and analyze flood-prone areas in Astana, Kazakhstan through the application of integrated hydrological modeling using MIKE+ and MIKE SHE software. Astana's geographical characteristics—including its location on the Ishim River floodplain, extreme climate conditions, and rapid urban expansion—create complex flooding scenarios. The integration of diverse datasets (topographical, meteorological, and soil properties) through advanced hydrodynamic modeling has not been applied to create actionable flood susceptibility maps for Astana, leaving a critical gap in the city's urban planning. This study will improve an existing flood susceptibility model by incorporating unsaturated soil mechanics, which implies realistic moisture distribution in the soil. The deliverable of the study will be a flood risk map for Astana region, Kazakhstan created using MIKE SHE and MIKE+ software.

Keywords: flood-prone areas, flood risk assessment, hydrodynamic modeling, MIKE+, MIKE SHE

1 INTRODUCTION

Despite the technological advancements of civilization, floods remain devastating for humanity. Every year, floods cause destructive consequences in Kazakhstan exacerbated by climate change. Climate shifts have altered precipitation patterns resulting in more intense and frequent rainfall. The rising temperature has triggered earlier and rapid snowmelt. The devastating nature of floods create a challenge to flood prevention approaches. Knowledge of climatic conditions and topographical specification of the area creates an opportunity for managing future flood events.

In order to manage these floods, i.e., predict time, place, and magnitude, engineers are using prediction models. However, these models rely on oversimplified assumptions about water distribution in the soil or simply ignore unsaturated soil properties and thus provide inaccurate results. For example, two studies performed research of flooding in the Yesil River using HEC-RAS software (Ongdas et al., 2020a; Akiyanova et al., 2023b). These studies provided insight into how flood is a snowmelt-driven event together with topographical and infrastructural aspects as main factors. Further, these snowmelt waters contribute to the Yesil River flow increasing inundation risks in nearby local communities and in Astana city.

2 METHODOLOGY

The methodology consists of five main parts: data collection, hydrodynamic model development, flood simulation, flood susceptibility assessment based on risk configurations, and validation. Meteorological datasets including precipitation intensity, duration, and temporal distribution were obtained from KazHydroMet for 2000-2020 period. Digital Elevation Models (DEMs) with resolutions of 30 meters were processed to extract elevation parameters. For soil infiltration parameterization properties such as soil moisture content, suction, and soil classification were derived from SoilGrids250m database and then validated through experimental work.

The MIKE SHE hydrodynamic modelling was utilized to develop spatially distributed flood simulation models for Astana region, Kazakhstan. MIKE+ will be configured to represent the hydraulic behavior of Yesil river networks. The calibrated river model then was

integrated into the flood model for overland flow simulation. Integrated MIKE SHE model consisted of soil property characterization, land-use classification, 30m resolution DEM, spatially distributed meteorological inputs. The coupling between MIKE+ and MIKE SHE allowed us to develop multiple simulation scenarios including historical events. For each scenario, key flood parameters (maximum depths, flow rate, and duration) were extracted and combined using a weighted multi-criteria approach to develop a composite flood susceptibility index as low, moderate, high, and very high. The validation of the model will be performed through historical flood records in the region.

3 EXPECTED RESULTS AND DISCUSSION

The coupling MIKE+/MIKE SHE model approach is expected to reveal insights into flood susceptibility patterns within the study area. Analysis of soil hydraulic properties is expected to discover spatial heterogeneity across the floodplain. The explicit representation of unsaturated soil mechanics is predicted to capture critical threshold in flood occurrence. The soil near the ground surface is considered to be unsaturated and thus have reduced hydraulic conductivity. Results are expected to demonstrate how the reduction in unsaturated hydraulic conductivity during rainfall creates a critical limit where infiltration capacity rapidly decreases, triggering surface runoff and subsequent flooding. These soil-moisture relations are expected to substantially influence flood timing and magnitude. The inclusion of unsaturated soil mechanics will increase flood prediction accuracy. The susceptible areas with “High” or “Very high” categories are expected to correspond primarily to low-lying areas adjacent to the river but also include areas with impervious surfaces such as pavement and asphalt.

Sensitivity analysis is expected to identify soil hydraulic conductivity as the most influential parameter affecting flood susceptibility classification. Uncertainty analysis will likely reveal greater prediction confidence in near river floodplains compared to areas prone to rain-caused floods, demonstrating the importance of integrated modeling approach in capturing different flood generation mechanics.

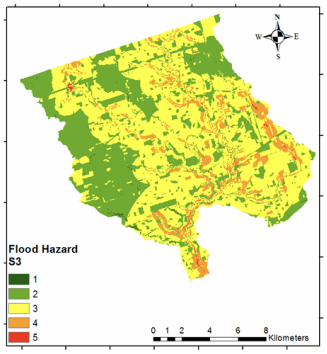


Fig. 1. Flood hazard map example (Rincon et al., 2018).

4 CONCLUSIONS

This integrated modeling framework enables the identification of areas susceptible to flooding by considering multiple flooding mechanisms, including river overflow, groundwater, and surface water accumulation. The model will be calibrated using historical hydrological data and validated against observed flood events. The study's outcomes are expected to provide insights for flood risk management and urban planning in Kazakhstan, contributing to flood mitigation strategies and improved water resources management practices.

ACKNOWLEDGEMENTS

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Debris flow modelling using FLO-2D

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ABSTRACT

Nowadays one of the most destructive and dangerous natural phenomena is considered as debris flow, often resulting in severe material damage and loss of life. By its nature, they occur suddenly, with localized, short-lived events in mountainous regions, which frequently experience intense rainfall and snowmelt. Due to its geographical location and specific climatic features, Kazakhstan are also highly prone to the hazardous effect of debris flow. Despite this, there is now limited literature providing a clear idea, following with logical sequence, and standardized procedure for debris flow modeling specifically using FLO-2D, a two-dimensional hydraulic modeling software. Therefore, this study focuses on spatial modeling of debris flow hazards using FLO-2D, and investigates the suitability in utilizing the software for debris flow analysis.

Keywords: debris flow, hazard assessment, FLO-2D, disaster resilience, hydrodynamic modeling

1 INTRODUCTION

Nowadays debris flows are considered as a global concern due to its destructive effect to the infrastructure and human life. The areas that highly prone to the effect of debris flow are mountainous regions as they frequently experience intense rainfall and snowmelt. Kazakhstan's geographical and climatic features also make it prone to destructive debris flows. Since 1841, over 800 events have been recorded, including two major disasters in the Kishi Almaty River valley (1921 and 1973) that left more than 500 people dead or missing (Medeu, 2011). Given such hazards, effective debris flow management is crucial.

The nature of debris flow is important to investigate due to its enormous hazards to the system. Therefore, researchers from around the world have investigated several case studies related to debris flow with a use of FLO-2D. A case study was conducted in Dashiling gully, Beijing, where debris flow event happened (Jia et al., 2024). The researchers demonstrated the magnitude of risk from the debris flow in their area, evaluated the conditions of the presence of retaining walls for debris flow mitigation measurements. It was concluded that the retention wall could be considered a competent mitigation measurement. Therefore, this study evaluates the suitability of the FLO-2D hydraulic model for debris flow hazard assessment in Kazakhstan to produce validated hazard maps for disaster risk reduction.

2 STUDY AREA AND DATA

The areas for simulation are chosen based on their exposure to debris flow risk, considering factors such as slope gradient, soil type, precipitation patterns, and historical debris flow events. The history events will be first checked to select the event area. According to Medeu (2011), on July 6, 2006, a first-category debris flow occurred in the Kumbelsu River basin, originating from the Mynzhylky River gullies, Kazakhstan. The event lasted from 19:30 to 22:00 with multiple surge waves, reaching peak discharges up to 800 m^3/s. Approximately 1 million m^3 of debris was deposited. Heavy rainfall (up to 61.5 mm/day) triggered the flow. The disaster disrupted access to nearby settlements and facilities, requiring evacuation of over 50 people by Kazselezashchita and EMERCOM forces.

Before modeling FLO-2D model, terrain data is downloaded from the online platform for GIS-ready maps and spatial datasets (https://data.nextgis.com/). The hydrological data is extracted from the "Kazhydromet", the National Hydrometeorological Service of Kazakhstan. The other relevant data is taken from the manual of FLO-2D.

3 METHODOLOGY

The study tries to simulate debris flow event, occurred in Kumbelsu River basin. The software was developed by O'Brien in 1988, and it uses Non-Newtonian model and the central finite difference numerical method (Jia et al., 2024). It also uses some governing equations such as continuity equation, equation of motion and rheological equations.

3.1 Governing equations in FLO-2D

The governing equations are used to simulate the nature and behavior of debris flow. The parameter that is responsible for characterization of debris flow is rheological parameters (O'Brien, 2004). The parameters α_1 and β_1 are responsible for the viscosity coefficient of debris flow, η, while α_2 and β_2 are related to the yield stress, τ_y.

$$\eta = \alpha_1 e^{\beta_1 C_v} \quad (1)$$

$$\tau_y = \alpha_2 e^{\beta_2 C_v} \quad (2)$$

Where C_v is volume concentration of debris flow.

3.2 Data collection

In this step, topographical and hydrological data of the study area are gathered using the geographic information systems (GIS). This data includes Digital Terrain Models (DTMs) to represent the terrain and slope of the investigated area, precipitation data and soil properties relevant to debris flow.

3.3 FLO-2D Model Setup

Geographical, hydrological, and geotechnical data will be consolidated into various input data files to configure the FLO-2D model. Some basic model files like TOPO.DAT (elevation and coordinates), MANNINGS_N.DAT (roughness coefficients), FPLAIN.DAT (floodplain characteristics), will need to be created. After setting all data, the study area will be aligned with the grid system. This will allow the model to be as accurate as possible without wasting too much time on computation.

3.4 Calibration and validation

During calibration, the debris flow parameters such as Manning's coefficient, sediment concentration, and yield stress will be adjusted for better simulating the actual events. As discussed in the previous studies, FLO-2D often requires extensive calibration, and therefore, the above mentioned parameters will be properly tuned (Wu et al., 2013).

4 RESULTS AND DISCUSSION

It is expected to have a map of characteristics of debris flow such as flow depth and maximum velocity. Based on the results, hazard map can be constructed, which enhances disaster preparedness. The mudflow in Barnard Creek, USA, is simulated based on additional data such as sediment concentration, viscosity, and the building's location. It is expected to have results similar to Barnard Creek mudflow (Figures 1-3).

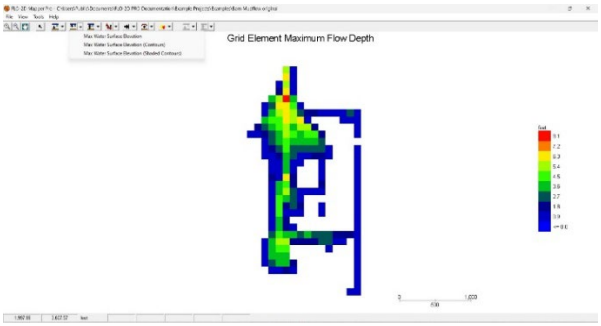


Fig. 1. Maximum flow depth of Barnard Creek mudflow.



Fig. 2. Hazard map of Barnard Creek mudflow.

5 CONCLUSIONS

Generally, the current research tries to fill the gap related to debris flow modeling in Kazakhstan by integrating innovative simulation tools with localized data. Overall, the findings of this study will contribute to enhancing disaster preparedness and resilience in vulnerable regions across Kazakhstan.

ACKNOWLEDGEMENTS

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Simple methods for measuring groundwater levels to estimate damage to low-rise housings due to liquefaction

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ABSTRACT

Important points to note when estimating the planar distribution of groundwater levels using boring data are presented at first. Then, three case studies are introduced, in which, detailed surveys were conducted using simple devices to examine the planar distribution of groundwater levels after earthquakes in order to investigate the impact of groundwater levels on damage to low-rise housing. In cities affected by the 2024 Noto Peninsula Earthquake, studies are being conducted to install underground drainage pipes to lower the groundwater level and counter damage from future earthquakes. Measurements of the planar distribution of groundwater levels are being used to select appropriate locations for installing the drainage pipes.

Keywords: liquefaction, earthquake, groundwater level, low-rise housing

1 INTRODUCTION

In the evaluation of liquefaction-induced damage to low-rise housing, the depth of the groundwater level is important because the damage is greatly affected by the groundwater level and by soil characteristics. A difference of just one meter can affect the extent of damage. In general, when estimating the planar distribution of groundwater levels, previous boring data is collected and used to make the estimate. Additional new boring may be conducted, but in these cases it is important to keep in mind the following:

a) There are too few boring sites to estimate the detailed planar distribution of the groundwater level.
b) Boring should be done with clean water without using bentonite solution until the initial groundwater level is reached, but old borings are not drilled with such care.
c) The groundwater level fluctuates depending on the season of the investigation and the amount of precipitation immediately prior to the investigation.

Given these limitations, it is desirable to improve the accuracy of estimating groundwater levels by using methods such as: i) measuring water levels at many locations using simple survey devices on the same day, and ii) continuously measuring groundwater levels at representative locations for one year to account for seasonal fluctuations in groundwater levels and the effects of rainfall.

This paper gives three examples where investigations were conducted using simple devices to examine the distribution of groundwater levels throughout an area.

2 THREE EXAMPLES OF MEASUREMENT BY SIMPLE DEVICES

2.1 Abehikona housing complex in Yonago City

Liquefaction occurred in the Abehikona housing complex in Yonago City during the 2000 Tottoriken-seibu Earthquake. There were 169 low-rise houses in the complex, and some of the houses settled and tilted by 15/1000 or more, while others were undamaged. The complex is built on reclaimed land, and it was estimated that there was a uniform sandy soil layer in this land to the depth of about 7 m. In order to investigate the influence of the groundwater level on the damage, the distribution of the groundwater level was investigated in detail (Hashimoto and Yasuda, 2002). 33 locations were selected, holes were dug by a small boring machine to a depth of 3 m , and vinyl pipes were installed in them. About one year after the earthquake, a day with similar rainfall condition to that at the time of the earthquake was selected, and the groundwater levels were simultaneously measured. As a result, the groundwater level of the entire housing complex was found to vary from GL-0.8 m to GL-2.1 m. It was revealed that if the groundwater level was shallower than GL-1.6 m, the houses tilted by 15/1000 or more.

2.2 Urayasu City

The 2011 Tohoku Earthquake caused liquefaction in many places in the Tohoku and Kanto regions. The most serious damage occurred in Urayasu City. About 85 % of the Urayasu City area liquefied and low-rise houses,

roads and buried pipes were seriously damaged. After the earthquake, the authors measured the groundwater level at 35 residential gardens in two areas of Urayasu City using a newly created simple device with an electric drill, a drilling pipe and a measuring pipe (Ishikawa et al., 2014), as shown in Photo 1. It is a miniature boring machine. Fig. 1 shows the estimated groundwater level distribution in Irifune area. This is a small flat area of 300 m x 600 m, but the groundwater level varied from GL-0.8 m to GL-1.8 m, depending on the location. Fig. 2 shows the relationship between groundwater levels and damage to low-rise houses in the two areas surveyed. The results showed that the shallower the groundwater level, the more severe the damage.

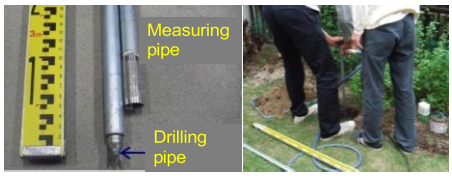


Photo. 1. Newly created simple device to measure groundwater level.

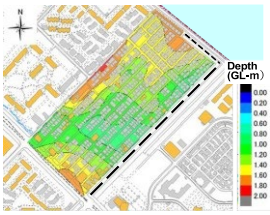


Fig. 1. Groundwater level distribution in Irifune area.

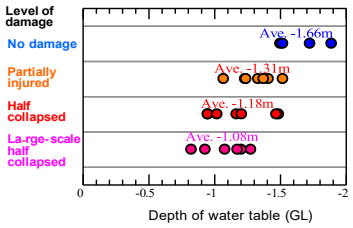


Fig.2. Relationship between groundwater levels and damage to low-rise housing.

2.3 Kanazawa City

The 2024 Noto Peninsula Earthquake also caused widespread liquefaction, causing many low-rise houses to settle and tilt. Furthermore, lateral flow with a maximum horizontal displacement of 3 m occurred on the gently sloping ground on the inland edge of sand dunes, exacerbating the damage to housing. In 10 cities affected by liquefaction, studies have begun on how to recover from the damage of that earthquake and how to prepare for future earthquakes. In the six cities that experienced liquefaction in the 2011 Tohoku Earthquake, underground drainage pipes were installed in the affected areas, and measures were taken to lower the groundwater level to about GL-3 m (Yasuda and Hashimoto, 2016). In order to implement similar measures in cities affected by the Noto Peninsula Earthquake, investigations have been launched to examine in detail the horizontal distribution of groundwater levels. Before these investigations began, the authors attempted to drill holes in the sand dune where lateral flow had occurred using a Screw Weight Sounding device (SWS), as shown in Photo 2, and measure the groundwater level in the holes. As a result, it was confirmed that the groundwater level could be accurately measured in less than one hour.

Then, in the Awagasaki area of Kanazawa City, 10 months after the earthquake, groundwater levels were simultaneously measured using SWS holes drilled at 18 locations, and the distribution of groundwater levels was estimated as shown in Fig. 3. This area has a gentle slope from the top to the bottom of the figure. The groundwater level in the liquefied areas was between GL-1m to GL-3 m, and consideration is currently being given to the installation of an underground drainage pipe network to lower this to a depth of around 3 m. Similar studies are also planned for other cities.



Photo 2. Screw Weight Sounding device and measuring groundwater level.

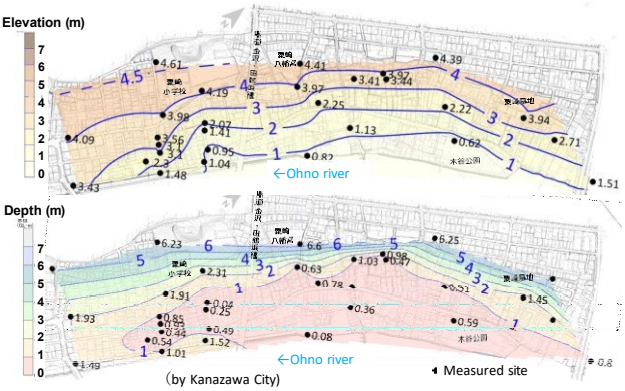


Fig.3. Estimated distributions of the elevation and the depth of groundwater level.

3 CONCLUSIONS

Groundwater level surveys using SWS are economical and can be done in a short time, even in gardens and roads. This method also allows the collection of soil samples to judge the risk of liquefaction, so it is expected to be used in the future.

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On detailed seismic zoning of regions of Kazakhstan using GIS technologies

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ABSTRACT

With the transition of the state building regulations of the Republic of Kazakhstan to the basic principles of Eurocode 8, the work on assessing and mapping the seismic hazard of the territories of Kazakhstan has also moved to a qualitatively new level. In this paper the main results of the research project on detailed seismic zoning of Kazakhstan's regions on a modern methodological basis are presented. The applied methodology corresponds to the main provisions of the Western approach, while including domestic developments. The hazard assessment is based on a probabilistic approach, detailed seismic zoning is presented in accelerations. Modern methods and tools of analysis, as well as the most complete and up-to-date information available for the territories under consideration are used. GIS tools are applied at different stages of the research - in spatial analysis of seismicity, mapping of active faults, modeling of seismic sources, construction of isolines and, in general, visualization of the calculated seismic hazard. The resulting maps are presented.

Keywords: probabilistic seismic hazard assessment, detailed seismic zoning, Kazakhstan regions

1 INTRODUCTION

In Kazakhstan, a three-stage system for seismic hazard assessment has been adopted, which consists of the sequential development of general seismic zoning (GSZ), detailed seismic zoning (DSZ) and seismic microzoning (SMZ) maps. Until recently, Kazakhstan's regulatory documents governing construction in seismic areas were based on a deterministic seismic hazard assessment and seismic zoning. The main parameter describing seismic impacts was macroseismic intensity. With the transition of the state construction standards to the basic principles of Eurocode 8, it was necessary to develop seismic zoning maps that meet these principles. The first such maps, included in the building regulations of Kazakhstan in 2017 and 2021, were the GSZ maps of the territory of Kazakhstan and the SMZ maps of Almaty city. In their development, a probabilistic approach to hazard assessment was applied and seismic effect was expressed not only in macroseismic intensities, but also in engineering parameters - peak ground accelerations. Since 2021, DSZ of Kazakhstan's regions has been underway as part of an ongoing project "Seismic hazard assessment of regions and cities of Kazakhstan on a modern scientific and methodological basis". The project covers a number of tasks, one of which is the probabilistic seismic hazard assessment (PSHA) and DSZ of regions in accelerations presented in the paper. In its implementation, the experience obtained in the international projects, which included PSHA of the territory of Central Asia, is actively used. DSZ is performed to determine possible seismic impacts on the territory of specific regions at a scale of 1:1,000,000. Compared to the first PSHA maps of 2017 and 2021, the databases have been radically updated and not only areal but also linear models of seismic sources have been used. GIS programs (QGIS, ArcGIS) are actively used in the analysis and presentation of a large volume of diverse data and modeling. The target DSZ maps themselves are created in GIS format.

2 METHODS AND DATA

The main PSHA elements include the compilation of a model of seismic sources, their magnitude-frequency characterization, calculation of ground motion at the site from all possible earthquakes from these sources and estimation of the probability that these ground motion will be exceeded for a given period of time. The studies are based on the geological-geophysical and seismotectonic information bases created in the project, include the study of the parameters of the seismic regime and seismotectonic deformation of the earth's crust, the creation of a bank of ground motion parameters, the definition of a system of ground motion models (GMMs), modeling and parameterization of areal and linear seismic sources, probabilistic analysis of seismic hazard, and the development of DSZ maps. The probabilistic analysis was performed according to the Cornell methodology

(Cornell, 1968) using the methodological formalism of Field et al. (Field et al., 2003), implemented in the OpenQuake Engine seismic hazard and risk calculation software (Pagani et al., 2014). In the seismic hazard analysis, the seismic source logic tree included two hybrid models – a combination of area sources with two active fault models. The GMM logic tree, depending on the seismotectonic conditions, included sets of 5 GMMs for crustal earthquakes in tectonically active regions, tested on regional data, and 3 GMMs each for earthquakes in stable areas and subduction zones. The analysis was performed for the probability of exceeding the calculated values of 10% and 2% over 50 years (return periods of 475 and 2475 years, respectively). Seismic impacts on the territory of the regions were calculated in average values of the geometric mean peak ground accelerations (PGA) for the case of rocky soils (shear wave velocity Vs30≥800 m/s) on a grid with an interval of 5 km. For each calculation point, the impact from seismic sources located within a radius of 300 km was taken into account.

QGIS tools were used at almost all stages of the study. Google terrain models based on elevation data, visualization and analysis of earthquake catalogs and active fault databases, creation of polygonal sources based on analysis of data from layers with different content, work with attributive layer databases when compiling models of active faults and zones, export of seismic source geometry to create input files for the probabilistic hazard analysis program, etc. were used in modeling seismic sources. Visualization and spatial analysis of earthquakes and their main parameters (area distribution, energy characteristics, depth, etc.) were also used in the preparation of a complete earthquake catalog. When parameterizing the area seismic sources, the earthquake catalogs within zone boundaries were extracted and analyzed in QGIS. After calculation of ground motion values at the points of a given grid, a "Contour plugin" allows to obtain the map of isolines with required parameters. The final DSZ maps are assembled in ArcGIS - the obtained isolines and layers are overlaid on the digital topo base and relief model.

3 RESULTS

As part of the first phase of the project (2021-2023), sets of probabilistic DSZ maps of the territories of East Kazakhstan, Abay, Almaty, Zhetysu and Zhambyl regions were developed. Hazard curves and uniform hazard spectra were obtained for the main cities. Within the framework of the second phase (2024-2026), studies on DSZ of the Turkestan and Mangystau regions are being conducted and preliminary results are obtained. Fig.1 shows the developed DSZ maps for the territory of Zhetysu region as an example.

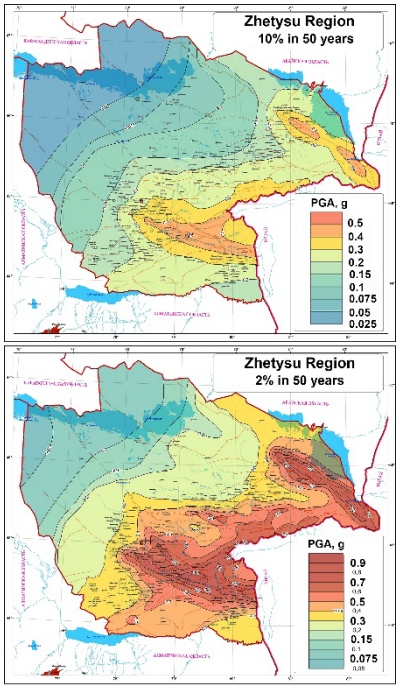


Fig. 1. Schematic DSZ maps of the Zhetysu region. Probability of exceedance - 10% (up) and 2% (down) in 50 years.

4 CONCLUSIONS

The development of modern PSHA in Kazakhstan is dictated by the need to update seismic design standards and harmonize them with European building standards. Modern methods and tools of analysis, as well as the most complete and up-to-date information were used in the development of the maps. Unlike the first probabilistic maps in Kazakhstan, the DSZ of regions includes modeling of not only area but also linear seismic sources. The results agree with the 2017 Kazakhstan's GSZ maps. By taking into account both area and linear sources and revision of the catalog, more pronounced, high-risk zones in the area of active faults were obtained. The development of modern seismic zoning maps is difficult without the use of GIS technologies.

ACKNOWLEDGEMENTS

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Application of GIS to assess permafrost degradation in southern Taimyr

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ABSTRACT

The Norilsk Industrial Region, a critical Arctic infrastructure hub contributing 1% of Russia's industrial output, faces escalating risks from climate-driven permafrost degradation. This study combines GIS-based landscape typology, 105 boreholes data, and thermal modeling to project permafrost dynamics across 8,000 km². A four-stage methodology identified 76 distinct landscape units, integrating geological, climatic, and permafrost data to simulate ground temperature evolution. Projections for 2050 using the SSP5-8.5 scenario (0.05°C/year warming) indicate complete disappearance of zones with temperatures below −3°C (18% of the area in 2024), while discontinuous permafrost (+1 to −0.5°C) expands from 20% to 33%. Seasonal thawing will initiate across 20% of the region, with active layer deepening by 2.2–8 m, threatening infrastructure stability. Despite incomplete historical data, the methodology enables actionable insights for infrastructure adaptation. The study advances GIS-based tools for climate resilience planning in industrial Arctic zones.

Keywords: permafrost degradation, Arctic, GIS modeling, thermal regime; thermal modeling, climate change, Norilsk

1 INTRODUCTION

Over the past two decades, the Arctic has experienced the most rapid air temperature increase globally (Rantanen, 2022; Wang, 2022). Climate data analysis for the Northern Hemisphere reveals that Arctic regions warmed 70% faster than the global land average between 2011 and 2020 compared to the 1980–2010 baseline (anomalies of 1.12°C vs. 0.66°C). Localized warming rates as high as ∼1.7°C (1997–2020) recorded over the Taimyr Peninsula (Watts, 2025). Rising air temperatures have elevated permafrost temperatures (Biskaborn, 2019), threatening infrastructure stability (Hjort, 2022; Melnikov, 2022). In Norilsk, ground temperatures at a 10 m depth increased by 4.3°C over 65 years (1959–2024), with significant thermal fluctuations observed down to 90 m (Kotov, 2025). Consequently, integrating climate-warming trends into infrastructure design, maintenance, and reconstruction is essential for Arctic development.

This study focuses on the Norilsk Industrial District, a major industrial hub on the Taimyr Peninsula and a critical contributor to Russia's economy (38.8% of Krasnoyarsk Krai's and over 1% of Russia's industrial output). Permafrost degradation forecasts are foundational for infrastructure planning, Arctic master plans, and natural risk assessments. Digital maps of mean annual air temperature, developed in QGIS, underpin this analysis.

2 METHODS

A four-stage methodology implemented to project permafrost degradation (mean annual ground temperature at 10 m depth) across the Norilsk-Dudinka region (8,000 km²):

2.1. Landscape Zoning. A typological landscape map generated in QGIS using geological, geomorphological, soil, topographic, and vegetation maps.

Landscapes are multi-component natural complexes. The main components of landscapes include:

- atmosphere;
- vegetation cover;
- soils (upper layer <50 cm);
- ground;
- surface and groundwater.

Landscape zoning is a consistent hierarchical division of the territory into sections: from larger ones, internally homogeneous by the main feature, to smaller ones, homogeneous by other features - the second, third and subsequent orders. That is, a step-by-step division of the territory into micro-districts by factors is carried out. Within one stage of division, only one feature is used, it must be indicated and justified. The feature of division at the next stage may be one for all already identified larger areas, or it may be different for each area, reflecting the specifics and playing a significant role in the formation of heat exchange.

The region was classified into 76 distinct landscapes.

2.2. Initial data. Data from 11 permafrost monitoring test sites and 80 historical boreholes (1975–2000) integrated (fig.1).

There are following data:

- geological cross-section;
- properties of frozen and thawing soils;
- one-time measurement of soil temperature for archive wells and monitoring of soil temperature from December 2023 to the present (with daily measurement and storage of data in the information system)

Unmonitored zones were analyzed using landscape analogs method.

2.3. 1D Thermal Modeling. Numerical models simulated heat transfer in vertical soil columns. Each layer was assigned site-specific thermal properties, surface boundary conditions (climate, vegetation, snow cover), and basal heat flux. Historical borehole data calibrated to 2024 using meteorological station records and geologic survey. Results were visualized as QGIS-based maps of contemporary ground temperatures.

2.4. 2050 Projections. To forecast future climate changes, data from the Russian Climate Center used in the form of high-resolution global climate models that took part in the 6th phase of the international project for comparing combined models CMIP6. The most critical scenario, in which the greatest change in air temperature occurs - SSP5-8.5, was adopted as the main scenario in the calculations (regional air temperature trend: +0.05°C/yr).

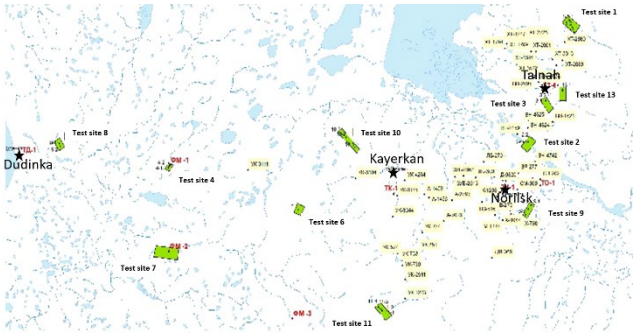


Fig. 1. Initial data (permafrost monitoring test sites and 80 historical boreholes).

3 RESULTS

76 unique landscapes identified after landscape zoning of the territory, which are characterized by different natural conditions. Using actual data on the ground temperature regime and mathematical modeling data, maps of average annual ground temperatures for 2024 and 2050 compiled in the QGIS program.

By 2050, average soil temperatures will increase. Areas with low temperatures (from - 3 to - 4 °C), which occupy 18% of the territory in 2024, will disappear completely. Areas with discontinuous permafrost distribution (temperatures from +1 to - 0.5 °C) will increase from 20% to 33% of the total area by 2050. According to the calculation results, perennial thawing will begin on 20% of the territory (temperatures from +1 to - 0.5°C) in 2050, the permafrost table for different landscapes will be from 2.2 to 8 meters (fig.2).

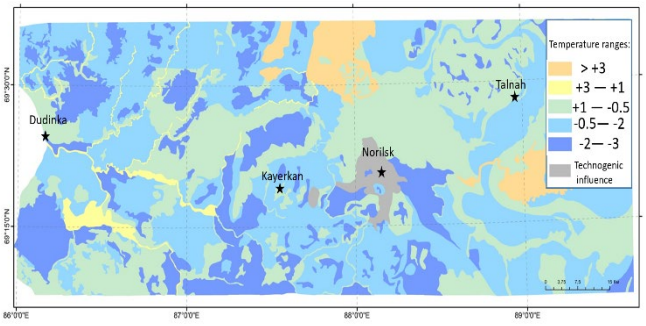


Fig. 2. Ground temperature map in 2050.

Digital maps are the basis for assessing current changes in permafrost. QGIS maps highlight spatially heterogeneous degradation patterns and the delayed thermal response of permafrost to climate forcing. The ground temperature map for 2050 is a clear evidence of the adverse impact of current climate change on the temperature regime of soils. The created model takes into account the inertia of the reaction of permafrost strata and reflects the zonality of permafrost response to general trends in climate change

Considering the incomplete coverage of a priori information on the actual temperatures of rocks, the data will be supplemented, which will significantly increase the reliability of forecasts. But the developed methodology and model for predicting degradation will be used as a basis.

4 CONCLUSIONS

For the first time in the northern Krasnoyarsk Territory, such large-scale permafrost studies have been conducted using GIS and modern modeling methods. The project combines actual regional data on geology, climatology, geocryology and landscape analysis in one GIS system, which allows for the most accurate forecasts. The project results will help predict and minimize the risks associated with permafrost degradation, such as the destruction of infrastructure, changes in landscapes and ecosystems in the region.

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DEVELOPMENT OF AN ACCESSIBLE REMOTE METHOD FOR MONITORING LANDSLIDE PROCESSES

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Abstact

Remote methods for studying and monitoring exogenous processes have been used since ancient times. In the 18th century, the determination of the size and spatial position of objects was done through their drawn images in central projection, which were obtained using a camera obscura from elevated places and ships. The next stage in the development of remote methods was the invention of photography, the photographic lens, and the stereoscope. Photographs of the landscape, taken from a bird's-eye view using balloons and kites, received high cartographic recognition. Currently, drones equipped with cameras are effectively used for studying and monitoring the development of landslide processes. However, even this method has its drawbacks. So, that it's virtually impossible to fix a drone with photographic equipment in a stationary position for an extended period at a chosen high near-distance observation point. The authors have proposed a simplified method of photogrammetric surveying for near-distance research of large landslide processes.

The method involves using various flying vehicles instead of aerial photography, periodic photography is carried out from elevated points of seismic-resistant high-rise buildings and structures (television towers, chimneys, high masts, high-voltage power line towers) located outside the landslide-prone area.

These stable elevated structures must be located in the immediate vicinity of the landslide-prone area under investigation. Special photo or film equipment is installed at an accessible elevated point on the stable high-rise structure and is temporarily secured there, operating in a preset automatic mode. It's carried out photo or film recording of specific surface zones of the landslide-prone area or special markers installed at specific surface points of the active landslide or landslide-prone area. Three photo or film cameras can be installed at a stationary point on the stable elevated structure, simultaneously with conducting parallel monitoring studies in the areas of the head, body, and toe of the active landslide or landslide-prone area.

If there are several stable elevated structures in the nearby zones of the active landslide or landslide-prone area, it's possible to use at least three elevated structures located in different adjacent zones outside the active landslide or landslide-prone area to install the equipment.

The observational and recording photo or film equipment can also be mounted on a trolley that can be remotely moved and fixed along a special metal cable stretched between two stable elevated structures located in opposite adjacent stable zones of the active landslide or landslide-prone area.

The method in comparison to aerial photography, is significantly simpler, cheaper, and allows for more accurate information about the dynamics of landslide, erosion, and abrasion processes due to the possibility of conducting multiple surveys from stationary points over different periods. First of all, we achieve the effect of stationary, relatively inexpensive research on the development of the landslide process over a long period, during which periodic photography of the designated area or object on the landslide slope surface is carried out automatically according to a set program. The method can also be implemented using two or more cameras installed

on two or more seismic-resistant high-rise structures located in the vicinity of the large landslide area under study. The equipment installed on the elevated structures can be equipped with automatic systems programmed for emergency response to the rapid development of the landslide process and alerting operators via mobile or satellite communication networks. The development is distinguished by its relative affordability, accuracy, and accessibility.

Keywords: landslide, periodic photography, high-rise buildings, method, landslide process.

AI-Powered Smart Sensors for Enhancing Safety in Mining and Construction

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ABSTRACT

Underground excavation activities, such as mining and tunneling in construction, present significant safety risks due to soil/rock deformation, ground collapse, toxic gas leaks, and equipment malfunctions. This paper explores the transformative potential of AI-powered smart sensors designed to enhance safety in these hazardous environments. The proposed solution integrates advanced sensing technologies with predictive analytics to address the dual challenges of machine and human safety in areas where wireless internet connectivity is limited. The paper focuses on the application of smart sensors equipped with edge AI technology, allowing for real-time data acquisition and local processing in remote, harsh environments without the need for constant internet connectivity. These sensors enable predictive maintenance of mining and construction equipment by detecting early signs of wear and damage, thereby facilitating proactive maintenance and minimizing downtime. Additionally, wearable AI sensors monitor worker health indicators, such as heart rate and fatigue, and provide real-time detection of dangerous gas exposure. Furthermore, an array of interconnected sensors detects ground deformations, helping to predict potential hazards due to mine collapse. The AI-driven system not only ensures equipment reliability but also significantly improves worker safety and operational efficiency in underground environments.
Keywords: AI-powered sensors, mine collapse, predictive maintenance, ground deformation

1 INTRODUCTION

Expanding underground mining and infrastructure development face persistent safety and operational challenges, including rockfalls, gas emissions, equipment failures, and exposure to hazardous conditions. Harsh environments—characterized by high humidity, dust, temperature extremes, and corrosive agents—accelerate the degradation of electronic systems. Connectivity constraints due to remote locations and geological shielding limit centralized data transfer, requiring localized processing. Moreover, underground hazards such as ground deformation and gas concentration fluctuations are highly dynamic.

Conventional safety systems primarily rely on reactive responses. Integrating Artificial Intelligence (AI) into sensor networks offers a shift toward predictive, real-time hazard detection (Davis and Miller, 2018; Chen and Wang, 2019; Smith and Jones, 2021; Johnson and White, 2022). Such systems enhance risk mitigation, equipment reliability, and personnel safety under demanding underground conditions.

2 SYSTEM ARCHITECTURE & METHODS

The proposed system utilizes artificial intelligence to convert raw sensor data into actionable insights, ensuring functionality even in the absence of continuous cloud connectivity. The architecture consists of three key components that work in tandem to enhance safety, operational reliability, and maintenance efficiency in underground environments. The first component, Geonose Base Stations, comprises stationary units strategically deployed across the underground network to establish a robust mesh communication system. These units are equipped with electronic nose (E-nose) technology for detecting multiple hazardous gases such as methane (CH_4), carbon monoxide (CO), and hydrogen sulfide (H_2S). They also incorporate structural and environmental monitoring sensors to detect ground deformation, convergence, vibration, and air quality metrics. Embedded edge AI processing capabilities enable real-time anomaly detection and predictive modeling directly at the source, minimizing the need for high-volume data transmission (Balakrishnan and Gupta, 2020)..

The second component, Geonose Watch, is a wearable AI-based sensor device designed to monitor individual worker health and environmental exposure. The watch tracks vital signs such as heart rate, body temperature, and include localized gas detection sensors for immediate alerts in case of toxic exposure. Integrated accelerometers and gyroscopes facilitate fall detection and real-time localization, enhancing emergency response capability. Communication is handled through Bluetooth Low Energy (BLE) for short-range interaction with nearby base stations and through LoRa technology

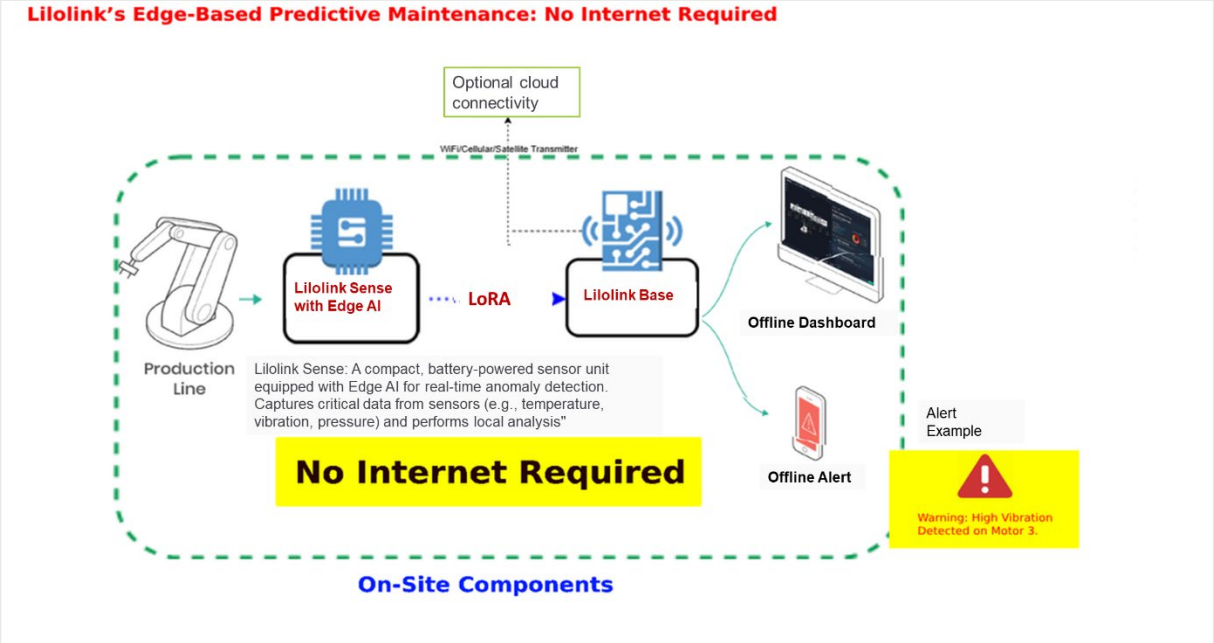


Fig. 1. Design components of the sensor technology

for long-range, low-power communication with surface control rooms, ensuring persistent connectivity across deep underground levels.

The third component addresses equipment reliability through the integration of Lilolink, an AI- and IoT-based predictive maintenance module. This system continuously analyzes operational parameters such as vibration and mechanical behavior of mining and construction equipment to predict component wear and impending failures. The predictive capabilities of Lilolink enable just-in-time maintenance interventions, significantly reducing unplanned downtime and extending machinery lifespan.

3 EXPECTED OUTCOMES AND IMPACT

Deployment of the AI-powered smart sensor system is expected to deliver significant improvements in underground safety and operational efficiency. Real-time monitoring of worker health and environmental conditions will enable prompt alerts and interventions, thereby reducing accident rates and improving response times. The predictive maintenance framework will enhance machinery reliability, optimize maintenance scheduling, and lower operational costs. Additionally, continuous data acquisition and AI-driven analytics will empower supervisors with high-resolution insight into system behavior, supporting data-informed decision-making. Collectively, these advancements contribute to safer, more efficient, and environmentally responsible underground operations.

5 CONCLUSIONS

Integrating artificial intelligence with smart sensor networks represents a transformative advancement in underground mining and infrastructure safety. By enabling localized, real-time monitoring of environmental, structural, and physiological conditions, the system facilitates proactive hazard mitigation and predictive maintenance. The architecture supports scalable deployment in complex subsurface environments and offers a cost-effective solution for improving safety standards, operational resilience, and sustainability across the industry.

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AsRTC19 «Geo-engineering for conservation of heritage monuments and historical sites»

Development of GIS for monitoring of dams in Kazakhstan: processing, analyses, and functionality

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ABSTRACT

Due to the progress in the development of computing system performance and the advancement of the Internet, an increasing number of areas of human activity are being reflected in databases and geographic information systems (GIS). Since water is a vital factor for human existence, numerous databases and GIS systems are related to water consumption, monitoring of water sources, and studying the dynamics of natural and artificial reservoirs.
Today, digitalization of water resources is an important strategic task for monitoring water resources of our country (Kazakhstan). Our team is engaged in developing a GIS system dedicated to monitoring the condition of dams in the Republic of Kazakhstan. The database and web services that provide the system's functionality and interface were developed. The system offers functionality for various types of monitoring, analysis, and statistics collection on dams, as well as working with the dam passport. In the future, the project will be developed and scaled up to expand the analysis instruments and reporting functions, as well as to enhance the use of machine learning. Performed the task of searching for potential dams based on cartographic images using ANN. The section dealing with potential dams is titled “Moderation.” The system has different user types. They perform specific roles that are integral to the application's functionality. The dam monitoring section is a complex system that comprises several monitoring types. Application developed using microservices architecture (REST API)—instruments: GIS-related APIs, Golang programming language, PostgreSQL DBMS, Fiber, Kubernetes, Docker. The project has its server for data processing. The domain ‘kazdams.kz’ is reserved for system operations.
The development of a GIS system for digitalization, automation, and data processing of the dam is a necessary and significant task in our project. The research is funded by the Ministry of Science and Higher Education of the Republic of Kazakhstan, IRN AP19675038.

Keywords: GIS; dams, water resources, monitoring; microservices; AI, Golang, Fiber, REST API.

1 INTRODUCTION

Since water is a vital factor in human existence, numerous databases and GIS systems are available for monitoring water consumption, tracking water sources, and studying the dynamics of both natural and artificial reservoirs.

As an example, let us cite the following open-source projects [1]:

- Digital platform ‘Water Data’: developed by the Federal Agency for Water Resources of Russia (Rosvodresursy) to provide access to information about water bodies, including interactive maps and open data.
- Automated Information System for State Monitoring of Water Objects (AIS GMVO): provides access to information on the state of surface and groundwater, as well as information on water management systems and hydraulic structures.
- GIS ‘Drinking and domestic water supply facilities’: a unified information resource containing

data on water bodies used for drinking and domestic water supply, created to improve the efficiency of their protection and management.

- Kamystybas Lake Monitoring Project using GIS technologies: a study demonstrating the application of remote sensing data and GIS to automate monitoring of water bodies.
- Research and digitisation of the Almaty city dam [2].

In [1], an analysis of the systems for monitoring the condition of HS in the Russian Federation was conducted, and attention was drawn to the lack of a centralized observation system. General ways of solving the problem were also indicated. A detailed review of the works of Russian scientists in this area is given in [3].

Report [4], 2021, draws attention to the need to create a GIS for monitoring the Amu Darya and Syr Darya river basins in Uzbekistan; however, such a system has not yet been developed. As the authors emphasize, Uzbekistan also needs to create and implement a GIS for water resources and hydraulic structures.

Our group is currently developing a geographic information system (GIS) dedicated to monitoring the condition of dams in Kazakhstan. Our country is primarily located in a region prone to risky agriculture, and our territory is susceptible to droughts during the hot season and floods and inundations during the snowmelt season (links to official websites). The study of water bodies is considered by local authorities to be one of the most critical activities. Currently, a centralized system for digitizing water resources in our country is being developed [5].

In the Global DamTracker project, described in [6] for 2023, a database of dams existing in the world was compiled, including descriptions and coordinates of more than 35,000 objects distributed worldwide. The authors filled in the information from open sources, so not all information was included in the database. For example, for our country, this database mentions only the largest 31 GTS.

In this paper, we propose the structure of a geoinformation system that collects information on dams, with the most essential part of the system being the collection, systematization, and storage of dam survey data. Members of our group have previously conducted geophysical surveys of several dams [7], as well as BLPA overflights and processed the overflight data. These data were also structured and included in the geoinformation system's database. The objective of our project is to develop and apply data processing methods that preserve the monitoring history, with a focus on data analysis and storage. In addition, we envisage storing in the database data on the dynamics of the mirror area of neighbouring objects, derived from publicly available space images.

GIS technology is naturally integrated with remote sensing techniques, which allowed the authors of [8] to develop GIS-based criteria for dam condition assessment from 16 indices, and the authors of [9] to propose a combination of GIS and remote sensing technology to study dam condition in dynamics for one dam in the Republic of Uzbekistan. In [10] a series of maps using GIS technology was developed for some water bodies in the Republic of Uzbekistan.

Note that the inclusion of UAV overflight data in the database enables the refinement of the digital surface model (DSM) and the storage of these data. The DSM data will help refine the results of existing geophysical soundings and for modelling different scenarios of leakage and overflows at dams.

Currently, water resource management is one of the key tasks in countries with insufficient water supplies, including Kazakhstan. Monitoring of water bodies, dams, and weirs is critical. In the territory of the former Soviet Union, such systems are present only in isolated cases and mainly at hydraulic structures (HS) owned by large companies capable of maintaining staff and monitoring these systems [11].

2 DATA AND INTERFACE

The description of data and interface is below.

2.1 Programme Purpose

- Monitoring and Assessment: Continuous monitoring and assessment of dams and reservoirs using advanced technology and data analysis techniques.
- Prevention and Risk Management: Develop and implement preventive measures and risk management strategies to prevent emergencies.
- Education and Awareness: Conduct educational activities and campaigns to raise awareness of hazards and safety rules in the dam and reservoir field.
- Cooperation and Partnership: Establish close cooperation with government agencies, scientific institutions, local communities and international organisations to implement joint projects and exchange experiences.

2.2 Brief description of the project database

The project database contains 50 tables. The main table is the DAMS table. It contains basic information about the dam. It is linked to the DAMS_COORDINATES table, which contains coordinates in longitude and latitude format. Figure 1 shows part of the diagram showing the DAMS, DAMS_COORDINATES and DAMS_PURPOSES tables.

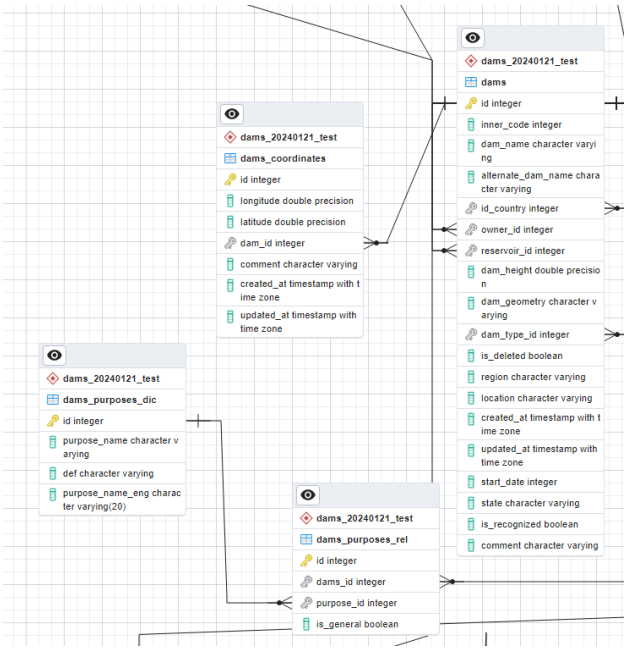


Fig. 1. DAMS, DAMS_COORDINATES and DAMS_PURPOSES relations

The table storing data from the dam passport, the main document of the structure, is quite interesting.

There are also tables reflecting reference information, utility tables that are not directly related to the subject area but are necessary for system maintenance, e.g. data on registered users, storage of logins, passwords and authorisation tokens.

Figure 2 shows the structure of the part of the database that corresponds to the UAV survey of the dam. The data are stored as links to different types of documents.

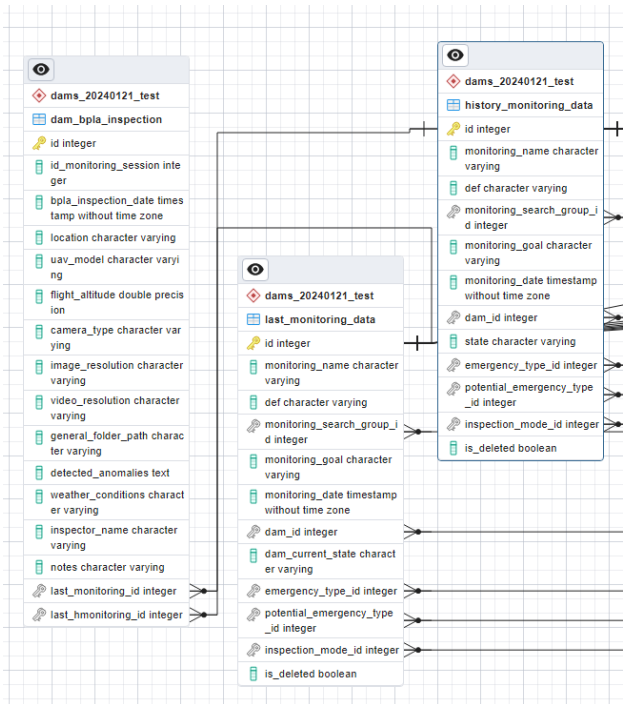


Fig. 2. BPLAinspection relation

2.3 Programme functions in terms of users

Work with the database can be performed for different users, the system functionality is implemented on the basis of different user roles. We have included 4 main roles in the system:

A brief description of the features of the functionality of these roles is given in Table 1:

Table 1. Users and roles.

No.	Role Names	Functionality
1	Admin	Administration Administration Role management Access rights Monitoring user activity
2	Guest	View dams on the map, search for a dam
3	Simple registered user	Viewing the list of dams on the map, their designations, search View ingmonitoring results
4	Geophysicist, Assistant Geophysicist	Viewing dam data Working with monitoring functionality Monitoring and reporting
5	Owner of the dam	Viewing, modifying, adding data on your dam Assigning researcher to monitoring Creating monitoring
6	All	Registration Authorisation Viewing basic statistics

2.4 System Interface

The reservoir monitoring system consists of backend and frontend. There are multiple endpoints and a developed interface. The microservice architecture makes the application flexible and functional, available for easy scaling.

All users have their own functionality. The administrator's functionality after authorisation is displayed in the menu on the left side (Fig. 3.).

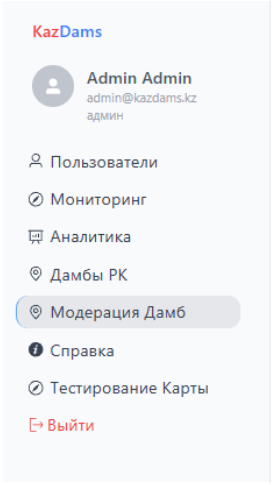


Fig. 3. Admin Menu

The dam monitoring section of the interface is

shown in Figure 4.

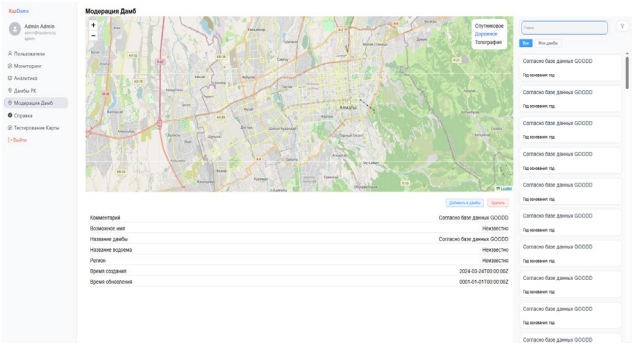


Fig. 4. “Dams monitoring” Interface

The domain ‘kazdams.kz’ is reserved for the system operation. Authorised user with appropriate rights can view the system interface. It is also possible to familiarise with the functionality in Swagger documentation.

In the menu section ‘Analytics’ we can see the analytics on the area of dam mirrors by years. It is possible to select a specific dam in the drop-down list (Fig 5).

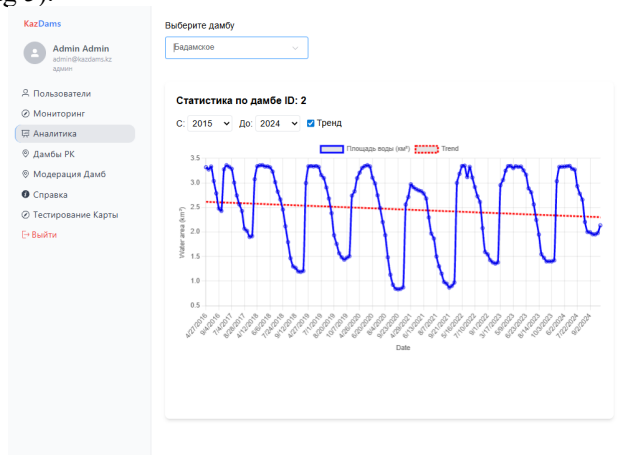


Fig. 5. Analytics

In the menu section “Dams” we can see the existing dams. We can select different dams and see them on the map and look at the data.

In the menu section “Monitoring” section where we can see the results of dam monitoring.

In the “Users” section, there is standard administrator functionality related to allocation of user rights.

The menu section “Help” contains reference information about the subject area of the system.

All maps in the application can be viewed in three modes by switching layers: satellite map, road map, topographic map.

3 DEVELOPMENT OF THE SERVER PART OF THE SYSTEM FOR MONITORING WATER RESERVOIRS AND DAMS IN KAZAKHSTAN

Reservoir and dam monitoring systems are critical for preventing accidents and ensuring water safety. This section considers the development of the server part of the platform, which provides collection, processing and analysis of data on the state of water bodies in Kazakhstan. Figure 6 presents Swagger documentation showing some of the endpoints of the developed programming solution.



Fig. 6. Swagger documentation

3.1 Server-side architecture

The backend service is implemented in Golang, which provides high performance and reliability. Fiber is used as the web framework, allowing for the processing of HTTP requests with minimal overhead.

Main components:

- Starting the service:
  - cmd/main.go is responsible for initializing the configuration and starting the server.
  - Logging is implemented using slog with JSON-format support.
  - A flexible configuration system is used, loaded via config.Init().
- API handlers:
  - All business logic is organized in internal/handler/.
  - A REST API is supported with various endpoints to handle reservoirs, inspections, lab samples, alarms, and data monitoring.
  - Used Swagger to generate API documentation.

- Monitoring data processing:
  - Data is collected from various sensors, drones and geodetic systems.
  - The system of aggregation of historical and actual data (history_monitoring_data,

last_monitoring_data) is implemented.

o Work with geographical coordinates of reservoirs and dams is supported.

3.2 Data security and protection

- Authentication and authorization

o JWT token is used to protect API requests.

o User passwords are stored in encrypted form (bcrypt).

o A mechanism for checking user activity and roles for access control is implemented.

o Security API. Middleware to validate incoming requests and protect against SQL injection, XSS and CSRF attacks.

o Use HTTPS to protect transmitted data.

3.3 Containerization and deployment

The project uses Docker for environment isolation and easy deployment. Docker Compose makes it easy to configure dependencies and services. CI/CD is implemented via gitlab-ci.yml, which automates build and deployment processes.

3.4 Machine Learning

Machine learning is used to find potential dams (dams with unknown coordinates) by analyzing map images. Designed an Artificial Neural Network (ANN) that receives as input fragments of 1300 m by 1300 m sections. The ANN analyses these digitized fragments. The output of this ANN is presented in the database of our proposed software solution. The output parameters and the result of the ANN are reflected in the tables of our database, and a complete description of this work is provided in a separate article.

Furthermore, it is planned to utilize machine learning to analyze the existing data and assess the dam's state (whether it is in an emergency or not). This aspect is currently under development.

4 NOVELTY AND PROSPECTS OF THE PROJECT

As aspects that are advantages of the developed software product are:

- Integrated monitoring. Combining different types of inspections (geodetic, geotechnical, physical-chemical, visual) in one service.
- Integration with drones and IoT devices for real-time data collection.
- Use of machine learning
- Prediction of emergencies based on historical data.
- Automatically analyses changes in dam structure using image and sensor data.
- Geographic Information System (GIS) - Support of work with reservoir coordinates and their dynamic display on the map.
- Integration with state monitoring systems.

Scalable architecture using Golang + Fiber for high performance. Containerization (Docker, Kubernetes) for easy scaling of the system. API extensibility to support new types of inspections and automated monitoring.

6 CONCLUSIONS

The developed server part provides:

- Reliable and fault-tolerant operation of the monitoring system.
- Flexibility through a microservice approach and containerization.
- Support for scalability due to high-performance solutions on Golang and Fiber.

In the future, we plan to implement machine learning for predicting emergencies based on collected data, as well as integration with government monitoring systems.

The development of a GIS system for digitalization and data processing of the dam is one of the necessary and significant tasks of our project.

FUNDING

The Ministry of Science and Higher Education of the Republic of Kazakhstan has funded the research, IRN AP19675038.

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Seismic and resistivity surveys to assess the damage to the sakuradani kofun tumuli caused by the noto earthquake

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ABSTRACT

The Noto Earthquake of January 2024 caused substantial damage to the Sakuradani Kofun Tumuli in Takaoka City, Toyama Prefecture, resulting in notable sinkholes and cracks across Tumulus No. 1 and Tumulus No. 2. To accurately assess this damage and inform future remedial measures, a comprehensive, non-destructive investigation was conducted using both seismic exploration and electrical resistivity tomography (ERT). In tumulus No. 1, a central 1–2 m-deep layer shows low shear velocity (Vs) and high-elevated resistivity, indicating a loose layer. On the other hand, a 4 m-deep area of the mound was observed with low Vs and low resistivity, consistent with a water-saturated, loosened fill. In Tumulus No. 2, a 6 m-deep southern-slope sinkhole appeared as a high-Vs and high-resistivity area. In addition, the northern slope exhibits a shallow resistive peak overlying a low-Vs and low-resistivity at 4m deep, indicating another potential loose area.

Keywords: seismic exploration, electrical resistivity tomography, earthquake, historical heritage,

1 INTRODUCTION

The Sakuradani Kofun tumuli, located in Takaoka City, Toyama Prefecture, were one of the historical heritage sites that suffered the most damage from the Noto Earthquake. In Tumulus No. 1, small sinkholes and cracks were mainly found near the slope of the burial mound. In Tumulus No. 2, a large sinkhole 6 m deep occurred on the south slope (Fig.1). In this study, we present the results of a comprehensive survey combining two-dimensional resistivity surveys and high-resolution seismic wave shear velocity (Vs) inversion as a non-invasive exploration method for the inner structure of the Sakuradani Kofun tumulus. The aim is to investigate the internal conditions and identify vulnerable areas up to a depth of approximately 10 meters in a mound with a height of 5 meters. The results of this survey provide a detailed description of the subsurface conditions up to a depth of 5 meters, enabling an assessment of the internal structure of the tumulus mound and the extent of damage that has penetrated the interior, while identifying areas of concern.

2 METHODOLOGIES

In the two-dimensional resistivity survey, a portable dynamic cone penetration test (DCPT) was performed to measure ground strength and to investigate a method currently under development that utilizes the DCPT device to complement the traditional resistivity survey.

In seismic explorations, 24 geophones spaced 0.5 meters apart were used. A chirp signal generated by a portable vibration-speaker source, with a high-frequency range of 20-300 Hz, and cross-correlation signal processing were employed to improve the signal-to-noise ratio and enhance the accuracy of investigating the shallow layer at a depth of approximately 10 meters.

3 RESULTS

Fig. 2 shows the results of the ERT survey and Vs inversion using the Rayleigh wave survey. Both Tumulus No. 1 and No. 2 had an average Vs velocity of 190 m/s and a resistivity of 700 Ω·m. In Tumulus No. 1, a low Vs of 140 m/s and a high resistivity of 1,500–2,500 Ω·m area were observed at a depth of approximately 1–2 m in the central part (a-1 in Fig.2) of the line A2-A3. Additionally, a low Vs of 110 m/s and a low resistivity of 300 Ω·m area were observed throughout the area at depths of 4 m or deeper (a-2 in Fig.2). The former was a relaxation area. Meanwhile, the latter is believed to be a water-saturated layer caused by rainwater infiltration, both of which are considered vulnerable areas. In Tumulus No. 2, a sinkhole with a depth of 6 m on the southern slope was observed as a high Vs of 350 m/s and

a high resistivity of 1200 Ω·m area (b-1 in Fig.2). Additionally, a high resistivity area was observed on the surface of the northern slope (b-2 in Fig.2). A low Vs and low resistivity area (b-3 in Fig.2) was observed at a depth of approximately 4 m in the deep part of the mound, considering that this area has become vulnerable due to the earthquake.

ACKNOWLEDGEMENTS

We want to express our gratitude to Takaoka City and Dr. Mai Sawada for providing us with the opportunity to conduct this field survey.

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Fig.1 Bird-eye view of Sakuradani Kofun tumuli

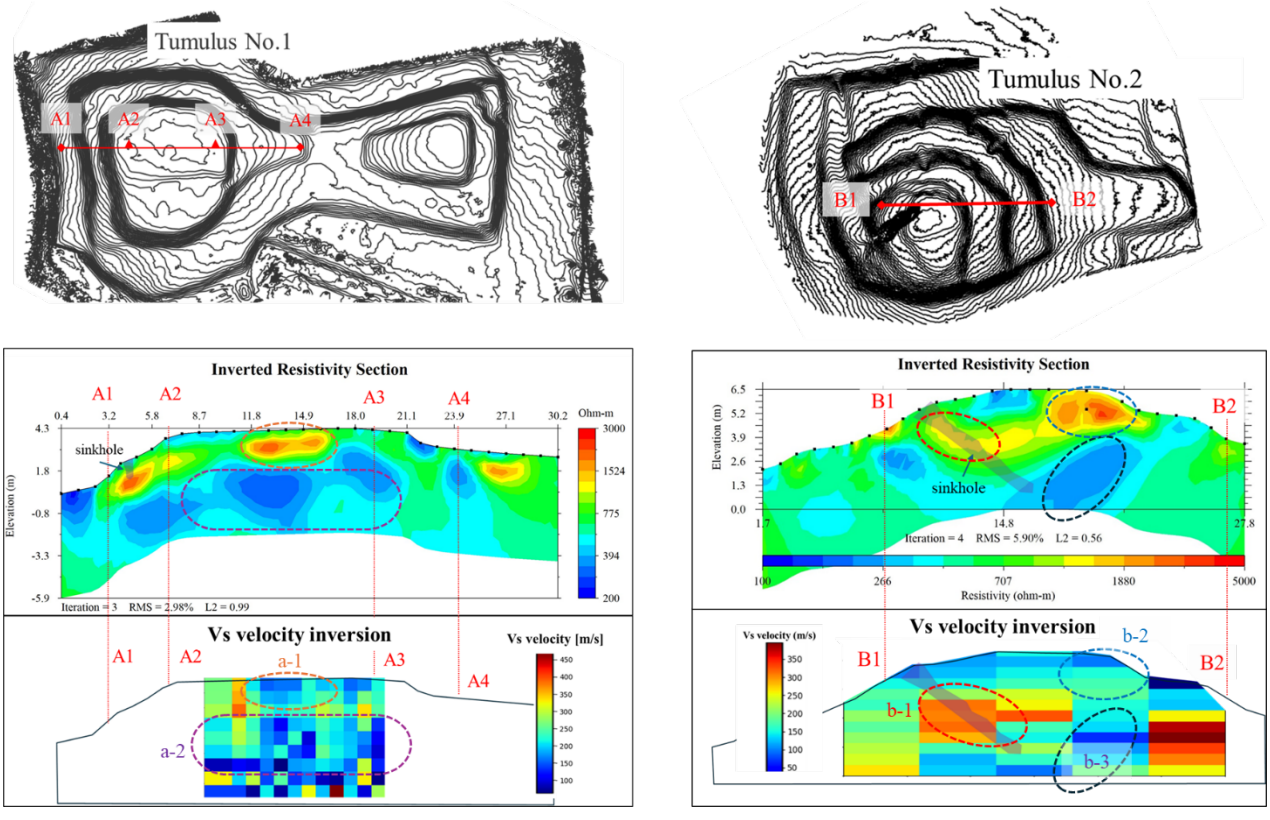


Fig.2 The result of ERT resistivity inversion and Vs inversion of Tumulus No.1 (left) and No.2 (right).

Artificial stone construction and its heritage structures in Japan: traditional and ecological method for SDGs

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ABSTRACT

Artificial stone construction is a method developed by civil engineer HATTORI Choshichi in Japan. It uses a combination of mixed soil "Tataki" and building stones to provide strength and water resistance. There are some artificial stone constructions which were build more than 100 years ago and have been well preserved especially in central Japan. Some of them are designated as cultural properties. The material for mixed soil is a combination of soil, slaked lime, and water. They are well mixed and compacted with wooden hammer. The compacted mixed soil is filled between building stones, and each building stone doesn't contact with each other. Therefore, the compacted mixed soil is working as filling and glue materials for building stones. The strength of the soil is not so much as concrete, however the combination of stones might keep enough strength as gate or retaining wall along the river and coast. Furthermore, artificial material is only the slaked lime $Ca(OH)_2$, which can be made with low energy compare to Portland cement.

Keywords: soil, slaked lime, water, compaction, building stone

1 INTRODUCTION

Artificial stone construction is a traditional civil engineering method developed by Japanese civil engineer, HATTORI Choshichi. The method is the combination of Tataki and building stones. It had been used for civil engineering construction, such as seawalls and embankments around 1900 before the spread of the modern concrete construction method in Japan. Some of them are well preserved especially in central Japan. However, those constructions have been used over 100 years and some of them were deteriorated. Near future, they have to be repaired by appropriate method as historical constructions.

2 ARTIFICIAL STONE CONSTRUCTIONS

Artificial stone construction is mainly composed of mixed soil "Tataki" and building stones. The mixed soil is composed of decomposed granite soil, slaked lime and water or seawater. A mixture of them is compacted with wooden hammer tightly. The compacted mixed soil is filled between building stones, and each building stone doesn't contact with each other. Therefore, the compacted mixed soil is working as filling and glue materials for building stones. Building stone, decomposed granite soil, and water are natural products. Slaked lime is from baked limestone which is also a natural one. Artificial stone construction is low carbon footprint method compared to modern Portland cement.

Some artificial stone constructions are still well preserved especially in Tokai area, central Japan. The old port of Yokkaichi had been constructed by HATTORI Choshichi in 1894. Fig 1 shows the old coastal embankment composed of artificial stone structure, which has many holes to throw tides for breakwater. It was designated as national cultural properties in 1996. New coastal embankment was established outsides of the port, and the old embankment doesn't work today.

There are some civil engineering structures built with artificial stone construction in central Japan (Fig. 2). Those structures are located along river or coast. The largest structure is the Port of Nagoya. It had been constructed by Aichi Prefecture from 1896 to 1911. Most of them might had been repaired by modern concrete method. Therefore, it isn't completely known that how many structures have been preserved in the port now.

Fig.3 shows the mixed soil and building stones zoomed up on the embankment in the old port of Yokkaichi (Fig. 1). Building stones are mainly granite. Coarse sand grains can be recognized in the mixed soil which fills between building stones. There are not so many original documents for the construction of artificial stone. An old report by Aichi Prefecture[1)]

records the many laboratory tests of mixed soil and explain the method of the artificial stone construction. However, some unclear details have been remained. Therefore, we firstly try to make the mixed soil with different ratio of slaked lime to decomposed granite soil.



Fig. 1. The old port of Yokkaichi, Mie-Prefecture, Japan.

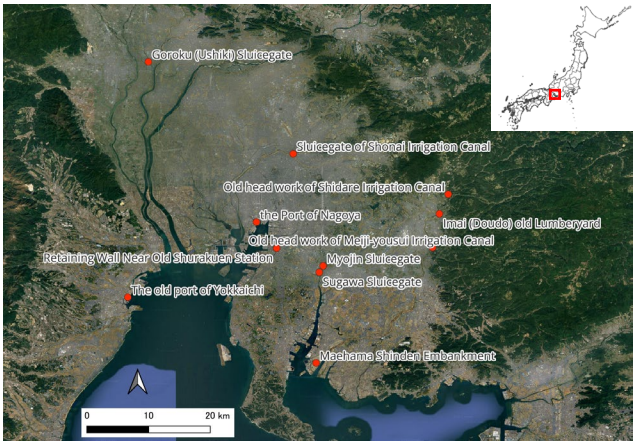


Fig. 2. Artificial stone construction structures in central Japan.

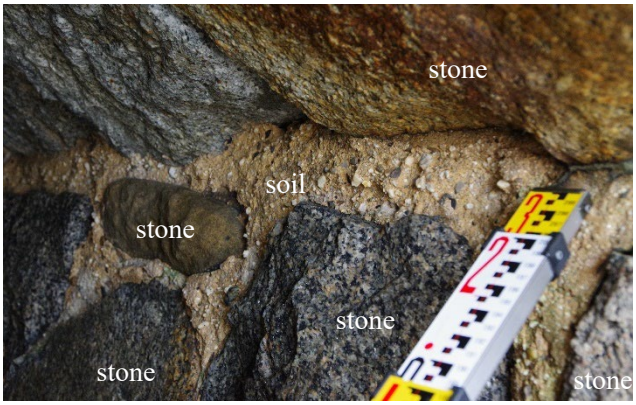


Fig. 3. Mixed soil and building stones of the old port of Yokkaichi.

3 LANORATORY TESTS FOR MIXED SOIL

3.1 TEST SPECIMENS

Cylindrical specimens were prepared to conduct unconfined compression tests. The size is 50 mm in diameter and 100 mm in length. An air-dried sample of artificial soil, which was created from the mixture of some grain size of siliceous sands and clay, and slaked lime is mixed with water. Weight ratio of slaked lime to artificial soil is prepared 3 kinds as follows, 1:8, 1:10, and 1:15. Weight ratio of water to soil is 10%. A hammer and mould were used to make specimens. Compaction energy was equal for all specimens. The first layer with 6 blows, the second layer with 7 blows and the third layer with 9 blows has been compacted.

Test specimens had been removed from the mould and cured in the desiccator, in which temperature and humidity keeps about 25 ℃ and 60 %. Unconfined compression tests had been conducted after the curing times of 7 days, 28 days, 56 days, 112 days, and 168 days. Three specimens had been prepared for 3 kinds of weight ratio and 5 curing times. Total 45 specimens had been ready for the compression tests.

3.2 Unconfined Compression Tests and Its Results

The tests had been conducted following JIS A 1216:2009 in 2020. Cylindrical specimens were uni-axially compressed in the direction of the axis of specimens. Fig. 4 shows stress-strain curves as a result of weight ratio of 1:8 and curing times of 24, 56, and 168 days. The stress shows the peak values of about 300 kN/m^2 to the strain of about 0.8 % at the curing time of 24 days. Compression stress is finished at about 2/3 value of the critical stress or 2 % strain after the critical value of the strain. Each specimens show each peak stress and similar stress-strain curves. Therefore, the value could be recognized as critical strength for unconfined compression tests. In addition, the peak stress is about 400 and 800 kN/m^2 for the curing time of 56 and 168 days. The critical strength has developed with the curing time.

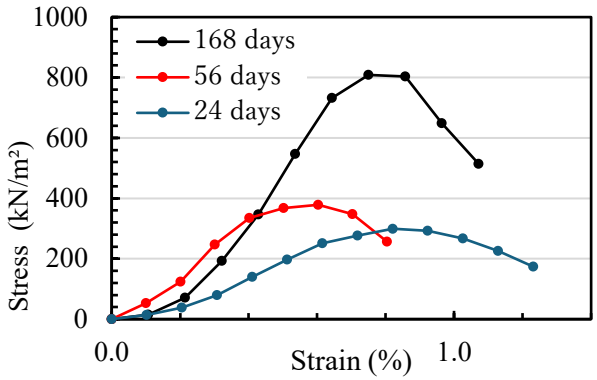


Fig. 4. A sample of stress-strain curve for compression test

4 CONCLUSIONS

We successfully confirm the strength development of “Tataki” with curing time.

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Mapping geological hazards – problems and trends

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ABSTRACT

The article discusses problems related with dealing with geological hazard (geohazard) maps for planning the engineering protection from hazardous natural processes of various scales, practically implemented in the Russian Federation. Standardization and unification of geohazard map compiling processes is discussed with regard to completeness and sufficiency of the information they should present. Hazardous process data exchange between different business entities implementing their projects in adjacent territories is discussed. Establishing a unified geoinformation system to include standardized geohazard data is also discussed.

Keywords: geological hazard, geological hazard mapping, engineering protection of territory, geoinformation system

1 INTRODUCTION

Completeness and sufficiency of the initial data precondition the quality of engineering protection solution design, thereby having a direct impact on the cost of construction and operation of facilities placed on challenging terrain. Moreover, unaccounted risks – and, as a consequence, errors made at early stages – can increase the costs of engineering protection solutions many times over and even make it impossible to operate a facility safely.

This article is aimed at identifying problems associated with both the geohazard mapping and establishing effective exchange of already obtained data, including the use of digital capabilities for the creation of geoinformation systems to contain information on natural hazards. Solving these problems would reduce risks and optimize costs for engineering protection for capital construction projects.

2 GENERAL REQUIREMENTS GEOHAZARD MAPPING

The first group of problems encompasses the geohazard mapping process. Russian regulations are limited by stating the need to compile such maps, while leaving quite a wide degree of freedom for mappers. A geohazard map should be the standardized tool where design engineers could obtain reliable information about real risks that can be protected against and use it to decide the placement of a facility in the area under consideration.

Geohazard mappers can go to extremes: either describing all possible risks, including low-probability ones, or failing to account for processes that could potentially threaten the structure.

Example: During construction of a facility in Sochi's mountain cluster, potential alterations to the hydrological regime of the stream basin were not accounted for. This resulted in the activation of mudflow processes, requiring the implementation of additional protective measures.

In addition to real risks to facilities under construction or existing facilities, the geohazard map should outline their influence areas. The mappers shall not only identify the hazards, but also use this data to either recommend some engineering protection solutions or claim the need to move the facility to be constructed out of the hazardous area.

3 IDENTIFYING ADDITIONAL RISKS AND THEIR INFLUENCE AREAS

The second problem that would require comprehensive solution is connected with the spread of hazards beyond the area of research. Existing regulations do not allow for the allocation of money for surveys outside the facility's location area, which would be necessary to compile a complete geohazard map.

4 DATA EXCHANGE PROBLEM

Another relevant problem is associated with data exchange and coordination of actions between customers

implementing separate isolated projects within the same area. Typical example is the large-scale construction of tourism and sports infrastructure in the Sochi mountain cluster.

Several customers constructed their facilities in stages, within a relatively small area. Each of them carried out his project-specific survey separately. As a result, not only the presence and potential impact of natural hazardous processes occurring in adjacent areas (for example, up the slope), but also man-made hazardous processes were neglected.

5 CREATION OF A GEOINFORMATION SYSTEM

The last matter discussed in the article highlights one of the trends in implementing various projects on challenging terrain. In the context of today's digitalization opportunities, it seems necessary to establish a unified geoinformation system (GIS) to include standardized geohazard data.

In our opinion, it would be desirable to create a unified state-owned geoinformation system of hazardous natural processes to include all deliverables of relevant engineering surveys. As an option, the data can be picked and loaded into the system at the stage of State Examination of a project in two stages: when the initial application is filed, with subsequent updating after obtaining the approval.

Should it be established, even with a step-by-step organization of data access, such a system would significantly reduce financial risks for investors. This would be especially true for those planning to develop areas with other projects already completed nearby. The capability to preemptively populate such a system with geohazard data would allow investors to share the burden of engineering surveys.

6 CONCLUSION

A number of measures are desirable for more efficient construction on challenging terrains and optimization of engineering protection costs. It is necessary to legally establish approaches to geohazard mapping, namely, such maps should present risks that actually threaten a facility under construction, identify hazardous process influence area, and provide recommendations regarding placement of the facility and necessary engineering protection measures. Customers should have the right, in case of specific combinations of conditions, to perform additional research to identify hazardous processes. All data shall be accumulated in a single geoinformation system to enable initial risk assessment and to reduce engineering survey costs.

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Features of the construction of buildings in the historical center of the city

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ABSTRACT

In the last decade, the reconstruction of buildings and structures in Moscow has taken on significant proportions. Particularly complex and responsible are the works related to the reconstruction and construction of facilities in the historical center of the city. This is due not only to the presence of numerous historical and architectural monuments and specific engineering-geological conditions but also to the high density of existing development in the area, the significant development of various underground structures and utilities, as well as the fact that the examined structures continue to be in operation.

Keywords: express method, historical center, geotechnical monitoring

Soil excavation and the construction of underground and above-foundation structures to varying degrees affect the surrounding geological environment, which, in turn, impacts the deformation behavior of adjacent buildings.

In this regard, regional regulatory documents prescribe the conduct of geotechnical monitoring throughout the entire construction period and for at least one year after its completion. Generally, geotechnical monitoring includes a set of activities that not only assess the deformation behavior of the reconstructed buildings and those adjacent to the construction site but also identify possible changes in the foundation soils and the surrounding soil mass during the construction process. The information obtained from geotechnical monitoring should ensure the development of measures to prevent and mitigate potential negative consequences from construction activities, thereby preserving the existing structures that fall within the influence zone of the new construction.

To implement the geological component of geotechnical monitoring, it is necessary to organize regular observations of changes in the geological environment, primarily regarding changes in the physical and mechanical properties of soils and the groundwater level (GWL).

However, carrying out traditional geotechnical monitoring methods, such as drilling geological workings, boreholes, and pits, installing deep marks, and other measures, is challenging in the historical center and dense urban environment.

For the safe conduct of construction work and to obtain information about potential changes in the engineering-geological conditions of the site during the erection process and the state of the structures of buildings that fall within the influence zone of the new construction, it is most advisable to apply survey methods that can operate and provide reliable information without disrupting the operation of the building or the ongoing construction work.

Based on the experience of organizing engineering-geological surveys for various construction purposes, progressive field pilot activities using mobile equipment for in situ testing can be noted.

In the historical center of Moscow, geophysical methods are successfully used for the examination of building foundations and the soils beneath them, as well as for conducting geotechnical monitoring. In particular, the electric contact dynamic cone penetration method (EDCP) is employed, where soil types are established based on electrical logging, and information about the physical and mechanical properties of soils is obtained from dynamic cone penetration results. In this case, there is no need for drilling boreholes or conducting laboratory soil tests. Another

geophysical method that can be applied in constrained conditions is the seismic reflection method in the modification of the common depth point (CDP) using SH-polarized shear waves. This method is used to study the structural features of the section at greater depths (up to 5 meters) to assess the karst-suffusion hazard of the area. It can be performed in the presence of asphalt-concrete coverings and permafrost.

The examination of the foundations of existing buildings is possible using the seismic acoustic probing method for piles and foundations. This method is used to determine the depth of the foundation base or the embedment of the pile, the contact of the foundation base or pile with the soil, the condition of the masonry or pile material, and significant defects and cracks in the masonry or pile.

The article discusses the advantages and disadvantages of various geophysical methods, as well as examples of their application on sites located in the historical center of Moscow. Some of these sites are architectural monuments and part of the cultural heritage.

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Accounting for resonant phenomena in the upper layers of dispersed soils in predicting the dynamic behavior of pile foundations

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ABSTRACT

A comprehensive numerical-analytical method for calculating pile foundations for seismic impacts is proposed. The report presents the results of numerical-analytical and natural studies on predicting vibrations of the soil base and pile foundations, taking into account the possibility of resonance effects in the upper soil layers under seismic impact. The analysis of the amplitude-frequency characteristics of the foundation was performed for analytical one-dimensional models using the analytical method in the DEEPSOIL seismological program and for spatial models of the soil massif in the SCAD structural analysis software. A high degree of correspondence is shown between the analytical approach, generally accepted for the purposes of seismic microzoning, and the numerical solution of the problem, which allows taking into account the influence of piles on the rigidity of the reinforced pile foundation at the design stage.

Keywords: amplitude-frequency characteristic of the soil base, SCAD Office, SHAKE, DEEPSOIL

Introduction. Experience shows that earthquakes do not always lead to catastrophic consequences. Costs and damage can be reduced by minimizing seismic risk when it is adequately assessed. The seismic effect of an earthquake depends on the dynamic properties of the soil, the thickness of the layers, water content, groundwater level, magnitude and frequency composition of the seismic impact. The main mechanisms of seismic vibration transformation are: deepening of foundations into more rigid layers, elimination of resonance phenomena, effective use of nonlinearity of stress-strain relationship. The action of these mechanisms leads to amplification of weak and weakening of higher intensity impulses.

Methods. Analysis of response spectra of real accelerograms shows that relatively small changes in the dynamic characteristics of the system can significantly change the system's response to seismic action. It is possible to either synthesize a single calculated impact in the form of a narrow-band process that is dangerous for the structure under study and tuned to its natural frequencies, or synthesize a broadband accelerogram based on the envelope of the standard spectrum.

In our studies, the synthesis of broadband harmonic action was chosen – the accelerogram CA-482 (Fig. 1). It was created to reduce the labor intensity of calculations for seismic effects by replacing a set of analog accelerograms with one synthesized model of action for a certain level of dissipation during vibrations (Fig. 1).

The disadvantage of this synthesized accelerogram is its imbalance. This problem was eliminated by using the "three-hit" balancing method in SCAD Office.

Results. A comparative analysis of methods for assessing resonance effects was performed for 9 types of soil foundations and 4 values of the thickness of a seismically homogeneous layer; 36 soil conditions were considered using 72 models in the DEEPSOIL and SCAD programs.

A comparative analysis of the soil response in one-dimensional soil columns in the DEEPSOIL program and in the spatial finite element SCAD model for a semi-rocky foundation of category I in seismic properties demonstrates the same picture (Fig. 2, *1*). For the normal soil conditions, represented by hard clayey soils (category II) and for soft water-saturated fine sands (category III), the results of the analytical and FEM models are also practically identical (Fig. 2, *1*). High-frequency pulses cannot be recreated in the kinematic formulation of the FEM problem solution and are naturally filtered.

This does not introduce significant errors into the calculation results due to the fact that the filtered frequencies are far removed from the ranges of the main natural frequencies of civil structures` vibrations.

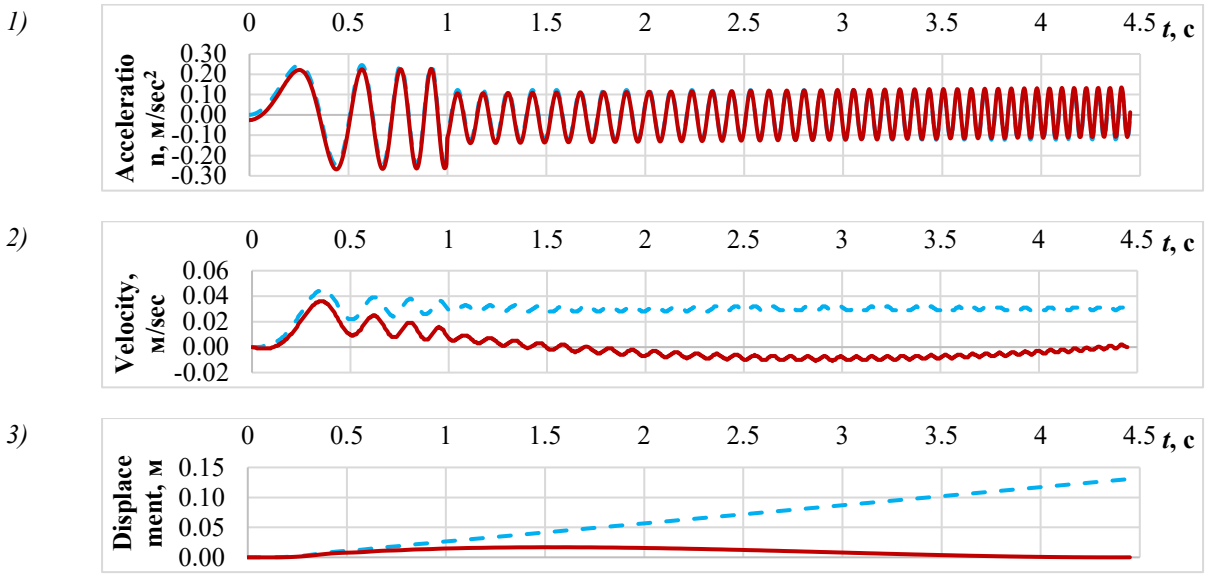


Fig. 1. Numerical analysis object, including accelerogram CA-482 before (blue line) and after (solid red line) balancing:
1 – acceleration in time domain; *2* – velocity; *3* – displacement.

Conclusion. For pile foundations design using numerical FEM models, it is recommended to use the thickness of the seismically realizing as the model calibration parameter. The validating frequency of the real subsoil should be taken from the results of a field assessment of the resonant characteristics of soils at the stage of surveys using the microseismic method and subsequent interpretation using the Nakamura method. A more accurate determination of the dynamic properties of soils and thickness of the seismically realizing layer is recommended to be performed using borehole seismic logging in the presence of boreholes.

Proposed complex survey and numerical calculation method is applicable for designing pile foundations, unlike 1D seismological soil models. It also allows for the consideration of the development of destructive surface waves, which can increase the shear forces in piles up to 3 times.

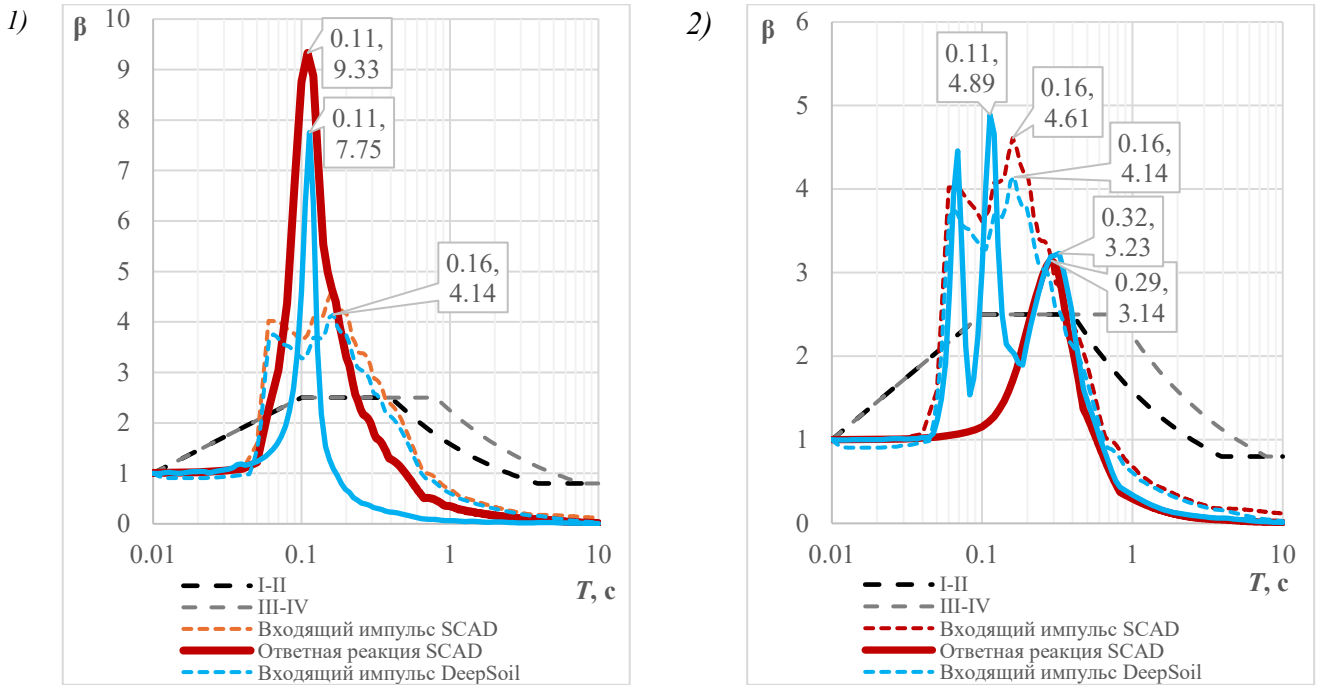


Fig. 2. The spectrum of the dynamic coefficient β for an array of 30 m thickness:
1 – of soft rock soils (category I); and 2 – of water-saturated silt soils (category III).

Stability Assessment of Ash Mound Using Traditional and Advanced GIS-enabled Modelling Approaches

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ABSTRACT

Slope instability in ash mounds poses significant environmental and economic risks, necessitating rigorous geotechnical assessment for effective management. This study evaluates the stability of a problematic ash mound using both traditional and advanced GIS-enabled modelling approaches. The assessment incorporates field surveys, laboratory testing, and numerical modelling of slope instability based on the limit equilibrium method. A two-dimensional (2D) slope instability analysis was employed using GeoStudio, focusing on six critical sections (A–F) selected based on the steepest contour gradients. Furthermore, three-dimensional (3D) slope stability analysis was carried out using Scoops3D. Site characterization included drone-based surveys and in situ testing with 12 boreholes (20–30 m deep). Notably, extensive erosion gullies and deep cuts (up to 2 m wide and 10 m deep) were observed, exposing underlying ash deposits and raising concerns about the serviceability and stability of the ash mound. Results from 2D analysis indicate that while the overall factor of safety (FoS) exceeds 1.5 across all sections, local instability (FoS <1.5) is identified in four sections (A, B, E, and F), particularly in areas with severe erosion. The 3D analysis identified failure-prone zones precisely compared to the 2D assessment, highlighting the limitations of traditional modelling while selecting cross-sections in complex geomorphological settings. Based on these findings, remedial measures such as slope regrading and gully stabilization are recommended to enhance slope integrity and prevent further erosion. This study emphasizes integrating GIS and 3D modelling into conventional slope stability assessments, as 2D analysis may overlook critical failure zones. The findings contribute to the broader understanding of geotechnical reliability in ash mound management, emphasizing the need for proactive monitoring and mitigation strategies. Future research should explore incorporating real-time monitoring and advanced numerical techniques to further enhance predictive accuracy in slope stability assessments.

Keywords: slope instability, ash mound, three-dimensional modelling, drone survey, geographic information systems (GIS)

1 INTRODUCTION

Ash mounds, formed from the disposal of industrial byproducts like fly ash, have faced numerous failures worldwide, causing significant environmental and economic damage (Santamarina et al., 2019). Ash mound failure due to slope collapses has led to groundwater contamination, air pollution, and structural harm to nearby infrastructure (Deonarine et al., 2023). Ash mounds are highly prone to slope failure due to poor compaction, weak shear strength, and water retention, with risks worsened by rainfall, groundwater changes, poor drainage, and loading (Srikanth et al., 2022, Niu et al., 2022).

Traditional 2D slope stability models are widely used but often fail to predict instability in complex cases due to simplified cross-sections. They overlook spatial variations in material properties, pore pressure, and slip surfaces, leading to inaccurate assessments. In contrast, 3D models provide a more realistic analysis, identifying critical failure zones, estimating depths, and predicting deformation for better stabilization design variations (de Kooker et al., 2024).

This study uses conventional and GIS-based modelling to assess the stability and capacity enhancement of an ash mound in Eastern India. Field surveys, lab tests, and simulations evaluate these methods for slope stability and risk mitigation.

2 MATERIAL AND METHODS

A site visit and visual survey were conducted to assess the existing conditions of the Ash mound. A drone survey captured the three-dimensional topography of the ash mound, and the aerial data was used to generate a 1m resolution digital terrain model

(DTM). Ash samples were collected and analyzed in the laboratory to determine key geotechnical properties, including grain size distribution, density, and shear strength parameters. A slope stability analysis was carried out using data from both field and laboratory assessments to evaluate ash mound stability concerns.

2.1 Numerical tools for slope stability assessment

GeoStudio (SLOPE/W) and SCOOPS3D were used for slope stability analysis. SLOPE/W, a 2D limit equilibrium tool, evaluates failure surfaces and calculates the factor of safety (FoS). SCOOPS3D, a USGS-developed 3D tool, applies the method of columns to assess FoS by integrating topography and material properties.

2.2 Methodology

The two-dimensional slope stability analysis was conducted for six cross-sections (A to F) selected based on the steepest gradients. In contrast, a comprehensive three-dimensional stability assessment was performed for the entire ash mound using a 1m resolution DTM in SCOOPS3D. This study evaluates two scenarios: (1) a dry state representing pre-rainfall conditions and (2) a post-rainfall wet state where pore pressure effects are incorporated using a pore pressure ratio (r_u) of 0.1.

3 RESULTS AND DISCUSSION

The 2D and 3D slope stability assessments identify regions where the factor of safety (FoS) falls below 1.5 under dry and rainfall conditions, as shown in Fig. 1 and 2, respectively. The results reveal localized instability in four sections (A, B, E, and F), where FoS dropped below the 1.5 threshold. While cross-sections C and D exhibited FoS values above 1.5 in the 2D assessments, the 3D analysis showed their proximity to unstable zones, highlighting a potentially misleading sense of stability in the 2D evaluation. The 3D analysis effectively delineated failure-prone areas, capturing instability risks not fully identified in the 2D assessments.

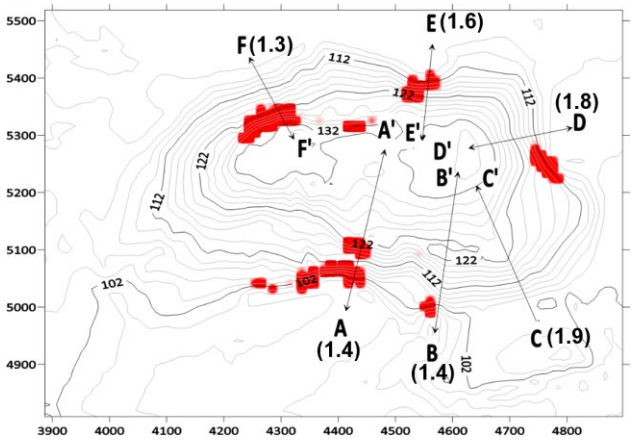


Fig. 1. FoS for cross-sections from 2D assessment and unstable regions with FoS < 1.5 from 3D assessment in the dry state.

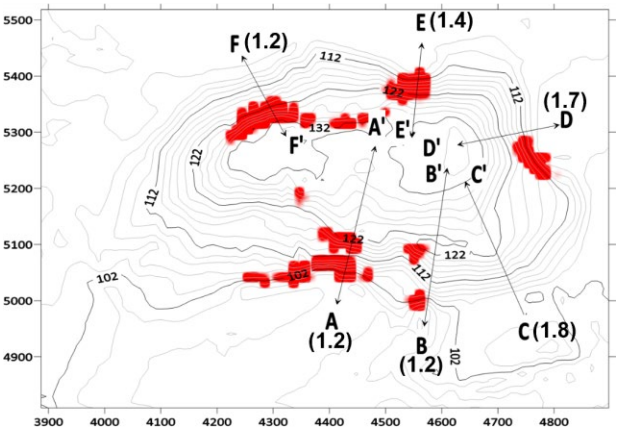


Fig. 2. FoS for cross-sections from 2D assessment and unstable regions with FoS < 1.5 from 3D assessment in the wet state.

5 CONCLUSIONS

Conventional two-dimensional (2D) analysis enables quick assessments but relies on cross-sectional representations, limiting its ability to capture spatial variability and potentially resulting in incomplete stability evaluations. In contrast, advanced three-dimensional (3D) modelling offers a more comprehensive assessment by accurately identifying failure-prone areas and providing a deeper understanding of overall slope behaviour. These findings highlight the need for an integrated approach, where 3D analysis first identifies critical failure zones, followed by detailed 2D evaluations for localized assessments.

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TC-305 Geotechnical Infrastructure for Megacities and New Capitals

Establishing Characteristics of Cyclic Freeze-Thaw of a Road Pavement and Subgrade in Almaty Region

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ABSTRACT

This paper analyzes the results of monitoring of temperature and moisture in layers of a road pavement and its subgrade on the section (58 km + 895 m) of the "Almaty-Bishkek" highway. The monitoring was carried out using a special measuring system. In particular, based on the measured temperature values, characteristics of cyclic freezing-thawing (number and duration of the cycles, minimum temperatures in the cycles) in layers of the road pavement and at points in subgrade are established.

Keywords: pavement, subgrade, sensors, temperature, moisture, freeze-thaw cycles

1 INTRODUCTION

Climatic conditions greatly influence the strength and durability of highways, especially in regions with pronounced seasonal variations in climatic conditions. Cyclic freezing and thawing significantly reduces the strength and durability of road surfaces (Teltayev et al., 2018; Teltayev and Suppes, 2019; Teltayev et al., 2020). Therefore, the study of the temperature regime and characteristics of cyclic freezing and thawing in road surfaces and roadbeds is an urgent problem.

2 EXPERIMENTAL SECTION

To conduct a long-term monitoring of temperature and moisture in layers of a road pavement and its subgrade, an experimental section was selected not far from Almaty (at 58 km + 895 m) on the "Almaty-Bishkek" highway, where a set of sensors for temperature and moisture were installed.

The road pavement structure on the section consists of the following layers:
Layer 1: Fine-grained dense asphalt concrete, thickness $h_1 = 5$ cm;
Layer 2: Coarse-grained porous asphalt concrete, $h_2 = 10$ cm;
Layer 3: Fine-grained dense asphalt concrete, $h_3 = 6.5$ cm;
Layer 4: Coarse-grained porous asphalt concrete, $h_4 = 15$ cm;
Layer 5: Fine sand and gravel mix, $h_5 = 25$ cm;
Layer 6: Coarse sand and gravel mix, $h_6 = 35$ cm;
Subgrade soil: Heavy sandy clay loam, $h_7 = 140$ cm.

3 TEMPERATURE AND MOISTURE MONITORING SYSTEM

In order to carry out long-term monitoring of temperature and moisture on the selected road section, a special measuring system with temperature and moisture sensors was installed. The sensors are installed in the layers of the road pavement and in the points of the subgrade at different depths (3 cm, 11 cm, 22 cm, 45 cm, 78 cm, 111 cm, 145 cm, 180 cm, 240 cm, 300 cm) from the fine-grained dense asphalt concrete pavement surface. The system is powered by solar panels. Temperature and moisture values of air and in the specified locations are measured hourly and the measurement results are saved.

4 RESULTS AND DISCUSSION

4.1 Freeze-Thaw Cycles

Figure 1 shows a graph of changes in air (asphalt concrete pavement surface) temperature on the experimental section from October 2021 to March 2022. This graph highlights freeze-thaw cycles. As can be seen, five periods containing freeze-thaw cycles are highlighted.

The current standard assessment methods and regulatory requirements (ST RK 1218-2003; ST RK 1225-2019) for the frost resistance of road materials and soils are analyzed. The experimentally obtained real characteristics of cyclic freezing and thawing of the road pavement and subgrade are compared with the standard ones and

recommendations for their improvement are given.

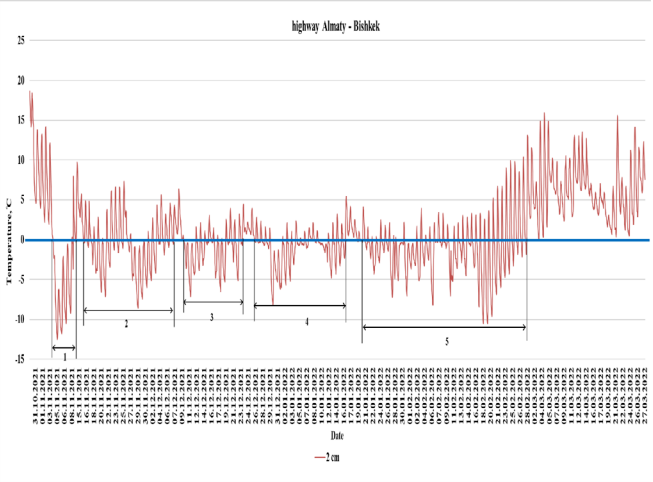


Fig. 1. Graph of air temperature changes indicating freeze and thaw cycles

4.2 Temperature on Different Days of the Cold Period

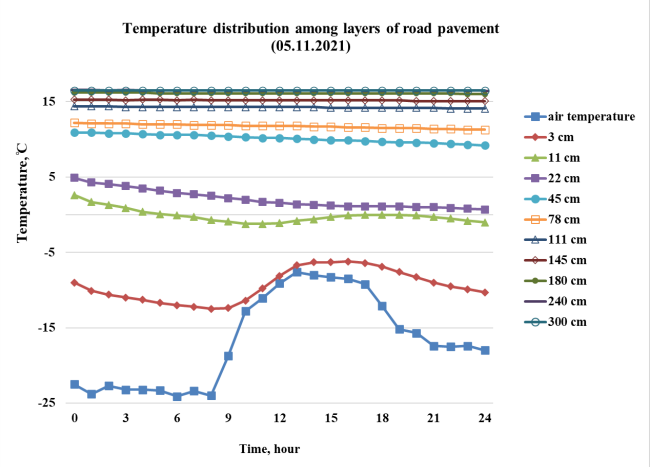


Fig. 2. Daily temperature distribution in road pavement layers and subgrade (November 5, 2021)

In the late autumn period (Fig. 2) a strong decrease in air temperature is already observed, which has a significant impact on the upper layers of the road pavement. Air temperature fluctuates between -22°C at night and about -4°C during the day. At a depth of 11 cm, the temperature is also subject to significant daily fluctuations, reaching zero and below during the day, indicating active freezing of the upper layer. From a depth of 22-45 cm and below, temperature fluctuations become less pronounced.

At the end of the winter period (Fig. 3), the air temperature is characterized by significant daily fluctuations (from -17°C at night to about 0°C during the day). These fluctuations are strongly reflected in the temperature of the surface layers (for example, at depths of 3 cm and 11 cm), where the temperature changes significantly during the day and drops below 0°C. With increasing depth, the amplitude of temperature fluctuations gradually decreases. Already at a depth of 78 cm, daily changes become noticeably less pronounced. Starting from a depth of 111 cm and below, the temperature practically does not change during the day and remains at a level slightly above 4°C.

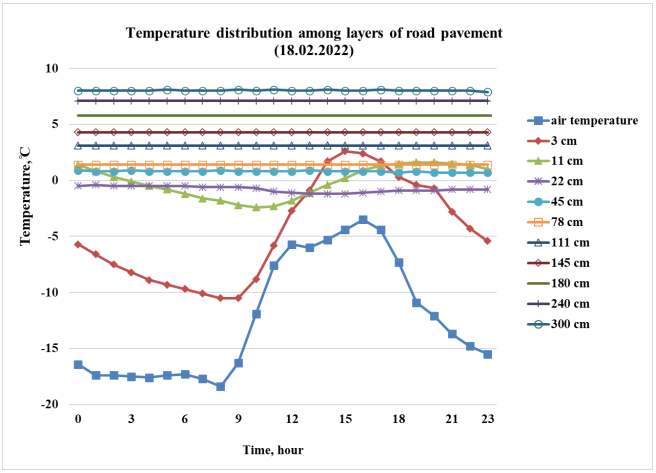


Fig. 3. Daily temperature distribution in road pavement layers and subgrade (February 18, 2022)

4 CONCLUSIONS

1. During the cold season of 2021 and 2022, there were 83 freez-thaw cycles of the pavement surface, which had different durations and minimum temperatures. The minimum temperature values in freezing cycles range from -7.1°C to -12.5°C.
2. During the cold season, daily temperature fluctuations occur only in the upper asphalt concrete layers (up to 22 cm) of the road pavement.

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Change of seismic properties of the base as a result of soil strengthening by deep mixing method

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ABSTRACT

Important points to note when estimating the planar distribution of groundwater levels using boring data are presented at first. Then, three case studies are introduced, in which, detailed surveys were conducted using simple devices to examine the planar distribution of groundwater levels after earthquakes in order to investigate the impact of groundwater levels on damage to low-rise housing. In cities affected by the 2024 Noto Peninsula Earthquake, studies are being conducted to install underground drainage pipes to lower the groundwater level and counter damage from future earthquakes. Measurements of the planar distribution of groundwater levels are being used to select appropriate locations for installing the drainage pipes.

Keywords: liquefaction, earthquake, groundwater level, low-rise housing

1 INTRODUCTION

The territory of Kazakhstan, covering a significant portion of Central Asia, is characterized by a complex geological structure resulting from a prolonged tectonic evolution. The structural diversity — represented by metamorphic formations, sedimentary basins, and massifs of various origins — forms the foundation of the country's rich mineral and raw material potential.

Under these conditions, a key priority in geotechnical engineering is the development and implementation of earthquake-resistant design solutions, including reinforced foundations and modern methods for stabilizing foundation soils.

2. GEOLOGICAL CHARACTERISTICS OF THE REGION

Engineering and geological investigations carried out in the southern regions of Kazakhstan reveal significant diversity in geological structures, with persistent stratigraphic patterns. In the territory of Almaty city, the geological and lithological conditions are primarily influenced by the presence of alluvial-proluvial deposits from the Middle and Upper Quaternary periods (apQII–III). Analysis of engineering-geological profiles allows for the identification of two most common stratification cases:

In the first case, the geological cross-section is composed of thick layers of collapsible loess loams of varying consistency and density, exhibiting pronounced structural instability when saturated.

In the second case, the entire stratum consists of water-saturated, weak, but non-collapsible loams, characterized by high compressibility and low bearing capacity.

Seismically, the territory of Almaty City is characterized by a high level of hazard. According to the probabilistic seismic microzonation map with a recurrence period of 475 years, peak ground acceleration values range from 0.4 to 0.65 g. Additionally, a significant number of tectonic faults are identified within the study area, the presence of which considerably increases the potential seismic threat. These factors necessitate detailed seismic justification in the design of capital construction projects within the investigated territory.

3 FIELD AND LABORATORY INVESTIGATIONS

At present, the Deep Soil Mixing (DSM) technology is widely used for strengthening the foundations of buildings and structures in the southern regions of Kazakhstan. In this context, the laboratory of JSC “KazNIISA” conducted research aimed at studying the strength and deformation characteristics of soil-cement columns. To obtain data on these properties, trial soil-cement columns are constructed. After reaching the required strength at 28 days from the time of installation, test columns are excavated to a depth of up to 3 meters. Core samples with diameters of 100 mm and 50 mm, extracted both horizontally and vertically, are subjected to laboratory testing.

The main objective of the research is to determine the unconfined compressive strength of the column material, the modulus of total deformation, and the Poisson’s ratio. The tests were conducted using uniaxial and triaxial compression apparatuses.

$$f_{cd} = a_{cc}\left(\frac{f_{ck}A_b}{\gamma_c}\right) \quad (1)$$

Based on the results of compressive strength, the bearing capacity of the material is calculated, which is subsequently used in structural design calculations:

Table 1. Summary table of specimen test results

Direction	State	$R_{c.max}$, MPa for DSM columns No.				
		1	2	3	4	$R_{c.\ average}$
Vertical	Natural	5,88	6,60	6,32	6,23	6,26
	Wet	3,46	3,80	2,83	3,70	3,45
Horizontal	Natural	5,22	-	-	6,60	5,91
	Wet	2,88	-	-	2,81	2,85

At a site located on Khaliullina Street, Medeu District, Almaty, Republic of Kazakhstan, tests were conducted using the Seismic Dilatometer Marchetti Testing (SDMT) as part of the study. In addition to the standard

Based on the results of shear wave velocity measurements, $V_{S,30}$ and $V_{S,10}$, in the upper 30-meter and 10-meter soil layers at five seismic boreholes, the following values were obtained.:

Table 2. Obtained values based on the results of velocity measurements.

No. of seismograph	Average shear wave velocity in a 10-meter-thick layer, m/s.	Height (m) — Average shear wave velocity in a 30-meter-thick layer, m/s
Test-1	249.2	275.1
Test-2	244.3	284.1
Test-3	246.1	270.5
Test-4	230.8	274.3
Test-5	249.4	-
Average	243.9	276.0

Moreover, the mentioned studies considered the possibility of establishing a relationship between the shear wave velocity and the unconfined compressive strength by applying a generalized empirical equation.

$$V_s = bf_c^{\varphi} \quad (2)$$

Where f_c - compressive strength,

b - correction coefficient

Excerpts from articles allow us to conclude that the shear wave velocities in soil-cement columns significantly exceed the corresponding values for natural (unimproved) soil. The increase in shear wave velocity leads to a rise in the natural frequencies of the foundation, which in turn reduces the vibration period of the superstructure. A shorter vibration period contributes to a decrease in the displacements of the upper part of the structure, thereby ensuring a safer and more effective design against dynamic loads.

4 TECHNOLOGIES OF REINFORCING ELEMENTS AND METHODS FOR THEIR CALCULATION

Regarding the design methodologies for foundations reinforced with structural elements, several different approaches exist, which are implemented in geotechnical software packages based on the finite element method, such as Midas and Plaxis.

1) Calculation using the "Equivalent Modulus of Deformation" method. The structural design of foundations incorporating reinforcing elements can be considered as a conditional (composite) foundation. Most existing theories for such foundations are based on the assumption of distributed loading and uniform deformation. Under these loading conditions, the stress distribution in the columns and the surrounding soil can be simplified, as illustrated in Figure 3.

$$E_{eq} = E_s(1 - A_s) + E_c A_s \quad (3)$$

Where E_{eq} = equivalent modulus of the composite foundation;

Based on the above-mentioned research results, it is proposed to use an approach analogous to the method for determining the equivalent deformation modulus to estimate the overall shear wave velocity in the reinforced soil mass:

$$V_{0,eq} = V_0(1 - A_s) + V_{0,c}A_s \quad (4)$$

Where $V_{0,eq}$ - equivalent shear wave velocity in the reinforced foundation;

2) The "beam element method" assumes modeling of piles and reinforcing elements as one-dimensional (1D) beam elements, which are not integrated into the mesh of three-dimensional (3D) soil elements. These elements can be placed in any position and orientation within the 3D soil mesh without directly interacting with its structure. Since the reinforcing elements are modeled using beam elements, this allows for a more detailed analysis, enabling the evaluation of displacements, internal forces, and moments in each element.

CALCULATION RESULTS

To perform verification calculations when comparing two design methods, the GTS NX Midas 3D software package was used. This program is designed for three-dimensional modeling of building–foundation interaction using the finite element method.

According to the results of the calculation with foundation reinforcement using two methods under seismic loading, the difference in deformation values is 0.8%, which indicates the acceptability of applying this method in seismic design.

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Geological features of rough masses in landslide sites in the area of motor roads along mountain slopes

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ABSTRACT

The article examines the geological and engineering-geological properties of soil masses in areas with increased susceptibility to landslides along highways running along mountain slopes. The influence of sliding of mountain massifs from mountain slopes and their influence on the general stability of slopes where motor roads pass are studied. As well as the structural or lithogenetic connection of mountain massifs on the stability of the slope from the impact of sliding soil massifs, which affects the change in the structure of the geotechnical horizon, i.e. on motor roads and other engineering structures located in this area. In field conditions, the authors established the causes of the occurrence of soil shifts from mountain slopes and the geometric parameters of the destruction of roadbed structures.

Keywords: landslide, mountain roads, soil masses, geotechnical properties, slope stability, road construction, mountain roads, lithogenetic anisotropy of mountain ranges, slope stability, strength criterion of mountain ranges, hazardous geological processes (HGP).

1 INTRODUCTION

The design and operation of motorways in mountainous areas are accompanied by a high probability of landslide processes. This is due to complex geological conditions, intensive water saturation and disturbance of the natural balance of slopes as a result of construction work. Existing forms of mountain relief leave a significant imprint on the choice of direction and placement of the motorway route. Laying a road route in specific local natural conditions cannot be subject to any rules that are of universal significance. However, according to the principles of tracing roads in various relief conditions, as well as the values of geometric parameters applied in this case, some analogies and similarities are found, allowing us to classify the resulting routes.

Understanding the geological features of soil masses is critical to ensuring roadway stability and traffic safety. A number of studies indicate the key role of lithological composition, rock fracturing and groundwater levels in the formation of landslides of different types of occurrence.

2. RESEARCH AREA

The study area is located in a mountainous area with steep slopes (30–45°), crossed by a road. The geological structure is represented by alternating layers of clay shales, sandstones and clastic rocks. Tectonic faults and weathering zones have been identified in places, which increases the risk of landslide instability.

The first and investigated landslide object, as noted above, is located in the Panfilov district of the Almaty region to the right of the road " Zharkent-Koktal-Arasan " (km 21+120) along the river. Borokhudzir. Geographically, the site is confined to the Dzungarian Alatau, the southern spur of the Dolantau ridge, the slope of southern exposure. The slope slopes and its exposure are the dominant factor in the formation of a landslide slope (Fig. 1).

a) b)



Fig. 1. Landslide section on the Zharkent-Koktal-Arasan ", located on Dolantau ridge :
a – landslide body on the site; b – second -order separation crack on the slopes.

All adjacent slope areas are covered with thick talus. The geological structure of the area includes effusive formations of different ages of the Carboniferous period. These are undifferentiated Lower Carboniferous tuff deposits of medium composition (Tournaisian, Visean), as well as tuffs and tuff lavas of the Upper and Middle Carboniferous age, represented by rocks of medium and acidic composition. The deposits are strongly dislocated and weathered, (C_{2-3}, m, n). In the southeast, in 300-, 400 метрахthey are superimposed by loosely detrital deposits of the Neogene and Paleogene.

The second object under study is located on the Dzhusaly- Kezen Pass at an altitude of 3,340 meters above sea level, near the Cosmostation. The Cosmostation is located 30 kilometers from the intersection of Navoi-Al- Farabi streets along an asphalt road.

4. MATERIALS AND METHODS

Field work included visual inspection of slopes, measurements of cracks, and sampling. Granulometric analysis, determination of plasticity limits, shear strength, and natural moisture were carried out in the laboratory. Additionally, stratigraphic sections and a hydrogeological survey of the groundwater level were carried out.

Making samples from mixtures by pressing. To determine the strength of road surfaces, standard samples are prepared from selected samples of asphalt concrete in laboratory conditions.

Cylindrical samples for determining the physical and mechanical properties of mixtures are made by compacting mixtures prepared in laboratory conditions. And also from asphalt concrete samples taken in the field. The cuttings or cores are heated in a muffle furnace to a temperature of + 165-180 ° C.

To prepare standard samples in field conditions, samples are taken from cuttings, then this cutting is

5. RESULTS OF FIELD RESEARCH.

5.1. At this site, the slope gradient and its exposure are the dominant factors in the formation of the landslide slope (Figs. 1 and 2). All adjacent slope sections are covered with thick talus. The geological structure of the site includes effusive formations of the Carboniferous period of different ages. These are undifferentiated Lower Carboniferous tuff deposits of medium composition (Tournaisian, Visean), as well as tuffs and tuff lavas of the Upper and Middle Carboniferous age, represented by rocks of medium and acidic composition. The deposits are highly dislocated and weathered, (C_{2-3}, m, n).

The movement of a landslide is evolutionary in nature (slow sliding) as a result of the constant introduction of debris into the upper part of the landslide body, which, “loading down”, presses on the lower block and moves it onto the road.

There are rows of collector wells near the road that are filled with debris. This explains the movement of the landslide up and towards the Borokhudzir River.

In the upper part, north-east of the landslide body, cracks of the second order of separation have begun to form (Fig. 1, b). The main sliding crack has been formed for a long time (5-10 years) (Fig. 2) and is gradually increasing.

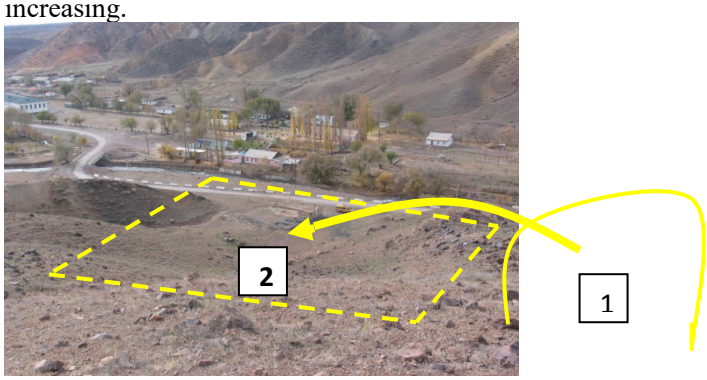


Fig. 2. Landslide body and recommended soil storage location (top view):
1 – landslide slope; 2 – place for storing cut soil



Fig. 3. Sliding cracks

RESULTS OF FIELD RESEARCH.

This section also presents the results of measuring the longitudinal and transverse slopes of the roadbed, the

transverse and longitudinal evenness of the roadway, the values of the longitudinal and transverse coefficient of adhesion and the hardness of road surfaces (Figure 4).

a) b)

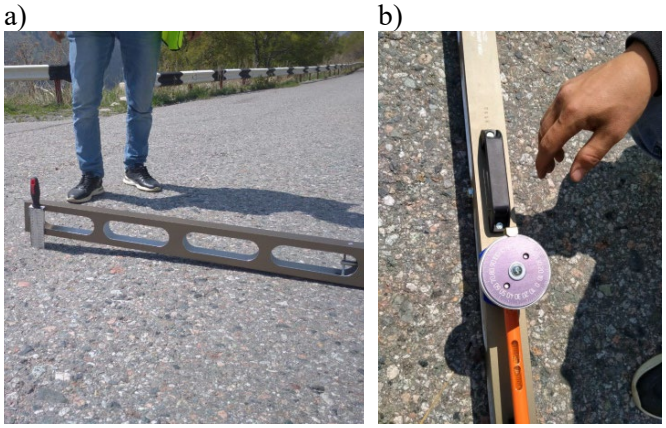


Figure 8 – Measuring the evenness (a) of road surfaces and the longitudinal and transverse slope (b) with a 3-meter ruler

6. CONCLUSION

The geological structure of slopes, the presence of water-retaining clay layers and seasonal water saturation are the main factors of landslide hazard when roads pass through mountain ranges. The following measures are proposed to increase stability: drainage, slope reinforcement, and monitoring of movements. A detailed geotechnical analysis is required before constructing new road sections.

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Geotechnical fiber-optic monitoring of the technical condition of bridges and tunnels in the Arctic coastal infrastructure

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ABSTRACT

Modern monitoring systems are not only sensors, they are complex automation systems involved, if necessary, in process management. Systems can be autonomous, or they can be one of the subsystems of more global systems. Monitoring is an important and responsible segment of modern technology, but with a number of organizational limitations related to the ongoing process of technological and digital evolution.
However, nowhere in the regulations have clear requirements been approved for the design of monitoring systems, other auxiliary systems and automation subsystems that would determine the necessary and sufficient number of sensors and other monitoring parameters. It is necessary to actively participate and combine the efforts of scientific and commercial structures in the development of the direction.
Due to the prospects for further growth and development of the monitoring segment, the main tasks of the industry arise:
- clearly identify the importance of monitoring systems and set the vector of their development and application;
- create uniform requirements (standards) for applied technological solutions and levels of solution organization, security, and the principles of solution organization themselves;
- identify patterns of behavior of operating organizations as part of the analysis of data received from monitoring systems;
- to determine the range of the main existing technical solutions, as well as to set the necessary tasks for the participants of the working group on research and development of new solutions;
- establish the role of participation in the operation of systems and facilities of scientific and design organizations;
- to attract international participants with world-leading experience in monitoring systems to organize joint work, exchange experiences and achievements.
Analyzing the main tasks, the general principle of organizing the construction and operation of systems should be based on an assessment of the current situation of the industry or the technological reserve. In the concept of the future, automated monitoring systems should include not only the usual sensors and the very definition of “monitoring” should be considered insufficient and replaced with “automated analytical systems".
Composition of analytical systems:
- physical parameter monitoring systems (sensors);
- analytical video surveillance;
- systems for monitoring weather and natural phenomena in the vicinity of critical infrastructure ;
- geotechnical monitoring and analysis systems;
- distributed vibroacoustic systems;
- event control subsystems (for example, UCSPS);
- alarm, warning and notification subsystems;
- subsystems for organizing data collection and transmission;
- the server part;
- software:
- for organization, management and parameterization,
- for operation and monitoring,
- for collection and storage,
- for physical modeling of soil properties
depending on changes in external climatic factors,
- for geotechnical numerical modeling,
- for analysis and study.
Combining all these technologies comes with one idea – it is the constant collection, processing and control of regularly changing parameters that (also) do not participate in already established global/conceptual systems, for example, on the Railway or in systems on highways and other facilities and other systems.

The parameters from all listed types of solutions are data that individually provide an objective picture of changes in the state of an element, external factors, or interference by third parties in the vital activity of an object. But the main advantage of the final solution concept is the ability to interconnect data to create analytical computing layers that will not only keep statistics on individual parameters, but will also be able to predict developing trends or upcoming events much more accurately in the future.
The design of the structures is carried out taking into account the following theses:
• Information on the strength characteristics of materials (standard resistances, calculated coefficients)
• Information on operating loads (location, magnitude, calculated coefficients)
• The structure is being designed to provide load-bearing capacity under operating loads with available materials (calculation scheme, coefficients of working conditions)
• Expected hazards of a different nature and dependence on other factors.
With the development of a new concept, it will be possible to supplement global solutions with new types of subsystems and solutions. There is a need to recycle and replace some outdated technical solutions, or combine different technologies in one solution. Some examples of new technical solutions:
- distributed control of deformation and temperature of permafrost soils at the base of buildings and structures;
- collecting speed information from systems along railway tracks using mobile complexes located on rolling stock;
- anti-karst systems along extended infrastructure facilities;
- a set of different standard solutions for HSR facilities;
- signaling solutions for ford crossings near populated areas without proper infrastructure;
- cable-stayed sensors, the technology of which is based on the magneto–elastic effect (Villari effect) - a change in the magnetization of the core when its linear dimensions change due to force action;
- development of domestic analogues of mid-range systems and server solutions and software.
This is an example of a part of actively developing technologies and solutions that have already been needed, and all of them are under active study or testing.
An additional, but not insignificant, task of the created concept is to actively identify the focus of the application of the described solution. The active use of analytical automated systems is aimed at providing a qualitative complement to the work of design and construction organizations. With the help of analytical systems, systems can be used, perhaps not only to prove the quality of design solutions and the reliability of structures depending on the impact of external factors, but also to assess the impact of periodic extreme situations, as well as provide numerical and quantitative facts of changes that accurately indicate the cause and remove unnecessary responsibility from specialists, giving information for making operational decisions. solutions and development of construction approaches and standards in the future.
The same approach is followed by the international construction and scientific communities.
There are many examples that point to the criticality of the lack of objective and relevant data.
For example, in the case of monitoring bridge crossings, we cannot assess and measure some impacts, moreover, they are not taken into account in the design – for example, solar radiation. As a result, the sensors show exceeding the thresholds set by the designers without taking into account parasitic stresses. Stresses that do not lead to the destruction of the structure, but disappear when the first plastic is reached. Such are, for example, stresses from welding deformations.
At the same time, using the example of bridge crossings, stresses may already reach the yield point at certain points, but this is not the limiting condition of the superstructure. Since our standards have adopted the elastoplastic work of the span .buildings. And the ultimate deformation is limited. That is, to control the limit state, it is necessary to control the deflection curve, and not the stresses at individual points. And voltage monitoring does not allow us to answer problems with the bridge or not, since it does not correspond to the calculation models used to decide whether the bridge is properly designed or not.
As a result, thresholds are set that "don't bother" or the system shuts down altogether.
The development of regulatory documentation will be designed not only to increase the relevance and applicability of analytical systems, but also to properly build the hierarchy and composition of participants.
Современные рабочие группы не включают одного участника “научные организации”, добавление этого участника радикально изменит восприятие систем мониторинга и аналитических систем и повысит их значимость и эффективность применения, наряду с высокой технологичностью самих решений.
Modern working groups do not include a single participant in “scientific organizations”, the addition of this participant will radically change the perception of monitoring and analytical systems and increase their importance and effectiveness of application, along with the high technology of the solutions themselves.

Keywords: monitoring, geotechnical engineering, permafrost soil
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Justification of changings in parameters of stressed-strained state of oil-contaminated sandy soils

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ABSTRACT

Laboratory tests were conducted to determine the strength characteristics of oil-contaminated sandy soils using the single-plane shear method. Coarse, medium, and fine sands contaminated with A-95 gasoline, diesel fuel, and heavy oil in concentrations ranging from 0 to 12% by dry soil weight were used as test samples. A methodology for conducting laboratory tests using the single-plane shear method for sands contaminated with oil products was described and tested. The results of laboratory tests showed that contamination of sandy soils with oil products leads to an increase in specific adhesion in the range from 12% to 10 times and to a decrease in the angle of internal friction in the range from 5 to 22% depending on the sand size, type of oil product and its concentration. Graphs of the internal friction angle and specific adhesion for sands of different sizes on the concentration of oil products are presented. A reduction in the design resistance of the foundation soil by 10–38% and the ultimate pressure on the foundation soil by 17–60% is justified, taking into account changes in characteristics of oil-contaminated sandy soils.

Keywords: stressed-strained state, angle of internal friction, specific adhesion, sandy soil, ultimate pressure, design resistance.

1 INTRODUCTION

An analysis of existing studies has shown that most authors adhere to the concept of the interaction of an oil product with sandy soil, in which the oil product is a wetting phase and covers the surface of mineral particles, forming oily shells, which cause the effect of particles slipping relative to each other and, consequently, the angle of internal friction decreases. [1-9]

It is known that the shear strength of sandy soils is provided precisely due to the friction forces that occur at the contact points of mineral particles, and therefore, in order to predict a decrease in the ultimate pressure and design resistance of the substrate in the event of an emergency spill of oil products, it is necessary to have information about the nature and degree of change in the strength characteristics of soils.

2 METHODOLOGY FOR CONDUCTING LABORATORY TESTS

Tests to determine the angle of internal friction and specific adhesion were carried out by single-plane shear method for sands of various sizes contaminated with gasoline A-95, diesel fuel and heavy oil at a concentration of C = 0.2, 6, 12% by weight of dry soil. The concentration of oil product and moisture in the test samples were set so that the pores were completely filled with pore liquid. Thus, the maximum and minimum concentrations of oil products were provided at minimum and maximum ground moisture.

3 THE RESULTS OF THE RESEARCH

The results of the research showed that contamination of sandy soils with oil products leads to an increase in specific adhesion in the range from 12% to 10 times and to a decrease in the angle of internal friction in the range from 5 to 22% depending on the sand size, type of oil product and its concentration. The calculated resistance, taking into account the obtained characteristics, is reduced in the range from 10 to 38% and ultimate pressure on the foundation soil is reduced in the range from 17 to 60%.

4 CONCLUSIONS

1. The maximum decrease in the angle of internal

friction is observed when the sandy soil is contaminated with heavy oil. With an increase in concentration from 0 to 12%, the angle of internal friction decreases by 16-22%, depending on the size of the sand.

2. The minimum decrease in the angle of internal friction is observed when the sandy soil is contaminated with gasoline A-95. With an increase in concentration from 0 to 12%, the angle of internal friction decreases by 8-17%, depending on the size of the sand.

3. The maximum increase in specific adhesion is observed when the sandy soil is contaminated with heavy oil. With an increase in concentration from 0 to 12%, the specific adhesion increases from 13% to 10 times, depending on the size of the sand.

4. The minimum increase in specific adhesion is observed when the sandy soil is contaminated with gasoline A-95 – with an increase in concentration from 0 to 12%, the specific adhesion increases from 14% to 5.5 times, depending on the size of the sand.

5. The intensity of the increase in specific adhesion depends on the type of oil product in the pore space of the soil, in particular on its viscosity. A significant increase in specific adhesion in case of contamination of sands with heavy oil and diesel fuel is presumably due to the high viscosity and, as a result, the high shear stiffness of oil products, in comparison with water, which has neither viscosity nor shear stiffness**.** Also, it should be noted that an increase in the adhesion of oil-contaminated sandy soils may be due to an increase in connectivity pressure.

6. According to the results of the calculations, there is a decrease in the design resistance of the foundation soil in the range from 10% to 38% and a decrease in the ultimate pressure on the foundation soil in the range from 17 to 60%, depending on the size of the sand, the type of oil product and its concentration.

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Application of hemihydrate and dihydrate calcium sulfate in road constructions

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ABSTRACT

All over the world, including in the Russian Federation, the production of mineral fertilizers is actively underway. Phosphate fertilizers used in agricultural activities all over the world are obtained by processing phosphorus ores. As a result, a huge amount of waste from such production accumulates at the landfills of such enterprises all over the world - hemihydrate (CaSO4 * 0.5H2O) and dihydrate (CaSO4 * 2H2O) calcium sulfate - phosphogypsum. The volumes of annual disposal of phosphogypsum at such enterprises are calculated in hundreds of thousands and millions of tons, which leads to the formation of huge dumps that are dangerous both in terms of groundwater pollution and in terms of technogenic safety. The main problem in terms of groundwater pollution with such storage is a large concentration of waste in one place (at the landfill). Almost everywhere in the world, mined and processed phosphorus ore is radioactive, which creates an additional threat to humans and the environment and greatly complicates the processing or disposal of such waste. It is important to note that the phosphorus ore processed in the territory of the Russian Federation, and the waste in the form of hemihydrate and dihydrate of calcium sulfate is not radioactive. This allows recycling production waste almost immediately after its receipt. This paper provides an assessment of the possibility and methods of using waste from the production of phosphorus fertilizers without additional measures for their purification, and without separating them into freshly processed material and material stored in landfills, which carries the potential of waste disposal without removal to a landfill and a realization of volumes stored in the landfills of industrial enterprises.

Keywords: waste processing, secondary raw materials, building materials, phosphogypsum, road bases

1 INTRODUCTION

Industrial enterprises of the Russian Federation and around the world produce mineral fertilizers, phosphoric and sulfuric acids from phosphorus ores and their processed products to solve agricultural problems and problems of the chemical industry. The raw materials in production are phosphorites and apatite Ca5(PO4)3(Cl,F,OH). During the processing, a large amount of waste is formed that requires disposal, storage, and, ideally, recycling and reuse. During processing, special attention is usually paid to the extraction of the most valuable impurities (for example, precious or rare earth metals). These issues are often and actively studied by domestic and Western scientists [1]. However, truly valuable impurities extracted during the processing of industrial waste often do not solve the problems of their storage or disposal, since their content in the total waste is not significant.

A by-product of the production of mineral fertilizers in this case is the hemihydrate (CaSO4*0.5H2O) and dihydrate (CaSO4*2H2O) of calcium sulfate - phosphogypsum. Phosphogypsum is stored in dumps at special landfills at production sites. The areas occupied by such dumps are estimated at tens of hectares, and the volume of stored waste is tens of millions of tons. At the same time, each enterprise producing waste in the form of phosphogypsum annually sends an average of 400-500 thousand m3 to landfills.

It is important to note that phosphogypsum obtained at enterprises of the Russian Federation is non-toxic and non-radioactive. However, when dried and mixed, it dusts, which is one of the problems of storing it in dumps. Russian scientists have conducted studies [2, 3] indicating the benefits of using phosphogypsum in agriculture to solve various problems (for example, to eliminate soil salinity, to add to soils to increase crop yields). Scientists have noted that with a certain treatment, a single local application of a small amount of material does not have a negative impact on the environment and can have some positive effect. As noted by the authors of scientific works, including [4], the presence of various impurities in phosphogypsum that require neutralization leads to the fact that only a small part of it is processed (less than 10% of the total production). If impurities are neutralized, the cost of phosphogypsum increases significantly in comparison with natural gypsum, its purification becomes economically unfeasible. The authors note many options

for using phosphogypsum, including in road construction. At the same time, the rate of accumulation of waste is an order of magnitude higher than the volumes of its actual use and processing, despite the potentially wide scope of use of phosphogypsum

2 MATERIALS AND METHODS

Large volumes of phosphogypsum are obtained as a waste product in the production of phosphoric acid, which is required for the further production of mineral fertilizers. Phosphogypsum is formed in two main forms – hemihydrate (CaSO4*0.5H2O) and dihydrate (CaSO4*2H2O) of calcium sulfate. The granulometric composition of phosphogypsum at the outlet of the line is close to silty sandy loam (particle size from 0.01 mm to 0.05 mm). As noted by domestic scientists [5, 6], the density of the skeleton at this stage is 0.83 - 1.27 t/m3, and the porosity coefficient is 1.3-1.9. Mineral particles of dihydrate have a density of 2.37 t/m3, hemihydrate – 2.52 t/m3. The filtration rate at the line outlet fluctuates in the range from 1 to 3 m/day; for phosphogypsum stored in waste heaps for a long time, the filtration rate decreases to 0.1 m/day. The dihydrate at the line outlet contains 19-20% of crystallization water, and the hemihydrate - about 6%. In this regard, the initial humidity of the hemihydrate is about 25-30%, and dihydrate about 35-40%.

The analysis showed quite specific, but not exhaustive prospects for using phosphogypsum, and the rate of utilization indicates the need to find areas for using this material.

3 RESULTS

Analysis of studies of existing technologies and experience of using phosphogypsum in some areas of construction indicates the presence of certain prospects for research into phosphogypsum. There are a few studies of phosphogypsum, technologies for its application and processing, extraction of rare earth metal microimpurities from phosphogypsum, etc. However, the practically undiminished storage volumes of this production waste indicate the impossibility, inexpediency of using the proposed technologies or the small volume of phosphogypsum processing associated with low demand for the final product.

Analytical calculations of road structures have shown the possibility of using phosphogypsum in road structures. From the point of view of calculating the road pavement for elastic deflection, both replacing sand with phosphogypsum without changing the overall thickness and replacing the crushed stone layer and part of the sand layer with a decrease in the overall thickness of the structure (for the considered structures) does not lead to a significant improvement in the performance of the structure in terms of elastic deflection. However, it is worth noting that phosphogypsum is a production waste - phosphogypsum itself, considering production, costs nothing, unlike sand and crushed stone, and its disposal at a landfill incurs additional costs for enterprises producing phosphogypsum. In addition, in certain cases, the use of phosphogypsum can lead to a decrease in the cost of excavation work due to a decrease in the overall thickness of the structure. Calculation of the thermodynamic resistance of road pavements showed that the value of the thermodynamic resistance of the considered multi-layer structure of road pavements increases up to 3 times.

4 CONCLUSIONS

The prospects for the use of phosphogypsum are significantly wider than the scope of road construction, and its use has a positive effect from both a technical and economic point of view.

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AsRTC10 «Urban Geo-informatics»

AsRTC10 «Geo-engineering for conservation of heritage monuments and historical sites»

Study on methods for extracting landslide risks using airborne LiDAR data on expressways

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ABSTRACT

In light of recent natural disasters in Japan, efforts are underway on expressways managed by NEXCO (Nippon Expressway Company) to acquire 3D point cloud data across the entire network, using Airborne LiDAR, to identify landslide disaster risks originating from outside the expressway area. In this study, we conducted trial verifications by performing differential analysis of data from multiple time periods, taken before disasters occurred at five locations along expressways where landslide disasters had previously occurred, to see if signs of disaster occurrence (landslide disaster risk) could be detected. As a result, while it was found that topographic changes before disaster occurrence could be detected in some areas, it was also understood that changes might not be detected depending on data accuracy, data acquisition timing, disaster occurrence patterns, etc., and we organized points to note for future data acquisition.

Keywords: Airborne LiDAR, 3D point cloud data, landslide disaster risks, differential analysis

1 INTRODUCTION

In light of recent natural disasters in Japan, efforts are underway on the expressways of NEXCO to acquire 3D point cloud data by conducting airborne LiDAR surveys across the entire network to understand disaster risks from outside the expressway area. It is thought that if the airborne LiDAR surveys are continued and data taken from multiple pre-disaster periods is accumulated, it may be possible to capture signs of disasters through differential analysis of the data. This paper describes the results of verifications conducted to determine whether signs of disasters (landslide risks) can be captured by performing differential analysis of pre-disaster data taken across multiple periods at locations along expressways where landslides have occurred in the past.

2 DIFFERENTIAL ANALYSIS LOCATIONS AND DATA COLLECTION

The locations where the difference analysis was performed are the five areas, within the jurisdiction of NEXCO, listed in Table.1, where landslides have occurred and for which data from multiple periods before the disaster was available. Since data from NEXCO alone could not provide sufficient pre-disaster data, we checked information on surveys conducted by national and local governments, such as the Geospatial Information Authority of Japan's database [1)], and after applying for and obtaining permission, utilized the data.

Table.1 Locations for differential analysis

No	Date of Disaster	Road Name	Location	Type of Disaster	LP Measurement Before Disaster (National, Local Government Data)
1	2019/10	Metropolitan Inter-City	Hachioji City, Tokyo	Slope failure (cut slope)	(2009)
					(2013)
2	2019/10	Tomei Expressway	Ashigarakami District, Kanagawa	Slope failure (natural slope)	(2012)
					(2019)
3	2021/8	Chugoku Expressway	Yamagata District,Hiroshima	Debris flow	(2017)
					(2018)
4	2019/8	Nagasaki Expressway	Takeo City, Saga Pref.	Landslide (road uplift)	(2009)
					(2011)
5	2023/7	Oita Expressway	Asakura City, Fukuoka Pref.	Slope failure (natural slope) and debris flow	(2009)
					(2017)
					(2019)
					2022
					(2022)

3 DIFFERENTIAL ANALYSIS METHODS

The two differential analysis methods described below were used. In conducting the difference analysis, positional correction processing using the CCICP method [2)] was performed on the ground data (ground point cloud data) from two periods to resolve small errors in positional accuracy.

3.1 Elevation difference analysis (Method 1)

This method involved calculating the difference in elevation values between two time periods using 50cm resolution grid data (Digital Elevation Model, DEM). The DEMs were generated through interpolation of the ground point cloud data. This approach offers simplicity in its implementation.

3.2 Normal direction difference analysis (Method 2)

The grid data used in Method 1 is generated by averaging and simplifying the original 3D point cloud data, which poses a challenge in capturing minute variations because small differences, such as how the laser points hit, particularly at slope-change points, can result in seemingly large difference values. Therefore, this method was attempted by referring to the M3C2(plugin) algorithm[3] of "Cloud Compare," a widely used free software for handling point clouds, which calculates the 3D distance between the nearest neighboring points using the original point cloud data. At that time, the search range for the nearest neighboring points was limited to the normal direction of the slope to reduce misidentification of slope-change points, etc.

4 DIFFERENTIAL ANALYSIS RESULTS

Fig.1 shows the results of analysis using the two methods for area No.2 of the five areas. Rainfall from a typhoon caused a large collapse at this location, where a tendency for small collapses to expand had been observed near the toe of a natural slope exhibiting unclear landslide topography. In the analysis results of Method 1, topographic changes that had occurred before the disaster at the site are clearly represented as differences. However, at other locations, many noises (false detections) indicating collapses were captured even though collapses of slopes, etc., had not actually occurred, making it difficult to say that topographic deformation had been accurately captured. This is thought to be due to the low accuracy of the data used (point density of ground surface data) and the effect of averaging it. In the analysis results of Method 2, while topographic changes at the disaster site are expressed as relatively clear differences, almost no noise is observed. This is thought to be because by using the point cloud data directly without averaging it, it was easier to exclude outliers and also because more accurate differences were calculated. In areas other than No. 2, no clear topographic changes at the disaster site were captured, or it could be that no clear topographic changes had occurred.

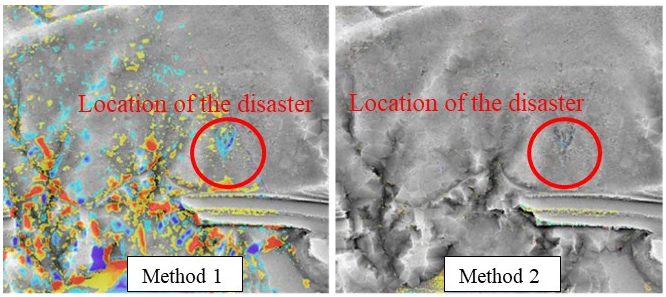


Fig. 1. No.2 Results of differential analysis before disaster (2019–2012)

5 SUMMARY AND FUTURE ISSUES

At the collapse site of the natural slope in area No. 2, topographic changes at the disaster site were captured without being affected by noise through normal direction difference analysis of two pre-disaster periods. On the other hand, in areas other than No.2, no clear topographic changes were captured, or it could be that no clear topographic changes had occurred. Future issues and considerations are listed below.

5.1 Ensuring data accuracy

It is desirable that the point cloud data have an accuracy of 4 points or more per square meter [4], but in some areas, the 3D point cloud data used in this analysis were significantly low in point density on the ground surface. Noise during difference analysis can be eliminated by ensuring measurement density, and also, if the point cloud density is sufficient, it might be utilized the elevation difference analysis, which is easy to implement. The irradiation capability of laser survey equipment has significantly improved, and high-density data is expected to be acquired in the future. Furthermore, it is thought that data accuracy can be ensured by standardizing methods for airborne LiDAR data acquisition, such as avoiding periods of dense vegetation growth in measurement, and irradiating diagonally on slopes when measuring slopes to account for laser reflection.

5.2 Data acquisition timing

Among the target areas selected this time, pre-disaster data acquired for areas No. 1 and No. 4 were taken considerably before the time of the disaster. Also, in area No. 3, there is a possibility that topographic changes did not appear because the data acquisition interval was short: one year. It is thought that if data is acquired regularly in the future, the number of cases where signs of disaster (topographic changes) can be captured will increase.

5.3 Examination of applicable disaster types and scales

For debris flows like those in No.3 and No.5, it is assumed that they rapidly lead to collapse and erosion during heavy rain, and it may be difficult to directly grasp the signs of occurrence. It is also assumed that signs may not be captured by airborne LiDAR data depending on the scale of the sign. It is desired to accumulate data on disaster cases in the future and analyze the types and scales for which disaster signs can be grasped by difference analysis.

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Use of sand covers for soil moisture control in archaeological sites

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ABSTRACT

Moisture loss due to evaporation decreases soil cohesion or causes cracks in archaeological sites under excavation or exhibition when the soil is covered with tarpaulins or buildings. Desiccation-induced degradation is difficult to fix; however, effective methods for mitigating evaporation have not been established. This work aims to demonstrate the potential application of sand protection covers to archaeological sites. Coarse sand covers with small water retention mitigate upward porewater transfer as they have small hydraulic conductivity in dry conditions. Evaporation experiments were performed on soil layers with sand covers of four different thicknesses, and the results were simulated with hydro-thermal coupled FE analysis. The experimental results demonstrated that all the 5 mm-thick or thicker sand layers significantly reduced evaporation speed and moderated desiccation cracks, compared to the bare soil case. The numerical simulations calibrated with the experimental data were extended to explore the effects of sand cover thickness. The results showed that the sand cover mitigated the evaporation, and the period of the 1st evaporation stage was extended with increasing sand layer thickness. The required minimum sand cover thickness, defined as the cover thickness that most efficiently reduces cumulative evaporation, was found to have a relationship with the period length of the continuous drying process. That is, the seasonal average rainfall interval of the archaeological site. The proposed protection method is useful to mitigate evaporation-induced damage in soil heritage and has prospective applications for long-term soil moisture retention.

Keywords: unsaturated soil, evaporation, soil water retention

1 INTRODUCTION

Soil moisture variation due to evaporation and rainfall infiltration can cause cracks involving strength reduction and hydraulic conductivity increase in archaeological sites under excavation and exhibition. Such degradation is hard to restore, while systematic protection methods have not been established. Sand exhibits a low water retention under atmospheric conditions and a rapid hydraulic conductivity reduction with increasing suction, which stops further evaporation by blocking the vertical soil moisture pathway. A set of laboratory elemental evaporation tests was performed to investigate the moisture control capacity of sand covers with different thicknesses. The experiments were numerically simulated, and a relationship between the sand layer thickness and evaporation reduction was developed. Furthermore, the influence of sand cover during drying/wetting loops was also investigated.

2 LABORATORY EVAPORATION TEST

Decomposed granite soil with a fine fraction content of 42% was utilized in this test, which is used for tumuli in the Asuka Village, Nara. The specimen (150 mm × 150 mm × 50 mm) was prepared by compacting the soil in an acrylic box to simulate the natural water content and void ratio of a burial mound (w=29%, e=0.927). An air-dried Toyoura sand layer with thickness of 0 mm (control case)/5 mm/8.5 mm/10 mm/20 mm was laid on the specimen, and dried in a constant environment chamber set at 20 °C and 60% RH for 160 h. The specimen mass variation was recorded with an electronic balance system every 15 min, and the cumulative evaporation amount and speed were calculated correspondingly, as shown in Fig. 1.

After test, the following aspects of soil desiccation influences were examined: 1. The severity of crack were determined by visual inspection after removing the sand. 2. The evaporation cumulative amount and speed were analyzed based on recorded data to emphasize the evaporation evolution patterns. 3. The vertical moisture distribution was measured after cracking the soil sample. 4. The monitored soil suction data were compared to the laboratory SWCC fitting curve.

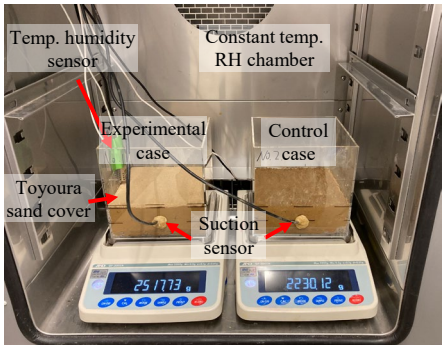
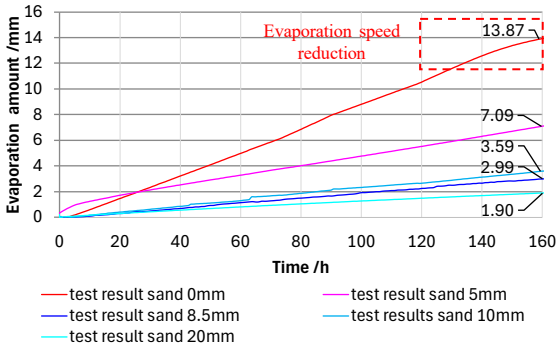
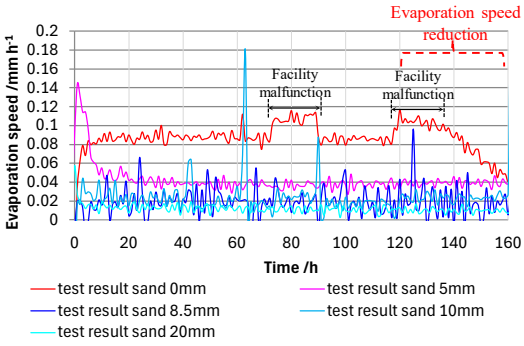


Fig. 1. Evaporation tests

Fig. 2 shows the measured evaporation cumulative amount and speed. the sand cover with a thickness of even 5 mm effectively decreased the total evaporation amount by 48.8% compared to the bare soil case; however, the effectiveness was less significant for further increase in the sand layer thickness. The bare soil evaporation speed exhibited a typical 3-stage pattern, in which the high-speed 1st stage ended just at 120h, while the other cases with sand cover kept a constant speed.



(a) Cumulative evaporation amount



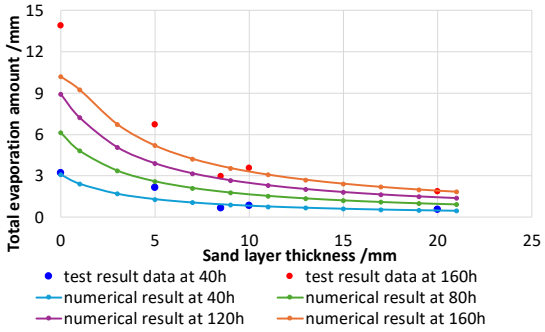
(b) Evaporation speed

Fig. 2. Test results of sand layer thickness 0/ 5/ 8.5/ 10/ 20 mm

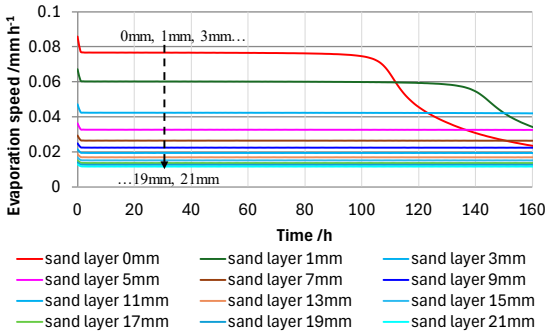
3 NUMERICAL SIMULATIONS ON SAND LAYER THICKNESS

A set of thermal-hydraulic coupled FEM analysis was conducted with Code_bright (Vaunat and Olivella, 2002). Numerical models with sand layers of different thicknesses from 0 mm to 21 mm were developed according to the evaporation test.

Fig. 3 shows the simulated evaporation process. The results qualitatively simulate the experimental data represented by the pink dots; however, relatively large deviations were observed in the thin sand layer conditions (0 mm/5 mm) because gaps between the shrinking soil and the inner wall were not considered in the simulations. A detailed speed variation plot is shown in Fig.3 (b), where the impact of increasing sand thickness is highlighted.



(a) Cumulative evaporation amount curve



(b) Simulated evaporation speed

Fig. 3. Simulation results of sand layer thickness 0~21 mm

4 CONCLUSIONS

1. The proposed sand layer protection was useful to reduce soil moisture loss due to evaporation, leading to a mild preservation condition.
2. The effectiveness of the sand layer was particularly obvious in cases with thin sand layers; however, the increase in the sand layer thickness did not significantly contribute to the evaporation reduction.
3. The relationship between sand layer thickness and reduced evaporation amount was obtained, and it showed that more than 3mm sand cover significantly suppressed both the evaporation cumulant amount and speed in the 1st evaporation stage of a typical 3-stage soil evaporation pattern.

ACKNOWLEDGEMENTS

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Detection of landslide potential based on regular deformation using DInSAR: a case study of Taipei city

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ABSTRACT

Around 70% of Taiwan's landscape consists of hilly terrain, and with the rapid urban growth in Taipei City, ensuring the safety of structures and roads built on these slopes has become a significant concern, especially with increasing population density. This study aims to use remote sensing methods to detect potential slope deformations proactively. Traditionally, this task relies on visual image analysis and on-site inspections, but Synthetic Aperture Radar (SAR) remote sensing technology offers a valuable tool for monitoring surface changes. This research applies DInSAR technology for an extensive, multi-temporal analysis to examine possible slope variations. We compare the in-situ survey method with observational data to classify slope conditions by assessing the degree and scope of deformation in slope units. Using data from both ascending and descending orbits, we gain a comprehensive view of surface deformation risks in Taipei City. Additionally, we explore the shifting potential of landslides, focusing on the heavy rainfall event of 2022.

Keywords:SAR, slope deformation, landslide hazard prevention

1 INTRODUCTION

To assess the potential risks associated with hillside slopes in Taipei City, municipal authorities conduct regular surveys, observations, and patrols to identify areas susceptible to instability and to evaluate the likelihood of slope failure comprehensively. In addition to conventional methodologies, the application of remote sensing technologies presents a viable alternative for performing multi-temporal investigations, facilitating the early detection of slope variability, and providing a comprehensive assessment of surface deformation trends. Such proactive measures are instrumental in enabling early warnings and mitigating potential losses. Remote sensing techniques should thus be employed to analyze continuous slope variation, geological characteristics, and the condition of critical preservation sites. This study utilizes a wide-area detection approach to inform disaster prevention and management strategies within Taipei City. Specifically, the Differential Interferometric Synthetic Aperture Radar (DInSAR) technique is employed to quantify vertical ground deformation and to infer slope instability through the analysis of various deformation indicators.

2 RESULTS

The analysis presented in this study covers seven years from March 2017 to May 2024. The short-baseline method enhanced interferometric pairs' coherence, limiting the temporal interval between image acquisitions to no more than 72 days. This constraint is particularly effective for maintaining high correlation in Sentinel-1 data, which operates at shorter wavelengths. The short-baseline, multi-temporal approach not only improves phase stability but also enables repeated verification of deformation patterns, thereby increasing the robustness of the analysis.

To mitigate spatial decorrelation, the spatial baseline threshold was set at 150 meters, a setting intended to optimize the accuracy of ground deformation measurements. Recognizing that different satellite tracks may offer varying sensitivities to slope orientation, the study further incorporated a track selection procedure post-analysis. This approach ensures that, for each slope unit, the detection results are derived from the track, which offers the most favorable observational geometry. This research's primary focus is on identifying potential landslide blocks within the administrative boundaries of Taipei City. Applying the short-baseline method, a high spatial density of measurement points - exceeding 1,000 points per square kilometer - was achieved, significantly enhancing the capacity for detailed slope stability assessments.

Subsequent processing involved independent deformation analyses for each orbital direction. As detailed previously, a moving window technique was applied to compare short-term deformation rates against long-term baseline rates, systematically recording successful detections. Key metrics were computed and categorized to support further interpretation, including the number of effective points within each slope unit, point density, mean displacement per point, and cumulative displacement for each unit.

The average cumulative displacement of effective points within each slope unit was utilized for final visualization and risk assessment, as illustrated in Figure 1. Slope units exceeding a predefined threshold were highlighted: specifically, those exhibiting a cumulative monthly vertical displacement rate of 4 mm or greater were classified as areas of concern. This threshold, set at twice the generally accepted attention value of 2 mm/month for continuous slope deformation, was adopted to ensure that significant and persistent ground movements were prioritized in the analysis. Red indicates high sliding deformation, while blue represents low sliding deformation.

Notably, the spatial distribution of detected deformation points was relatively dispersed across many slope units, complicating the delineation of coherent deformation blocks. Nevertheless, the results provide a critical foundation for subsequent hazard assessment and management planning.

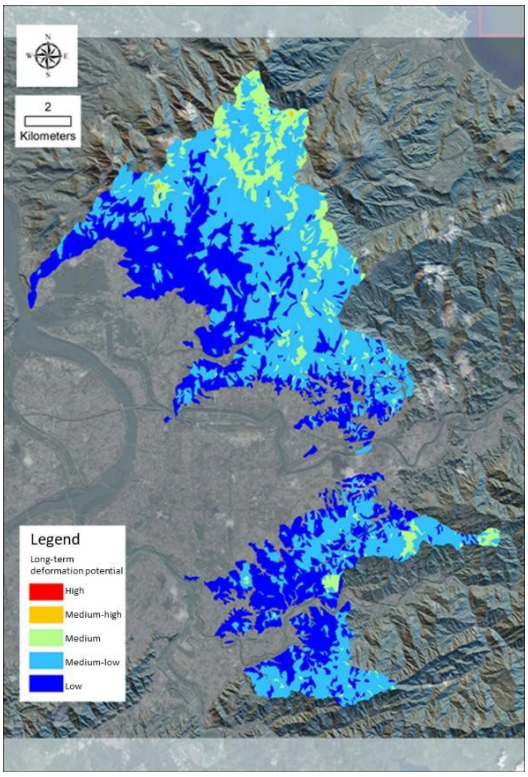


Fig. 1 The deformation-based landslide potential map of Taipei City

3 CONCLUSION REMARKS

This study develops a methodology for detecting ground surface changes and quantifying the magnitude and rate of vertical surface deformation. Based on the analysis conducted over seven years, the method has demonstrated the capacity to produce reasonable and reliable results under conditions of long-term monitoring. The principal conclusions and recommendations are summarized as follows:

3.1 Comparison of Evaluation Methods

When applying the point-count method based on unit area, the detected displacements are often too minor to effectively determine whether a site should be prioritized for further investigation or remedial measures. Conversely, while the point-displacement method allows for a broader assessment of slope unit behavior, it does not adequately capture the formation of coherent sliding blocks. In contrast, using long-term average displacement values for individual detection points more effectively highlights critical areas of concern, thus enhancing the reliability of risk identification.

3.2 Advantages of Event-Based Detection

When treating detections as discrete events, applying a moving average approach for temporal analysis has proven advantageous. Employing multiple rounds of statistical screening over time enables the identification of more confirmed surface changes compared to traditional, seasonally fixed detection intervals.

3.3 Limitations and Recommendations for SAR-Based Analysis

Although Synthetic Aperture Radar (SAR) analysis presents inherent limitations, this study has sought to mitigate these challenges by developing a large-area monitoring framework to identify potential sliding blocks across extensive slope units. However, the wide-area approach carries a risk of omission, particularly in regions with subtle deformation patterns. Therefore, for areas of critical importance, such as slope edges or zones characterized by low gradient, small-scale, high-resolution analyses are recommended to capture deformation trends more accurately and to elucidate sliding mechanisms.

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Underground space of Moscow. A case history

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ABSTRACT

The paper considers some geotechnical aspects of the underground space development within the boundaries of the historical center of Moscow, notably, beneath Bolshoi Theatre, Manege Square, and transportation tunnel in the Sokol subway station area. The underground work was carried out with preserving existing buildings and underground structures. An observational method was applied to ensure the safety of reconstruction process. The method involved constantly performed monitoring and settlements management on the base of the monitoring results. A case history of the application of the observational method throughout the entire reconstruction period is considered.

Keywords: underground space, underground structures, monitoring, observational method.

1 INTRODUCTION

Underground space development in densely built-up areas either of small cities or of metropolises has been one of the major problems in geotechnical engineering, in the past years.

Reasons for designing measures mitigating the impact of underground space development are usually the following:
-Existing conditions;
-Subsoil conditions;
-Site engineering conditions;
-Deterioration of materials and structuress;
-Possible impact of the environmental changes;
-Technical and economic feasibility and necessity of the preservation.

2 UNDERGROUND SPACE DEVELOPMENT IN THE CENTER OF MOSCOW

The paper presents a number of case histories of the underground space development in the center of Moscow:
-The Bolshoi Theatre area;
-The Manege Central Exhibition Hall;

The 20 m deep tunneling beneath the existing development of the XX century.

Geotechnical aspects of the project regarded as unique:
- The 20 meters deep underground section under the preserved superstructure;
-The total load transferred from the preserved superstructure to temporary supporting frame structure, which included jacked piles was 230 tf. manege central exhibition hall.

The second example of underground space development is a reconstruction of the Manege Central Exhibition Hall destroyed by the fire in 2004. The fire burnt up floor framing – trusses with roof sheathing, only leaving exterior walls.

Structurally, after the fire, the building was represented by supporting brick walls with a thickness of 2-3.5 m, located in plan along the contour of a rectangle covering the area of 160 m x 45 m.

The designed project envisaged and realised following geotechnical improvements:
-soil strengthening using Tube-a-Machette grouting
-method with hydraulic fracturing in soil with poor mechanical and deformation properties;
-bored piles instalation for strengthening fill or loose soil under the foundation bottom.

3 TRANSPORTATION TUNNEL IN A DENSELY DEVELOPED AREA

Alabyano-Baltiyskiy Transportation tunnel was constructed in the densely developed area with 5-13-storey buildings of different age (50-80th years of the XX century). Structurally, buildings comprised frames and frames mixed with transverse and longitudinal bearing walls made of bricks. The depth of a tunnel bed varied from 5 to 35 meters. The distance between the excavation retaining system and buildings varied from 1 to 14 m. The reconstruction project prescribed installation of geotechnical barriers constructed from a mixture of soil and injected grout.

Considering geotechnical features of the project made it possible to construct a sophisticated

underground transportation facility in the densely developed area.

4. CONCLUSION

The considered case histories of underground space development of the areas with different structural members and engineering conditions show the need to take into account geotechnical aspects when designing, and, especially, when preservation of existing development is demanded.

Borehole Information System and Subsurface Modelling Using GIS

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ABSTRACT

In metropolitan cities like Chennai, understanding the subsurface environment is critical for effective urban growth and infrastructure planning. Also, subsurface models with clear distinctions of various layers are beneficial for geotechnical analysis and the successful execution of engineering projects. Borehole data serve as the primary source of information that can yield valuable insights into the underground conditions of the Earth. However, accessing this data can be challenging due to its extensive and diverse nature. In this study, a digital information system has been built using collected borehole data that act as a repository for storing borehole information, including stratigraphic characteristics and other geotechnical parameters along the depth. The database also facilitates data integration, visualization, retrieval and analysis within a GIS framework. Using interpolation techniques and geostatistical methods, maps of subsurface layers and a 3D subsurface model are developed, highlighting the spatial distribution of various soil and rock layers at various depths. These models effectively depict the subsurface conditions, providing valuable insights into geological variations that assist in decision-making for urban planning, infrastructure projects, and environmental risk assessment. This framework is then incorporated into the website, enabling the locating of the boreholes across various regions of Chennai and facilitating an understanding of subsurface conditions before commencement of the project, which allows for informed site selection, effective foundation design, and sustainable urban development.

Keywords: borehole information system, subsurface model, GIS, sustainable urban development, stratigraphy

1 INTRODUCTION

Chennai, a rapidly expanding coastal metropolis in southern India, faces complex urban challenges exacerbated by its unique geological setting. Positioned along a dynamic coastline, the city contends with seasonal flooding, variable soil conditions, fluctuating groundwater levels, and heterogeneous subsurface layers, ranging from soft marine clays to weathered bedrock. These factors, combined with intense urban sprawl and infrastructure demands, underscore the critical need for robust subsurface characterization to guide sustainable development. Subsurface modelling emerges as a vital tool to address these gaps, enabling informed decision-making by visualizing the hidden layers that shape Chennai's foundation. As the city undertakes large-scale projects—from metro rail expansions to flood mitigation systems—the absence of a unified subsurface database poses significant challenges. Current borehole data, though abundant, remain fragmented, limiting its utility for city planners and engineers. This project addresses these gaps by developing a Borehole Information System (BIS), a geospatial database tailored to Chennai's subsurface complexity. By harmonizing dispersed borehole logs—detailing soil composition, layer depths, and lithological variations—the system will generate dynamic 3D stratigraphic models. Users, including urban planners and engineers, can input location-specific queries via a web interface to access real-time subsurface profiles, enabling data-driven decisions for construction and renewal projects. The integration of this model into Chennai's planning framework aims to enhance infrastructure resilience, reduce environmental impacts, and foster sustainable urban growth in the face of escalating climatic and developmental pressures.

2 STATE OF THE ART

Borehole Information Systems (BIS) have advanced the storage, analysis, and spatial visualization of subsurface data, supporting effective urban planning and resource management (Tipping et al., 2012). Integrating techniques like geostatistical interpolation (kriging, IDW) and 3D modeling tools (GeoModeller, Leapfrog) further refines subsurface representations by combining

borehole and geophysical data (Royse et al., 2009). In Chennai, a geotechnical GIS database has shown promise for 3D modeling (Divya Priya & Dodagoudar, 2018), yet issues with fragmented data and lack of real-time, location-based insights persist. This study aims to bridge these gaps by developing a unified, spatially enabled BIS with a web interface tailored to the complexities of Chennai's subsurface.

3 STUDY AREA

The study focuses on four key zones in southern Chennai, Tamil Nadu, India: Adyar (13.0067° N, 80.2572° E), Guindy (13.0102° N, 80.2157° E), Taramani (12.9863° N, 80.2432° E), and Velachery (12.9815° N, 80.2204° E). These regions, strategically positioned within Chennai's southern urban corridor, represent critical hubs of the city's rapid infrastructure growth and socioeconomic development.

4 METHODOLOGY

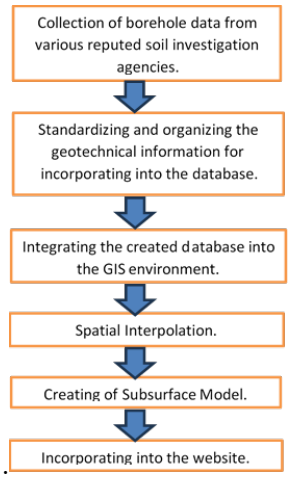
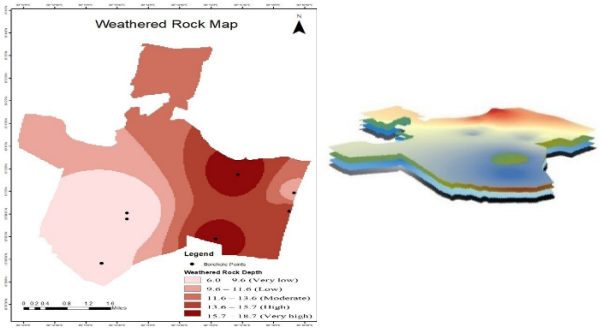


Fig. 1 Flowchart depicting the methodology

5 RESULTS AND DISCUSSIONS

The study employs MongoDB, a NoSQL database, to manage borehole data and user-defined queries due to its scalability and flexibility in handling geospatial datasets. ESRI ArcGIS 10.3 integrates with MongoDB to enable spatial analysis, mapping, and visualization of subsurface layers. Borehole datasets are interpolated using Inverse Distance Weighting (IDW), selected for its robustness in modeling sparse datasets and estimating soil properties across unsampled locations.

This method underpins the development of detailed 2D and 3D subsurface models (as shown in Fig.2), illustrating spatial variations in soil stratigraphy and composition. A web-based interface is designed to allow users to input location-specific queries, retrieve real-time borehole data, and visualize subsurface conditions interactively within a geospatial environment (as seen in Fig.3) By bridging database management, GIS analytics, and dynamic modeling, the system empowers urban planners and engineers to optimize site assessments, foundation designs, and resilience strategies, directly supporting sustainable infrastructure development in Chennai's high-growth zones.



a) Depth to weathered rock b) Soil layers map in 3D

Fig. 2 2D depth maps and 3D map of subsurface.



a) Web Interface b) 3D Representation of Borehole in the study area

Fig. 3 Web-based interface to retrieve real-time borehole data

6 CONCLUSION

The study establishes a robust Borehole Information System (BIS) integrating MongoDB and geospatial tools to model Chennai's heterogeneous subsurface, addressing critical urban planning challenges. By leveraging IDW interpolation and dynamic web interfaces, the system enables real-time visualization of soil stratigraphy, enhancing site-specific decision-making for infrastructure resilience. Thus, the project underscores the transformative role of integrated geospatial databases in fostering sustainable, climate-resilient urban development.

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Integration of 3D geological model and architectural model for design and construction of bored tunnel in Singapore

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ABSTRACT

This study focuses on the development of a three-dimensional (3D) geological model based on the exploratory borehole data and its integration with architectural model of the Riviera Interchange Station and Tunnels for the Cross Island Line Punggol Extension of Mass Rapid Transit (MRT) in Singapore. The integrated model was used during the design and construction planning stages to support decisions related to excavation volume estimation and tunnelling strategy. One key achievement is the ability to calculate the anticipated excavation volume by geology for each 1.5 to 2-metre ring of tunnel lining during shield tunnelling, which assisted in aligning excavation methods with ground conditions. However, challenges remain, particularly in addressing uncertainties inherent to 3D geological modelling. Due to the linear nature of tunnels, borehole investigations are typically aligned longitudinally, resulting in limited cross-sectional data and increased uncertainty in geological predictions perpendicular to the tunnel alignment. Future refinements may help improve the reliability of geological modelling and its application in tunnel construction planning.

Keywords: 3D geological model, 3D architectural model, underground railway, bored tunnel, tunnel boring machine (TBM)

1 INTRODUCTION

The construction of underground infrastructure in urban areas faces numerous challenges, such as limited space, complex ground conditions, and the need to minimise disruption to the existing environment. To address these challenges, the integration of three-dimensional (3D) geological models with architectural models has become increasingly important. In this study, we present the implementation of such an approach for the design and construction of a bored tunnel as part of the Cross Island Line Punggol Extension of Mass Rapid Transit (MRT) in Singapore. The objective is to demonstrate how digital modelling technologies can enhance planning and improve safety.

2 PROJECT DESCRIPTIONS

The Riviera Interchange Station and associated tunnel works form a key component of the Cross Island Line - Punggol Extension (P103), a new underground MRT line currently under development in Singapore. This project involves the construction of a new underground interchange station adjacent to the existing above-ground Riviera Station of the Light Rail Transit (LRT). The station box will be constructed at an approximate excavation depth of 35 metres below ground level. In addition, approximately 4.3 km of bored tunnels will be constructed. The bored tunnels will be excavated using Earth Pressure Balance (EPB) Tunnel Boring Machines (TBMs), with an outer diameter of approximately 12.7 metres. The tunnels will run at depths ranging from 7 to 34 metres below ground level (Yugami, 2023).

Given the highly urbanised surroundings and the complex subsoil profile, careful engineering design and construction are essential to minimise risks to nearby buildings, roads, and existing utility services.

3 GEOLOGICAL AND GEOTECHNICAL CONDITIONS

The project alignment is situated within the old river valleys of Singapore, characterised by marginal-marine, coastal, and fluvial sediments dating from the late Pleistocene to Holocene age, collectively known as the Kallang Group (Leslie et al., 2021). The Kallang Group represents the youngest stratigraphic unit in Singapore and unconformably overlies the irregular upper surface of the Bedok Formation. The Bedok Formation, predating the Pleistocene, comprises weakly consolidated to unconsolidated beds of gravel, sand, silt and clay.

The tunnelling depth occasionally coincides with the

alluvial sand layer of the Kallang Group, which is highly permeable and poses a potential risk of groundwater inflow and instability during construction. Additionally, at the interface between the Kallang Group and the Bedok Formation, the irregular surface of the Bedok Formation results in sudden variations in soil strength, increasing the risk of excavation difficulties, TBM performance disruptions, and tunnel instability.

4 METHODOLOGY

The 3D geological model was generated using Leapfrog Works software by Seequent. The model was developed using data from approximately 400 exploratory boreholes, each ranging from 40 to 50 metres in depth. Boreholes are spaced at intervals of 30 to 50 metres along the overall tunnel alignment and approximately 10 metres within station areas. Geological layers and geotechnical soil classifications were integrated into the model.

Subsequently, the architectural model created by the design and construction contractor was imported into the 3D geological model environment. Each tunnel ring, composed of tunnel segments spaced 1.5 to 2 metres, was imported for tunnel alignment. The 3D integrated model is shown in Fig. 1.

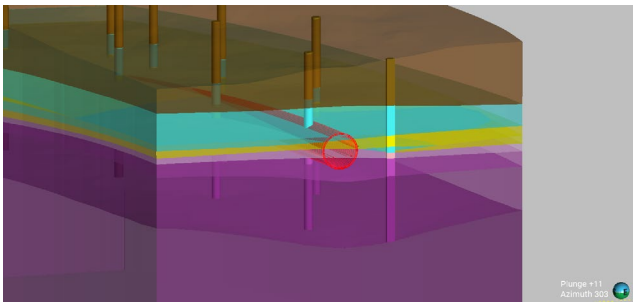


Fig. 1. Close-up of the 3D integrated model.

5 APPLICATION TO TUNNEL DESIGN AND CONSTRUCTION

The integrated model provided significant advantages in the planning of tunnel excavation. By calculating the expected excavation volume for each geological unit along 1.5 to 2.0 metre tunnel segments, it became possible to optimise shield tunnelling operations. The model also allowed visual comparison between underground structures and subsurface conditions. This helped engineers and geologists identify areas where ground conditions might affect construction and make informed adjustments during design and planning.

While the integrated model was useful, its accuracy is limited by uncertainties in geological interpretation, particularly where borehole data are sparse in directions perpendicular to the tunnel alignment. To manage this, it is important to record the sources of data, the assumptions made, and the level of confidence in each part of the model. Sharing this information with designers, contractors, and owners helps maintain a consistent understanding of the risks involved and supports practical decision-making during design and construction.

6 FUTURE POTENTIAL

The use of an integrated geological and architectural model could contribute to better coordination between design and construction teams. One possible improvement is the refinement of tunnel segment designs based on geological conditions predicted by the model. For example, segment types could be adapted in advance for areas with soft soils or stiffer soils or bedrock, which may improve efficiency and reduce construction risk. Further development of such applications requires improved geological prediction accuracy and closer collaboration among stakeholders involved in tunnelling projects.

7 CONCLUSIONS

The integration of 3D geological and architectural models provided a practical tool for coordinating design and construction planning for the bored tunnels of this MRT project. In particular, the ability to estimate excavation volumes by geological unit helped align tunnelling operations with expected ground conditions. While the model improved visibility into subsurface conditions, limitations remain due to the inherent uncertainties of geological prediction, especially where borehole data are sparse. Continued refinement of modelling techniques, including more comprehensive data collection and clearer representation of uncertainty, will be necessary to improve the reliability and usefulness of such models in future tunnelling projects.

ACKNOWLEDGEMENTS

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Snow Avalanche Hazard Zonation in the Indian Himalayas: A case study of Rudraprayag region, Uttarakhand, India

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ABSTRACT

Avalanches are one of the most common and deadly hazards in the Indian Himalayan Region (IHR), particularly in districts like Rudraprayag, where the convergence of high-altitude terrain, intense snowfall, and tectonic instability exists. In recent years, more and more cases of avalanches affecting infrastructure, pilgrimage routes, and communities have raised the immediate need for spatially explicit hazard zonation to guide risk management efforts. The study uses object-based image analysis (OBIA) for snow avalanche hazard zonation in the Rudraprayag district of Uttarakhand, the first of its kind for the region using such a high-resolution, terrain-sensitive approach.

Merging remote sensing and GIS techniques, OBIA aims to identify avalanche-prone zones by extracting terrain attributes from high-resolution DEMs. The ALOS PALSAR RT1 DEM with a spatial resolution of 12.5 m was used as the base dataset in this context. Using the DEM, the critical terrain parameters slope, aspect, curvature (plan), and vector ruggedness measure were generated. These features were chosen based on their known effect on avalanche initiation and runout.

OBIA was applied using a rule-based segmentation and classification system. First, areas with slopes between 30° and 45° were extracted, representing the optimal range for snowpack instability and avalanche initiation. Areas of Extreme Ruggedness (subclass VRM > 0.03) and solitary values of obscene planar curvatures that may be classified weirdly were eliminated to skip rock faces, ridges, and gulleys where a stable accumulation of snow is not likely. Besides, isolated avalanche areas smaller than 600 m2 were removed, which meets the best operational mapping practice.

Aspect slope plays an important role because generally, an area that faces north or leeward will preserve the snow for long periods and may be more likely to be unstable. By OBIA, the aspect layer was classified into cardinal directions and merged with slope and curvature classes. The aspect was given double weight than slope for multiresolution segmentation, since the aspect has greater influence on melt cycles and snowpack cohesion.

The Rudraprayag region is particularly complex, with elevation gradation varying from 800 m to above 7000 m and several glacial valleys forming important terrains, like Mandani and Kedarnath catchments. Field level observations and reports by various agencies, i.e. DGRE and local disaster authorities, were used to validate the hazard zones mapped through OBIA. The final avalanche hazard map indicates zones highly susceptible to concentration in the upper Kedarnath valley, surrounding the Bhairav glacier, and the Triyuginarayan-Mandal corridor, which are all seasonal traveller routes and areas of heavy infrastructure. The object-based method, unlike the classical pixel-based classifications, greatly reduces noise, delineates the avalanche release and runout zones, and improves their integration with thematic planning layers like road networks, settlements, and pilgrimage routes. Such spatial precision becomes a must in high-altitude settings, where complex terrain can result in misguiding pixel-based models.

The study highlights the ability of OBIA to be a scalable or replicable method for avalanche hazard assessment in the Himalayas. By accurately delineating possible avalanche zones in Rudraprayag, it provides essential inputs to planners, engineers, and emergency managers. The products can support route planning for the Char Dham pilgrimage, infrastructure layout (e.g., helipads, roads), and prioritization of early warning systems in high-risk locations. In addition, the approach conforms to the national perspective of the Sendai Framework for Disaster Risk Reduction and Sustainable Development Goal (SDG) 11—making cities and human settlements inclusive, safe, resilient, and sustainable.

In summary, this study establishes a powerful, data-driven framework for snow avalanche hazard zonation in Rudraprayag through OBIA and high-resolution DEM analysis. This can be applied in many more vulnerable districts falling in the IHR, thereby contributing to the proactive mitigation of disasters and resilience-building in mountainous terrans that are prone to cryospherical hazards

TC-305 Geotechnical Infrastructure for Megacities and New Capitals

Results of numerical modeling of ground improvement with rigid inclusions as dry concrete columns

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ABSTRACT

Ground improvement using vertical elements, such as rigid inclusions, is widely applied worldwide. These inclusions are commonly used in soft soils to reduce settlements, providing a cost-effective alternative to traditional pile foundations. One such method involves dry concrete columns (DCC). However, a standardized methodology for settlement calculation in this approach has not yet been developed. Therefore, this study aims to establish a calculation framework using numerical modeling based on the finite element method. The numerical model was validated against results from full-scale static load tests. Using the validated model, the influence of DCC parameters on settlement was analyzed. The results indicate that the length and spacing of DCC, as well as the initial soil parameters, significantly influence the final outcome and the overall effectiveness of the technology. The settlement can be reduced by 44%, depending on the depth and spacing of the inclusions.

Keywords: numerical modeling, finite element method, settlement, ground improvement, dry concrete columns

1 INTRODUCTION

One of the effective methods for ground improvement is the application of vertical elements such as dry concrete columns (DCC), which have been implemented in Belarus (Nikitenko and Tronda 2014, Sernov and Tronda 2019, Tronda 2016). However, existing calculation methods show significant deviation from field test results (Tronda 2023).

The objective of the study is to determine the influence of DCC parameters on the settlement of foundations. To achieve this, numerical modeling was carried out.

The work was conducted in two stages, such as model validation and numerical study.

2 MODEL VALIDATION

2.1 Conditions

The numerical model was validated against the results of full-scale plate load tests (Tronda 2023). The test sites are characterized by soft silty-clayey soils with low deformation properties.

2.2 Models

To describe reinforced soil behavior, the following constitutive models were considered: Mohr-Coulomb (MC), Hardening Soil (HS), and Hardening Soil Small-Strain (HSS). Model parameters were determined based on laboratory and in-situ soil tests, as well as established empirical correlations (Hardin and Drnevich 1972, Mayne and Kemper 1988, Sharafutdinov 2022 and 2023).

DCC were modeled using two approaches: Volume Elements (VE) and Embedded Beams (EB).

3 NUMERICAL STUDY

3.1 Conditions

A reinforced concrete slab foundation was adopted for a frame building. The structural stiffness of the building was modeled in accordance with (Shulyatyev 2013, SP 22.13330.2016).

The soil base was assumed to consist of three layers, such as the sand transfer layer, soft silty-clayey soil and sandy soil.

3.2 Variations

In the course of the study, the following DCC parameters were varied: the length L = 0…15 m and the spacing a = 1.0 ($3d$)...2.0 ($6d$) m, where d is the diameter of the DCC. Additionally, the deformation modulus of the improved soil E_{oed}^{ref} = 5…12 MPa, as well as its thickness h = 5…15 m, were varied.

4 RESULTS

4.1 Model validation

The soil model was selected and justified by comparing ‘pressure–settlement’ curves obtained from numerical simulations and plate load tests conducted at experimental sites. The results of one of these tests are presented in Fig. 1.

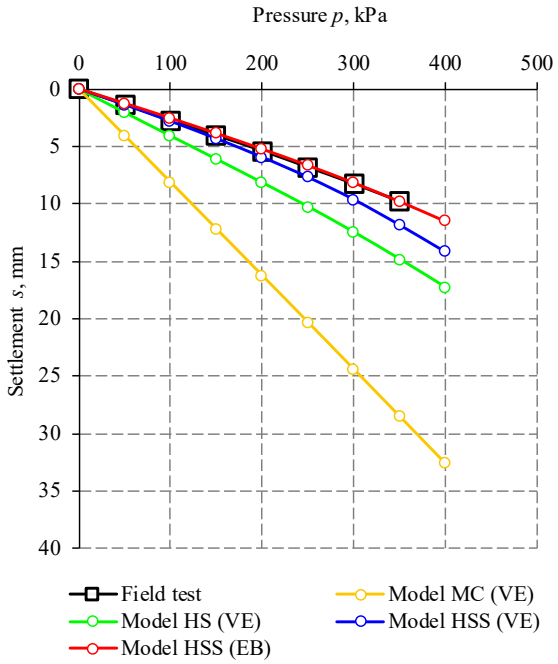


Fig. 1. 'Pressure-settlement' curves based on field test results and numerical simulations.

The MC model significantly overestimates settlement values and does not adequately capture the deformation behavior. Similarly, the HS model also overestimates deformations. The best agreement with field test results is achieved when modeling the soil mass using the HSS model. DCC can be accurately modeled using either the VE or EB approach.

4.2 Numerical study

The influence of DCC parameters on foundation deformability was assessed using the relative settlement ratio s_{imp}/s, where s_{imp} is the average absolute settlement of the foundation on a reinforced base, and s is the average absolute settlement of the foundation on an unreinforced base (Fig. 2).

The effectiveness of the DCC technology is higher for soils with lower stiffness. For instance, in conventionally soft soils with $E_{oed}^{ref} = 5$ MPa, settlement is reduced by 5…44% depending on the depth and spacing of the inclusions. In soils with $E_{oed}^{ref} = 7…12$ MPa, settlement reduction reaches 18%.

As the spacing between DCC increases, the effectiveness of the technology decreases. At a spacing of 6d, the reduction in settlement ranges 5…19%.

5 CONCLUSIONS

The best agreement with field test results is achieved when modeling the soil mass using the HSS model.

The effectiveness of the DCC technology is greater for soils with low deformation properties. For highly compressible soils, settlement can be reduced by 44%, depending on the depth and spacing of the inclusions.

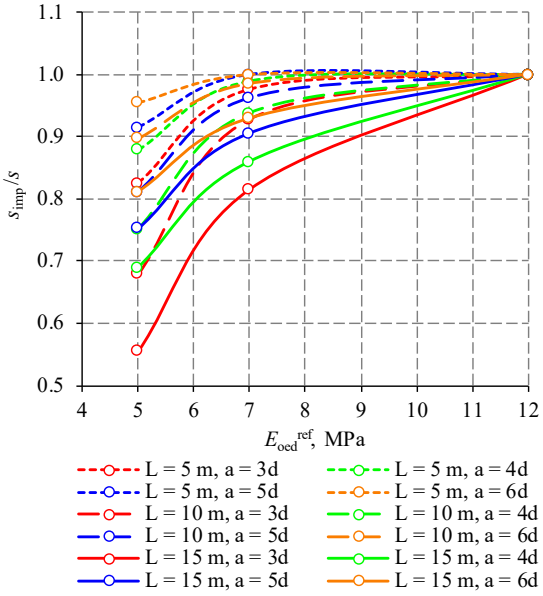


Fig. 2. Relative settlements s_{imp}/s at $p = 400$ kPa for the deformation modulus E_{oed}^{ref}.

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Dynamic stability evaluation of underground structures considering back analysis of rock mechanics parameters: a case study of the Oya tuff quarry

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1 INTRODUCTION

The abandoned Oya tuff underground quarry in Utsunomiya, Japan (Fig. 1-2), faces critical seismic stability challenges due to its room-and-pillar architecture, high porosity, and location in a seismically active zone. Historical collapses (e.g., 1991 incident) and water seepage have compromised 70% of its 200+ chambers, some beneath residential areas. Conventional methods fail to address dynamic interactions between low-tensile-strength tuff and seismic waves. This study bridges this gap by identifying rock mechanics parameter back analysis with 3D dynamic simulation to evaluate stability under seismic loading (2024 Ibaraki M5.3 earthquake).



Fig. 1. Collapse at the Oya tuff underground quarry site (April 29, 1991) (Geospatial Information Authority of Japan, 2025)



Fig. 2. Internal view of abandoned Oya tuff underground quarry.

2 METHODOLOGY

2.1 Inversion method of rock mass parameters

Based on the previous studies (Huang R. et al. 2024), a 3D finite element model, i.e. 20m×20m, is established. The core process of identifying rock material parameters with the proposed inversion method is as follows:

1) Objective function construction and optimization framework.

Define the displacement time-range error squared integral objective function, Equation (1):

$$J(\mathbf{P}) = \frac{1}{2}\int_0^t \sum_{i=1}^{n}\{u_i(\mathbf{P},t) - u_i^*(t)\}^2 dt \qquad (1)$$

The vector $\mathbf{P} = (E_1,\ E_2,\ldots,\ E_m\ ,\ \alpha,\ \beta)^{\mathrm{T}}$ contains the unknown elastic modulus for mth layer, E_m and Rayleigh damping coefficients α and β. $u_i(\mathbf{P},t)$ and $u_i^*(t)$ represent the calculated and the measured displacement–time t histories at the observation points of number n, respectively.

The Gauss-Newton-like iterative algorithm is used to solve this nonlinear optimization problem to derive Equation (2):

$$\mathbf{A}d\mathbf{P} = \mathbf{b} \qquad (2)$$

where $\mathbf{A}$ is the sensitivity matrix and $\mathbf{b}$ is the error vector, which is stabilized by truncated singular value decomposition to solve for the increment parameter vector $d\mathbf{P}$.

2) Sensitivity Computational System

A sensitivity differential equation is developed based on the derivation of the parameters from the dynamical control equations.

3) Rayleigh damping coupling identification

Initial values of damping coefficients are determined by eigenvalue analysis, α, β and elastic parameters are optimized synchronously in iterations to characterize frequency-varying energy dissipation properties through critical damping ratios.

This inversion framework realizes the simultaneous identification of elastic modulus and damping properties of rock mass, and provides theoretical support for the estimation of geological material parameters.

2.2 Numerical Model

A 3D numerical model of the underground quarry of Oya tuff was constructed based on FLAC3D with a size of 500 m × 500 m × 100 m (L × W × H), covering both the vertical shafts and the horizontal roadway system (Fig. 3). The model is meshed non-uniformly: the overall maximum zone size is 50 m × 50 m × 10 m, and critical

excavation areas are encoded to 2 m × 2 m × 2 m to accurately capture potential damage. The rock mass is simplified to a homogeneous material, with a depth of 40 m, and the overburden pressure is used as the vertical principal stress boundary. The model provides a geometric basis for dynamic stability analysis by presenting multiple views, i.e. overall configuration, excavation details, pillar layout, etc.

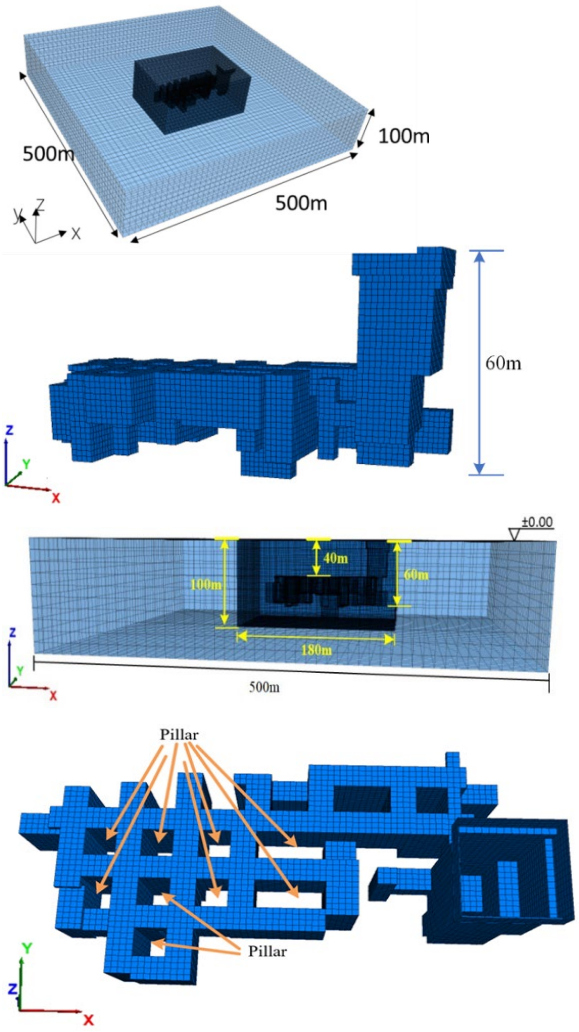


Fig. 3. FLAC3D model: (a) overall model configuration; (b) detailed view of excavated areas; (c) elevation view of the overall model; (d) elevation view of the excavation area; (e) location layout of some pillars.

The model adopts perfect elasto-plastic state following the Mohr-Coulomb yield criterion, those parameters are shown in Table 1.

Table 1. Physical and mechanical parameters of Oya tuff.

Property	Value
Bulk modulus K (GPa)	2.8652
Shear modulus G (GPa)	1.7191
Density ρ (kg/m^3)	1770
Poisson's ratio ν	0.23
Cohesion c (MPa)	2.10
Tensile strength σ_t (MPa)	1.08
α (1/s)	29.254
β (s) ($\times 10^{-4}$)	3.576

3 DYNAMIC RESPONSE ANALYSIS

Numerical simulation results show that the seismic wave input area (excavation floor) exhibits the most significant displacement response, and the cumulative effect of its plastic deformation leads to the displacement continuously exceeding the displacement of the top plate; at the same time, a persistent stress concentration area is formed at the corner of the pillar base, which presents the typical characteristics of high shear stress distribution.

It is worth noting that the central area of the top slab maintains a relatively stable dynamic response, while the excavated bottom slab shows a strong multidirectional dynamic amplification effect, and this difference in the local dynamic characteristics leads to the bottom slab becoming a key area where the risk of instability accumulates. The coupling of the above dynamic responses and damage modes suggests that stress concentration in the geometrically abrupt areas (e.g., corners of pillar bases) and the multidirectional vibration amplification effect of the footing plate together constitute the core mechanism of seismic instability of underground structures.

5 CONCLUSION

In this study, the seismic response characteristics and destabilization mechanism of the underground quarry of Oya tuff are revealed through inversion of rock mass parameters and three-dimensional numerical simulation. The results show that the displacement response of the excavation base plate is the most significant under seismic loading, the persistent stress concentration exists at the corners of the pillar base, and the geometry and seismic wave characteristics jointly control the deformation pattern and stress redistribution. The study emphasizes the need to incorporate dynamic conditions into the safety assessment system and to improve the seismic performance of similar underground structures through continuous monitoring and strengthening.

ACKNOWLEDGEMENTS

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Patterns of Activation and Manifestation of Landslide Processes on the Foothill Slopes of the Zerafshan Oasis, Uzbekistan

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ABSTRACT

In the Zarafshan Oasis of the Republic of Uzbekistan, mountainous and foothill slopes are widely developed. The movement of soil masses near populated areas and various socio-economic facilities currently poses an emergency situation.

The broad distribution and distinct manifestation of landslides and other hazardous geological processes in the mountainous and foothill regions complicate territorial development. These processes have intensified significantly over the past 20–25 years, due not only to natural factors but also to increased human intervention in the natural environment.

Keywords: landslides, Zarafshan, geohazards, monitoring, loess soil, Uzbekistan

1. INTRODUCTION

Continuous monitoring of landslides has been conducted for over 25 years in cooperation with the State Service of the Samarkand Station for Monitoring Hazardous Geological Processes. Instability of soil masses and hazardous geological phenomena (landslides, soil flows, rockfalls, etc.) are actively developing in the mountainous and foothill zones of Uzbekistan.

The primary cause of landslide initiation and development is soil oversaturation due to heavy precipitation, contact with bedrock, and both surface (springs) and groundwater, as well as landslides triggered by frequent seismic activity.

Currently, monitoring is being conducted over more than 25 active landslide-prone and 63 potential minor and major landslide sites. Additionally, nine particularly hazardous landslides in the Urgut District of Samarkand Region—villages: Sayguss, Upper Guss, Lower Guss, Upper Aktash, Lower Aktash, Khojaabdall, and Oltykhauz—remain under constant observation. These major landslides were initially triggered by a forced explosion on November 13, 1994, and they continue to remain active and hazardous.

2. RECENT EVENTS AND CONDITIONS

In Zaamin, a major landslide occurred on highway section 73-4 Zaamin–Bakhmal, fortunately without casualties. On August 27, 2024, a landslide in the village of Yettikechu reached the walls of School No. 23, causing the suspension of classes. There is an ongoing risk of further mass movement. In Mullabulak (Jizzakh Region) and on the Almaty–Bishkek–Tashkent–Termez highway, as well as on the Kitab–Samarkand mountain pass (Takhta-Karacha Pass), a landslide occurred on November 6, 2024, causing the road to close due to the fall of a massive boulder, approximately 50 cubic meters in volume, onto the roadway. The boulder crushed a vehicle with passengers, resulting in three fatalities.

Long-term observations indicate that heavy precipitation and high seismic activity play a significant role in the formation of landslides and other hazardous geological processes.

Landslides develop most intensively along highways and roadways. This leads to the conclusion that development of mountainous and foothill territories

without prior geotechnical investigations to assess slope stability and implement protective recommendations by specialists, designers, and experts from the "Station for Monitoring Hazardous Geological Processes" service is one of the key contributors to landslide formation.

3. GEOLOGICAL CONDITIONS AND SEASONAL PATTERNS

The development of these hazardous geological processes is also associated with widespread distribution of loess soils (notably collapsible loess), which, under the influence of moisture, rapidly deform and transition into a flow-like state.

The dynamics of landslide manifestation depend significantly on meteorological conditions, particularly during the preceding autumn (October–November) and spring (March–April to mid-May) seasons. During the transition from winter to spring, the amount of autumn-winter precipitation is critical, as it leads to gradual and prolonged moisture infiltration in slope soils, thereby increasing landslide activity—especially of the superficial type—in late February through April.

Based on the analysis of regional patterns of landslide activation, the following conclusions can be drawn:

— The activation of large-scale landslides is most prominent from March to mid-May in zones with clayey soils;

— Moisture conditions are influenced by the temperature regime at the slope surface;

— Most snow reserves are depleted through surface runoff. Even increases in monthly precipitation above long-term averages do not necessarily activate landslides on disturbed slopes.

4. HUMAN INFLUENCE AND MITIGATION CHALLENGES

The results of 25 years of observations clearly demonstrate the substantial impact of human activity on landslide development. Analysis of recorded landslides shows that over 50% occurred due to the construction and operation of various structures.

To address the global challenges arising from landslides, it is essential to correctly assess the causes of landslide development and to effectively design and implement engineering protection methods, including the construction of anti-landslide structures.

Current practices in landslide mitigation vividly reflect the erroneous interpretation of causes and solutions, often overlooking natural factors, which remain outside the focus of researchers and engineers.

Seismic resistance of continuous monolithic bridges, accounting for post-tensioning cable tension during the interaction between support foundations and soil

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ABSTRACT

This paper examines the features of calculating the seismic resistance of a continuous monolithic highway overpass. The study focuses on a 120-meter-long three-span continuous monolithic highway overpass located in an 8-point seismic intensity zone in the city of Jizzakh, above the Tashkent-Samarkand high-speed railway line (Republic of Uzbekistan). The results of calculations for changes in normal stress in the upper and lower parts of the span structure elements of the continuous monolithic overpass, performed using the SHARK and MIDAS Civil software packages, are presented. As a result of the numerical calculation of the seismic resistance of the continuous monolithic overpass, taking into account the interaction between the soil and the foundation of bridge structure, it was revealed that the overpass retains its operational characteristics during an earthquake with an intensity of 8 and 9 points.

Keywords: monolithic continuous overpass, reinforced concrete, span structure, soil structure interaction, real earthquake records.

1. INTRODUCTION

An earthquake is an underground tremor and vibration of the Earth's surface that occurs as a result of sudden displacements and ruptures in the Earth's crust or upper mantle and is transmitted over long distances in the form of elastic vibrations. The problem of ensuring the seismic resistance of transport structures in the territory of Uzbekistan is very relevant [1], and special attention is being paid to the construction of transport structures, including bridges, overpasses, and overpasses.

2. RESEARCH MATERIALS AND METHODS

In this article, we will consider a three-span monolithic reinforced concrete overpass located in the city of Jizzakh, with a length of 120 m and a width of 21 m, the span structure has variable thickness along the overpass, designed as a continuous monolithic reinforced concrete calculation scheme of individual design 35+50+35m. The intermediate supports are two-pronged, having dimensions: height 12m, width along the facade - 2m, and along the side direction have variable height sizes from 3m to 5m.

The tension efforts of high-strength reinforcement are carried out by applying a cable tension force up to 4000 kN. The number of cabel canals are 19 pieces and each cabel canal is made of 31 ropes and is arranged in a polygonal pattern, which have high-strength reinforcement is made of seven-wire twisted reinforcement ropes d=7.8 mm made of steel.

The seismicity of the Jizzakh city territory, according to the seismic microzoning map completed by the Institute of Seismology AS of RUz in 2017, is assessed at 7, 8 points. The designated construction site is located in an 8-point zone.

According to KMK 2.01.03-19, the category of soils by seismic properties is III (third) - loess-shaped loams with sandy layers with a porosity coefficient of e<0.8) [2]. Taking this into account, it is recommended to take the seismicity of the designed construction site as 8 points.

3. RESEARCH RESULTS

The calculation of the overpass for static and dynamic loads is performed using two software complexes: MIDAS Civil and SHARK (Step-by-Step Design Calculation Algorithms) [3], developed by Professor I. Mirzaev, based on real earthquake records.

The results of calculations of the monolithic overpass from the dynamic load, according to the records of real seismograms of the Gazli earthquake of 17.05.1976 (Uzbekistan), more than 9 points on the MSK-64 scale, maximum acceleration, speed and displacement in the direction, longitudinal axis of the bridge, propagation of seismic waves: maximum horizontal and vertical accelerations are 7.22 m/s2 and 13.163 m/s2, velocities are 0.62 m/s and 0.57 m/s, displacements are 0.18 m and 0.216664 m, digitizing step is 0.005 s, duration is 28 s. Earthquake records are taken from the European Database of Strong Earthquakes. For discretization, the overpass was divided into 278 finite elements, taking into account the work of each type of end element. The calculations were performed according to an implicit scheme with a time step of 0.001 s. Energy loss is accounted for in the Rayleigh form.

Figure 1 shows the graphs of changes in normal stress over time in the upper (a) and lower (b) sections of the span structure in the middle of a monolithic overpass (Gazli earthquake).

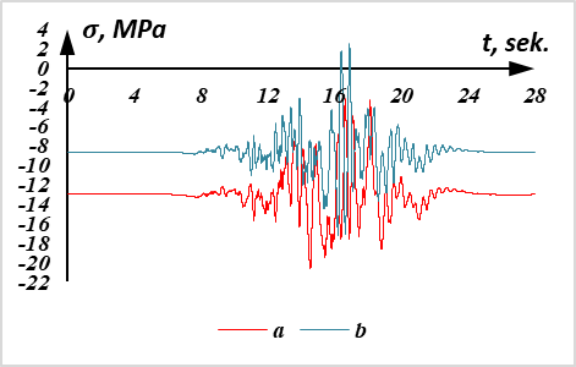


Fig. 1. Change in normal stress over time in the upper (a) and lower (b) parts of the span structure in the middle of the overpass (Gazli earthquake)

According to the calculation results of the SHARK software complex, the maximum compressive stress in the upper part of the span structure during an earthquake is -21.17 MPa, and the maximum tensile stress in the lower part is 2.2516 MPa. According to the calculation results of the continuous monolithic overpass in the MIDAS Civil software package for 8 intensity by MSK-64, the maximum compressive stress in the upper part of the span structure is -10.10 MPa, and the maximum tensile stress in the lower part is 2.32 MPa.

In Table. 1 comparative analysis of the results of calculations for the change in normal stress in the upper and lower parts of the span structure elements of a continuous monolithic overpass in the SHARK and MIDAS Civil software packages is presented.

Table 1. Maximum stress values in the span structure in the upper (a) and lower (b) parts in the middle of the overpass

No.	Software	Maximum stress in the upper part (compressive), MPa	Maximum stress in the lower part (stretching), MPa	Maximum allowable compressive/stretching stress according to SHNK 2.05.03-2022, MPa
1.	SHARK	-21.17	2.25	-50.4 / 3.22
2.	MIDAS Civil	-10.10	2.32	-50.4 / 3.22

4. CONCLUSION

The results of normal stress calculations in the upper and lower parts of the span structure along the length of the monolithic overpass were analyzed taking into account soil-structure interaction. Numerical calculations have shown that using the spectral calculation method (MIDAS Civil) and dynamic calculation based on real earthquake records (SHARK) in absolute stiffness soil, the overpass maintains its operational characteristics and ensures its seismic resistance. Reducing stiffness of soil to the increasing of stresses in the span structure of overpass.

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Influence of fibre and carbon negative material on the stability of infinite slopes

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ABSTRACT

Slope instability poses a critical geotechnical engineering challenge in debris flow-prone areas, especially infinite slopes. Conventional stabilisation methods are often costly and environmentally damaging; thus, simpler and more environmentally friendly solutions are needed. Soil improvement using sustainable materials such as biochar and fibre offers a promising approach for slope stabilisation. Biochar is a porous, carbon negative material (i.e., biochar) that can enhance soil cohesion, while fibre is also very effective in composite soil improvement. However, there are limited studies on their combined effects. Therefore, this study investigates the combined effects of biochar and fibre as a sustainable slope reinforcement approach. A two-layer infinite slope model was numerically analysed using the limit equilibrium method, considering different biochar contents and fibre contents. The results show that biochar and fibre reinforcement significantly improve slope stability, with FoS increasing from 2.955 (unreinforced) to a peak of 4.548 at 8% biochar.

Keywords: slope stability, biochar, fibre, factor of safety, infinite slope, soil amendments

1 INTRODUCTION

Slope stability remains a significant challenge in geotechnical engineering, especially in areas prone to debris or mudflows. Infinite slopes, characterized by uniform slopes and shallow failure surfaces, are particularly susceptible to failure under hydrogeological stresses. Traditional stabilization methods are often costly and cause significant environmental damage.[1]. These solutions may not be feasible in rural or ecologically sensitive areas, so there is a need to explore sustainable, nature-based alternatives.

In recent years, biochar (a carbon-negative material) has been widely studied as a soil amendment for improving geotechnical properties [2]. On the other hand, adding natural or synthetic fibres to soil is well known to improve soil strength. Fibres reinforce soils by increasing tensile strength, providing mechanical interlocking, and enhancing ductility.

Although biochar and fibre contribute individually to slope stability, their combined effects remain relatively less explored. Understanding how these two approaches interact is critical to developing optimised, sustainable slope reinforcement strategies.

This study uses a two-layer infinite slope model to evaluate the stability of infinite slopes reinforced with different combinations of biochar content (2%, 4%, 6%, 8%, and 10%) and fibre content (0.1%, 0.2%, 0.3%, and 0.4%). The main objective of this study is to determine how biochar and fibre affect the factor of safety (FoS) of an infinite slope.

2 METHODOLOGY

2.1 Infinite Slope Model

The present study assumed a two-layer soil profile for the analysis. A stability analysis was performed using the Limit Equilibrium Method (LEM). Two analytical formulations were utilised to compute the Factor of Safety (FoS), including Taylor's method from 1948. For simplified, single-layer slope conditions, the

FoS was calculated using Taylor's infinite slope equation:

$$FoS=\frac{c'+(\gamma h\cos^2\beta-u)\tan\phi'}{\gamma h\sin\beta\cos\beta}$$

Where, c′: Effective cohesion (kPa), ϕ′: Effective internal friction angle (°), γ: Unit weight of soil (kN/m³), h: Depth to the potential failure surface (m), β: Slope angle (°), u: Pore water pressure (kPa)

2.2 Soil and reinforcement parameters

In this study, the soil properties were sourced from the literature [3]. The Chongzuo expansive soil is classified as a high liquid limit clay, with a liquid limit of 72.4%, a plastic limit of 23.1%, and a plasticity index of 49.3%. The soil has an optimal moisture content of 21.5% and a maximum dry density of 1.60 Mg/m³.

2.3 Hydrological Conditions

To incorporate the effect of moisture, this study applied a pore pressure ratio (r_u) approach.

The pore pressure ratio is defined as:

$$r_u=\frac{u}{\gamma h}$$

Where, u: Pore water pressure (kPa), γ: Unit weight of soil (kN/m³), h: Depth to the failure surface (m)

In this analysis, FoS was calculated using a representative value of r_u=0.3, simulating partially saturated field conditions, such as moderate rainfall or vegetated slopes with some drainage. With a unit weight γ=18 kN/m3 and failure depth h=1m. Consequently, a pore water pressure of u=5.4 kPa was applied uniformly in all reinforcement scenarios.

3. RESULTS

3.1 Effect of Biochar Amendment

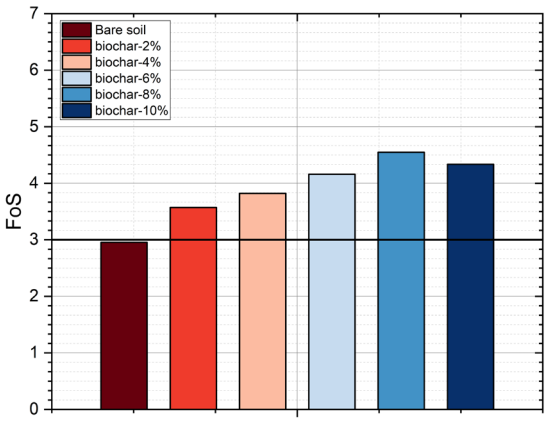


Figure 1: Factor of Safety (FoS) of infinite slope for different biochar content

The influence of biochar on the factor of safety (FoS) is depicted in Figure 1. Initially, the FoS for the untreated slope (0% biochar) was assessed at 2.95. Notably, the slope's stability improves significantly as the biochar concentration increases. At a biochar content of 2%, the FoS increases to 3.571, followed by a further rise to 3.822 at 4% biochar. The upward trend continues, with the FoS reaching 4.159 at 6% biochar and peaking at 4.548 when the biochar content reaches 8%. However, at 10% biochar, the FoS experiences a slight decline, dropping to 4.336.

3.2 Effect of Fibre Reinforcement

The combined impact of biochar and fibre is illustrated in Figure 2, demonstrating a significant positive influence on slope stability. The FoS values for 8% of biochar combined with varying fibre contents are as follows: 4.859 for 0.1% fibre, 5.200 for 0.2% fibre, 5.541 for 0.3% fibre, and 5.257 for 0.4% fibre. The highest FoS was observed at 0.3% fibre content, indicating an optimal range where fibres enhance soil cohesion and shear resistance.

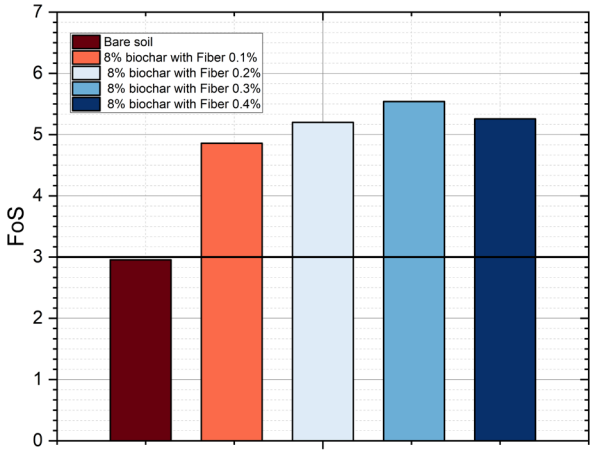


Figure 2: Factor of Safety (FoS) of infinite slope for different biochar and fiber contents

4. CONCLUSION

This study explored the effects of biochar and fibre reinforcement on the stability of infinite slopes using a simple two-layer infinite slope model under a representative pore pressure ratio (ru = 0.3). Results showed that adding biochar enhanced the Factor of Safety (FoS), with values rising from 2.955 (0%) to a peak of 4.548 at 8% biochar. Fibre reinforcement proved even more effective, with the highest FoS of 5.541 observed at 0.3% fibre, an 87.5% improvement over the unreinforced slope. A slight decrease at 0.4% fibre suggests an optimal content beyond which effectiveness diminishes. The findings highlight that biochar–fibre combinations offer a promising, sustainable alternative to enhance slope stability.

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TC-305 Geotechnical Infrastructure for Megacities and New Capitals

The concept of employing monitoring systems on roadways and railways situated in permafrost regions, with consideration of potential climate change

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ABSTRACT

This article introduces the concept of utilizing monitoring systems on roads and railways located in permafrost soil regions. It emphasizes the necessity of transitioning from the notion of "monitoring" to "automated analytical systems" to comprehensively fulfill the safety and reliability requirements as mandated by the Federal Law "Technical Regulations on the Safety of Buildings and Structures" dated December 30, 2009, N 384-FZ. For the proposed automated monitoring system, a specific array of equipment is recommended, which includes strain gauges, accelerometers, inclinometers, short-base and long-base displacement sensors of the leash type, temperature sensors, weather stations, and additional technical means, complemented by controllers for data collection and processing. An illustrative example of transport infrastructure facilities located on the Yamal Peninsula, encompassing both railway and road sections, demonstrates the execution of a monitoring system designed to identify pre-emergency and emergency situations, assess potential threats, address systemic issues, and analyze trends in anomaly development, with consideration for potential climate change influences. Within these facilities, scientific support is augmented by the automated monitoring system, incorporating diagnostic results obtained through unmanned aerial vehicles (UAVs) utilizing airborne laser scanning (ALS) technology, along with contributions from a mobile automobile laboratory (MAL). The article delineates the outcomes of VLS and MAL processing through information modeling technologies, investigates the effects of climate change on the surveyed locations, and provides recommendations for mitigating the impact of climate change.

Keywords: monitoring, automated analytical systems, permafrost, transport infrastructure, climate change, diagnostic, laser scanning

1 INTRODUCTION

Drawing on international experience, monitoring constitutes a cultural aspect of facility operation—a tool that enables comprehensive lifecycle management of infrastructure while providing critical data on the effectiveness of construction technologies specified in design documentation, as well as maintenance and repair strategies.

Monitoring is essential to address several critical objectives:

- Validation and calibration of the design model;
- Oversight of construction processes and external factor impacts during implementation;
- Assessment of the commissioning phase;
- Development of an operational model;
- Quantitative parameter evaluation for optimized maintenance planning.

These parameters are vital for ensuring quality control in construction and installation works, validating or adjusting planned measures, and enhancing collaboration across all project phases among stakeholders.

2 ADVERSE NATURAL PROCESSES, PHENOMENA, AND THEIR IMPACTS

A fundamental challenge in design and construction is the impossibility of fully accounting for all natural and anthropogenic parameters inherent to each climatic and geographic zone. Climate emerges as the primary driver of requirements for construction and operational technologies, determining facility behavior, wear rates, and performance beyond purely calculated technical

parameters.

Analysis of transport infrastructure projects planned under Russia's Transport Strategy to 2030 (extended to 2035) and the Unified List of Strategic Settlements highlights the necessity of monitoring facilities located in permafrost zones prone to adverse natural processes. Climate change scenarios must be rigorously incorporated into such evaluations.

3 TYPES OF MONITORING

Monitoring systems are often siloed, assessing individual components without integration. However, transitioning to digital twins demands holistic approaches.

Current monitoring methodologies include::

- Borehole instrumentation: Inclinometers, extensometers, and piezometers for subsurface soil dynamics;
- Thermal cables for frost depth and ground stability assessment;
- Distributed fiber-optic systems based on Mandelstam-Brillouin scattering;
- Terrestrial geodetic surveys;
- UAV topographic mapping;
- Seismographs for high-seismic zones.

Only when unified under expert analysis do these tools form a robust diagnostic framework.

4 TRANSPORT INFRASTRUCTURE MONITORING

Priority for advanced monitoring integration lies with linear infrastructure systems: roads, railways, and pipelines. These require distributed monitoring networks supplemented by subsystem-level sensors for high-risk segments, enabling granular operational insights.

5 CONCLUSIONS

Integrating technologies, evaluation methods, stakeholders, and theory with practice reveals monitoring as the critical nexus for lifecycle management. It facilitates digitization, modeling, and forecasting of design, construction, and operational methodologies.

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Underpinning and deep foundations load transfer of a ten-storey building in Lisbon

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ABSTRACT

The aim of this paper is to present the underpinning and load transfer solutions adopted at ten-storey 60 years old buildings, in Lisbon, to allow the construction of the Lisbon Metro line enlargement, new circular line between Santos to Cais do Sodré stations, very close to the Tagus River right bank. The buildings foundations, reinforced concrete bored piles, were demolished and the loads transferred, through a reinforced concrete underpin stiff slab to the sideways double row jet grouting columns, reinforced with steel tubes, with the following main functions: tunnel earth retaining walls, compatible with a cut and cover solution, and building new foundations. Most of the jet grouting columns integrating the lateral earth retaining walls were performed inside the building, with low head room. The main design and execution criteria are presented, including the BIM model, as well as some real-time monitoring results, crucial to assisting the load transfer operations using hydraulic jacks. The complexity of the advanced monitoring system is also pointed out, comprising robotic measurements, IoT sensors, a web platform and real-time processing.

Keywords: underpinning, foundations, BIM, monitoring

1 INTRODUCTION

The new Lisbon circular Metro line crossed a densely urbanized part of the city, connecting Rato station, located at one of the hills of the city, to Cais do Sodré station, at the Tagus River right bank. Where the tunnel is closer to the river and its cover becomes lower, the cut & cover method was used, the Metro line intersected a piled foundation of nine upper floors and one basement reinforced concrete buildings (nº42 and 44 from D. Carlos I Avenue), demanding the underpinning of those structures (Figure 1).

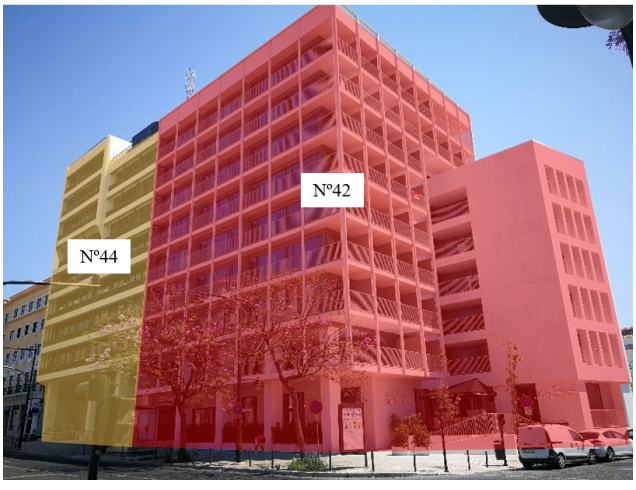


Fig. 1. Reinforced concrete underpinned buildings.

2 MAIN RESTRAINTS

2.1 Geotechnical restraints

The works took place close to the Tagus River, intersecting heterogeneous landfills and progressive thickness increase of sandy alluvial materials in direction of Tagus River side. Those layers rest over the Miocene bed rock, overlying the units of the Lisbon Volcanic Complex (Figure 2).

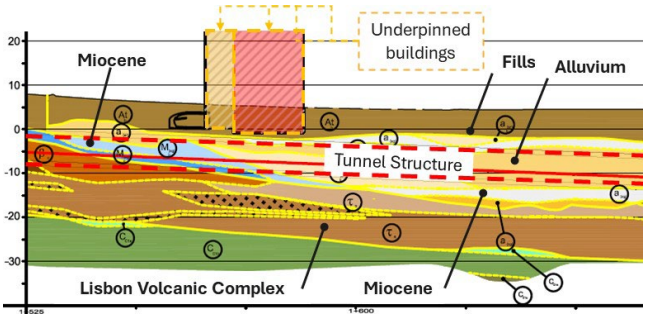


Fig. 2. Geological profile (Tagus River at the right-hand side).

2.2 Construction restraints

The adopted solutions had to respect the local constraints, mainly the accessibility of equipment and urban impact. Considering the need to work inside the building basement, the solutions had to be compatible with equipment that would be able to operate with a minimal headroom of about 3,0m.

3 ADOPTED SOLUTIONS

The adopted solution can be divided in excavation and underpinning (Figure 3 and Figure 4).

3.1 Excavation solution

The excavation cut & cover solution integrated a double row jet-grouting curtain wall with 1000mm diameter columns, spaced 700mm apart, reinforced with tubular steel profiles, with a triple function: foundation of the underpin slab, retaining wall and water inflow control into the excavation pit.

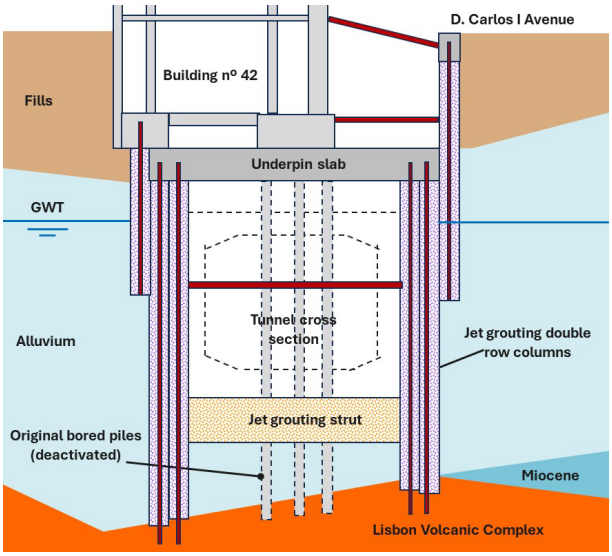


Fig. 3. Excavation typical cross section.

3.2 Underpinning solution

The underpinning solution integrated a stiff reinforced concrete underpin slab, not prestressed, resting over the double row jet-grouting columns wall. The underpin slab, located below the existing pile caps and above the new tunnel cross section, bridges a span of 13,0m, with a variable thickness between 1,40m and 1,80m. This solution allowed the load transfer from the original foundation bored piles to the underpin slab.

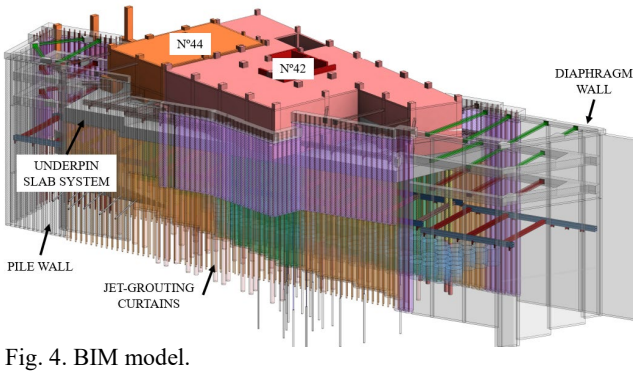


Fig. 4. BIM model.

4 REAL TIME MONITORING

To address the stated challenges, it was mandatory the implementation of an advanced monitoring system comprising robotic measurements, IoT sensors, a web platform during the load transfer operations, including real-time processing to support engineering decision related to hydraulic jacks operation for displacements compensation. Figure 5 identifies the monitoring devices (liquid level sensor and extensometer) for the specific case of underpinned buildings.

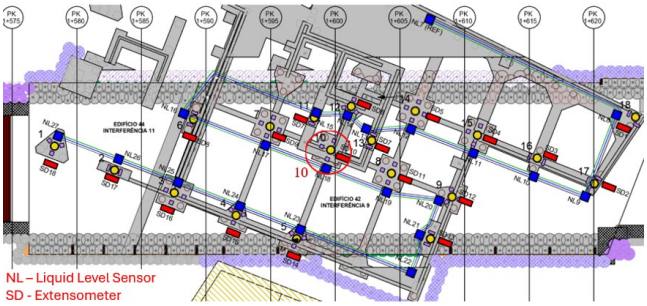


Fig. 5. Monitoring devices plan.

Figure 6 shows the load transfer mechanism and the measured vertical displacements at the underpin slab during the load transfer operations.

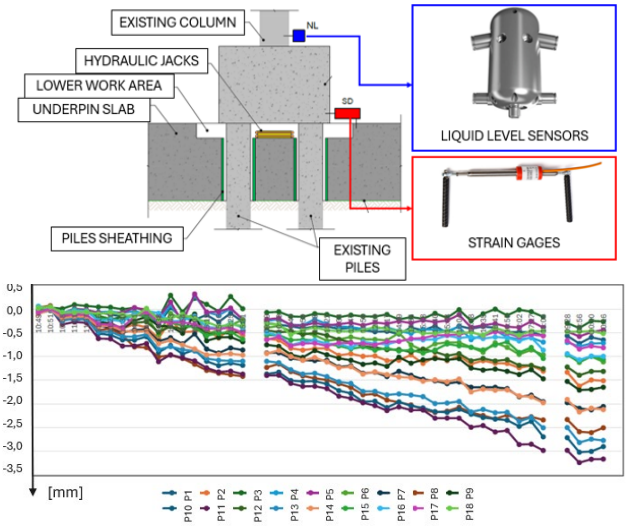


Fig. 6. Vertical displacement at the underpin slab (3,1mm maximum value) measured with liquid level sensors.

5 FINAL REMARKS

The construction of a new Metro line in densely urbanized and low cover areas led to interference with existing structures. In those scenarios, real time monitoring to support engineering decision, should be pointed out as a key tool.

ACKNOWLEDGEMENTS

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Dewatering and its complexities in design and implementation

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ABSTRACT

The article examines the main issues in the design and implementation of deep groundwater lowering systems. It discusses the creation of a water level not exceeding 148.6 meters and the use of groundwater lowering wells. The disadvantages of using the method of drilling water wells without backwashing, which contributed to the mixing of clay and sandy soils during drilling, were revealed.

The influence of gravel casing on the effectiveness of the systems is analyzed, along with violations regarding the placement of filters and the dimensions of filter sieve openings. The mutual influence of groundwater lowering measures is investigated, including the duplication of wells, the use of sumps, and the creation of additional systems, which may improve the effectiveness of water drainage.

The necessity of a comprehensive approach to groundwater lowering issues is emphasized for optimizing technologies and achieving stable results.

Keywords: groundwater lowering, deep wells, water level management, soil mixing, gravel casing.

1 ABSTRACT

The TATE residential complex implemented by Kortros Group on Vetkina Street in Moscow is a modern business and premium-class project comprising two towers with a height of 41 and 49 floors, as well as a significant underground part intended for parking and engineering facilities (fig.1).



Fig. 1. Project Reference

The presence of underground space in combination with the height of the complex imposes special requirements to the stability of the foundation and safety of construction, which makes the excavation development and the construction dewatering system the key stages of the project realization.

Elba Engineering Company acts as a technical consultant and designer, supporting the project at all stages, from pre-project surveys to quality control and commissioning. Within the framework of works on Vetkin's TATE housing estate, special attention was paid to the analysis of the general contractor's design documentation and development of measures for deep dewatering, including the design and assessment of the effectiveness of dewatering wells along the perimeter of the excavation.

Fig. 2. ELBA ENGINEERING LLC logo.

2 MAIN PART

Construction dewatering in the conditions of dense urban development and in the presence of deep underground part of the object requires the application of a set of engineering solutions aimed at artificial reduction of groundwater level to the levels safe for excavation and foundation construction. In the arsenal of designers there are both surface methods (open drainage,

needle filter installations) and deep systems (wells with submersible pumps, vacuum installations), and combined.

Each method forms an individual depression funnel, the parameters of which depend on the filtration characteristics of the soil, geometry of the excavation and hydrogeological conditions of the site. Of particular importance is the study of the mutual influence of several dewatering systems operating simultaneously or in stages. Incorrectly executed system of deep water leveling leads to impossibility of formation of design depressions of water leveling wells, which can lead to redistribution of water flows and formation of local zones of overwatering or overdrying, which can cause significant difficulties during works in the excavation, as well as unevenness of building settlement. Climatic factors such as seasonal precipitation and temperature fluctuations have a significant impact on water removal efficiency, requiring dynamic adjustment of pumping equipment operation parameters. Energy synergy when combining vacuum and gravity methods requires optimization of equipment capacity to prevent resource overruns and improve overall system efficiency.

In the construction of a high-rise residential complex with a developed underground part, strict adherence to design solutions is critical, as even minor deviations from the parameters set by the designers can lead to destabilization of the excavation, subsidence of adjacent buildings and damage to drainage systems. Examples of typical violations include exceeding the rate of dewatering, which provokes suffosion of soil particles and reduces the bearing capacity of the foundation, as well as ignoring the layering of aquifers when drilling wells, which leads to localized breakthroughs of pressure water. Practice shows that the reliability and safety of water dewatering works are largely determined by the quality of design solutions and competent selection of technologies, taking into account the specific hydrogeological conditions of the site.

3 COMPANY EXPERIENCE

As part of the implementation of the TATE Vetkin Residential Complex project, a scheme of deep water leveling was initially adopted with the use of dewatering wells along the perimeter of the excavation, designed to lower the groundwater level to a mark 1m below the design level of the foundation footings.

However, the analysis of the system operation revealed a number of shortcomings: the filtration parts of the wells were placed in layers with low water yield, which did not provide the required water inflow from the overlying horizons, and the lack of backwashing during drilling of the wells and incorrect selection of the filter packing resulted in their clogging and reduced performance. In addition, violation of the requirements of the regulatory documentation of the state where the construction took place, expressed in the mismatch of the filter mesh size to the granulometry of sands, aggravated the situation and did not allow to achieve the necessary level of dewatering of the excavation site. As an emergency measure the scheme of surface water drainage using sump and trenches was implemented, but due to errors in execution (lack of fixing the backfill of trenches with crushed stone, use of plastic tanks instead of sump) this solution was ineffective, and the excavation remained in a state of overwatering.

In addition, violation of the requirements of SP 250, clause 7.2.5, which was expressed in the mismatch of the filter mesh size with the granulometry of sands, aggravated the situation and did not allow to achieve the required level of site drainage. As an emergency measure, a scheme of surface water drainage using sump and trenches was implemented, however, due to errors in execution (lack of trench fixing, use of plastic tanks instead of sump), this solution was ineffective and the excavation remained in a state of overwatering.

Special attention in the project was paid to the area of elevator pits, which have a significant depth compared to the main part of the excavation. Needle filters were provided here, which allowed to stabilize the hydrogeological situation locally and to ensure drainage of the foundation in the most critical points. After implementation of the needle filter system around the perimeter of the excavation and in the zones of local depressions, the required level of dewatering was achieved, which allowed to continue construction without any risk for machinery and structures.

This experience has clearly demonstrated that only a comprehensive approach, including the correct selection of technologies, consideration of hydrogeological features of the site and strict compliance with the project regulations, ensures successful performance of water dewatering works in the construction of high-rise facilities with a developed underground part.

4 CONCLUSION

Thus, the analysis of the design and implementation of the dewatering system at the TATE Vetkin Residential Complex emphasizes the high importance of scientifically-based choice of technologies and organizational solutions, as well as strict adherence to the requirements of SP 250.1325800.2016. In the conditions of dense urban development and in the construction of buildings with deep underground part even local errors in dewatering can lead to significant economic and technogenic risks.

The experience of the case analyzed by the company proves the necessity of implementing solutions that could predict hazardous situations at the early stages of design. And also would help to prevent them by means of situation modeling, as a numerical experiment, so that it would be possible to envisage measures for their elimination.

Justification of the choice of dust suppression materials for roadside areas with low coverage and construction sites: international and Kazakhstan experiences

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ABSTRACT

The article provides a comparative analysis of modern dust suppression materials used in the road and construction sector, with an emphasis on their efficiency, sustainability and applicability in various climatic conditions. Particular attention is paid to Kazakh developments based on the use of local technogenic raw materials. A review of the works of authors from different countries, including China, the USA, Spain and Kazakhstan, is presented.

Keywords: dust suppression, dirt roads, polymers, ash and slag materials, road construction, Kazakhstan, sustainable development.

1 INTRODUCTION

Increased dust formation on low-coverage road sections and construction zones has a significant impact on air quality, the state of ecosystems and public health. In the conditions of Kazakhstan, where arid and semi-arid zones predominate, the issue of effective dust suppression is of strategic importance. The use of local and innovative materials allows us to reduce both direct maintenance costs and negative environmental impacts.

Table 1. Comparative analysis of efficiency

Autor(s)	Material type	PM-reduction (%)	Sustainability	Scope of application
Huang et al	Copolymers	up to 90%	Average	Dirt roads
Sun et al.	EC + PVAc	up to 85% (PM2.5)	Tall	Dusty construction sites
Parvej et al	Biopolymers, salts	up to 60%	Low	Short-term road facilities
Suleimenov and others.	Ash, slag	up to 80%	Tall	Permanent coatings
Asmatulaev and others.	Rolled concrete	up to 85%	Tall	Temporary roads, agricultural facilities
Rysbekov and others.	Waste from enrichment	up to 70%	Average	Technological passages

Hygroscopic salts are among the most effective materials for suppressing dust on low-surface roads and construction sites.

The choice of dust-control materials should take into account climatic and geotechnical conditions, as well as environmental aspects. Modern studies by domestic and foreign authors confirm the high efficiency of using hygroscopic salts, complex salt compositions and man-made waste. The use of such materials helps improve the environmental situation and increase the durability of road surfaces.

The authors of the article examined the use ECO-ROADS stabilizing additive. The following types of work were performed:

1. Soil samples were divided into 3 groups according to the plasticity number: the first group – sandy loam soils, plasticity number up to 7; the second group – loamy soils, plasticity number – 7-17; the third group – clayey soils, plasticity number over 17. The plastic characteristics of the soils were determined in accordance with.

2. Prepared dust-control solutions of various concentrations.

3. Soil samples were pre-moistened using prepared solutions, taking into account the norm of their distribution.

4. After weighing, the moistened soils were dried in the open air (Fig. 3), the mass of the samples was measured at 30-minute intervals and their drying time was recorded.

5. The measurement results were recorded in a laboratory journal.

For experimental tests, traditional salt solutions of various concentrations (NaCl, $MgCl_2$, $CaCl_2$, $MgCl_2 \cdot 6H_2O$, etc.), as well as Durasoil and Soiltac solutions were used. SOILWORKS company.

Processing of results. Based on the results of testing three samples, a graph of the dependence $S = f(p)$ is plotted in the laboratory work log in accordance with the graph shown in Figure 3.9.

Strength characteristics of soils – angle of internal friction φ with an accuracy of 1 0 and specific adhesion *WITH* with an accuracy of 0.001 MPa is determined using formulas [3,4].

$$tg\varphi = \frac{S_3 - S_1}{(p_3 - p_1}, \quad (1)$$

$$C = S_i - p_i \cdot tg\varphi, \quad (2)$$

Where are the indices at S_i and p_i correspond to the i - th sample numbers.

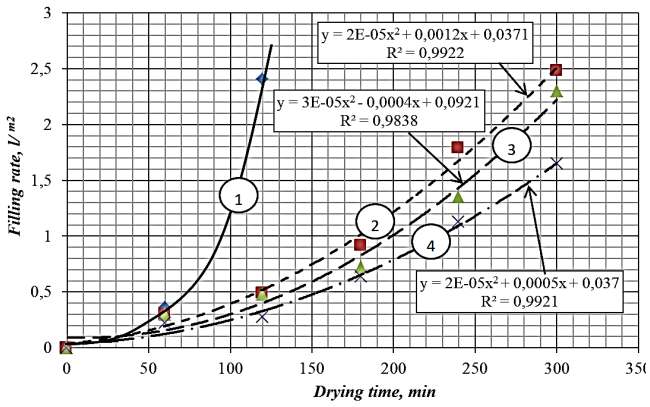


Fig. 1. The drying process of pre-moistened loamy soil with a 25% concentration of various salts:

1 – water; 2 – calcium chloride; 3 – sodium chloride; 4 – magnesium chloride with six water molecules (bischofite)

The stability of soils to retain moisture is also significantly affected by the concentration of the solutions used. The duration of the dust-removing effect of a 25% solution is 2.5-3.5 times longer than that of a 5% solution. Therefore, the use of solutions of increased concentration is the most effective.

From analysis The test results follow, What the coefficient of hydrophobicity increases With increase concentration stabilizer solution. At this at increase concentrations o t 0.02% to 0, 1% from the mass waters coefficient hydrophobicity increases about t 0, 3 up to 0.45, further increase concentrations V 2 times (about t 0.1% to 0.2%) slightly increases the hydrophobicity coefficient (by 0.01). For further testing selected concentration 0,1%.

CONCLUSION

1. Experimental tests have shown that bischofite is the most effective dust-removing material among hygroscopic salts. Its action time is 47 minutes longer than that of sodium chloride, 63 minutes longer than that of calcium chloride, and 206 minutes longer than that of ordinary water.

2. The duration of the effective action of chemical solutions depends on their concentration. For example, with a 5% solution of bischofite, its effective action will last only 64 minutes, with a 10% solution - 98 minutes, and with a 25% solution - 192 minutes.

Dust suppression technologies are a key area of sustainable road construction, especially in arid regions. International experience confirms the effectiveness of enzyme, biological and polymer compositions. Kazakhstan's experience emphasizes the value of waste recycling and its secondary use in road infrastructure.

It is recommended to intensify field testing, introduce hybrid technologies and support domestic developments through government programs and private investment.

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Detection of the unbonded sections of fully grouted rock bolts using acoustic waves

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ABSTRACT

Unbonded sections may occur at various points along the length of fully grouted rock bolts. This research explores the condition assessment of these bolts through acoustic wave analyses and controlled model experiments. Rock bolts with varying lengths of unbonded sections were embedded in a concrete block. Hammer strikes were applied to the bolt heads to induce longitudinal vibrations in the rebars. The resulting acoustic signals were measured by a microphone, and their frequency characteristics were subsequently analyzed. It was observed that a longer unbonded section corresponded to a reduction in the dominant frequency of the longitudinal mode. This study demonstrates that the structural soundness of fully grouted rock bolts can be effectively assessed using microphone measurements and analysis of vibration frequency responses.

Keywords: acoustic wave, flexibility, fully grouted rock bolts, microphone, unbonded section

1 INTRODUCTION

Rock bolting is widely recognized as a cost-effective and reliable technique for reinforcing rocks in tunnel linings and along slopes (Li et al., 2021; Kang et al., 2025a). Once installed, rock bolts serve to interlock fractured rock segments, apply tensile resistance, and minimize deformation around excavated areas (Yu and Yoon, 2024). The performance of a rock bolt is governed by its own mechanical strength (tension and shear), the properties of the grout, and the characteristics of the rock mass in contact with the bolt (Sarfarazi and Tabaroei, 2020; Yokota et al., 2020; Xu and Gutierrez, 2021). However, grout may not always completely occupy the drilled hole due to issues such as material leakage, inefficient injection techniques, grout separation, or infiltration into fractures in the rock (Cui, 2013; Yu and Lee, 2024). Consequently, certain portions of the rebar may remain unbonded to the rock along the bolt's length. These unbonded regions can degrade the bolt's structural effectiveness (Liu et al., 2022).

To assess the condition of in-situ rock bolts, pull-out testing is often utilized. While pull-out tests are straightforward and economical, they may not provide a uniform stress distribution along the bolt and typically only reflect the behavior near the exposed head. As a result, identifying the exact extent of unbonded sections remains difficult. To overcome this limitation, nondestructive testing methods have been introduced. However, the nondestructive testing methods generally demand costly sensors and sophisticated instruments (Kang et al., 2023; Kang et al., 2025b). Moreover, sensor-to-structure coupling conditions can affect the reliability of the collected signals, potentially introducing distortions (Hu et al., 2018; Kang et al., 2025c).

This research focuses on estimating unbonded lengths in fully grouted rock bolts using a microphone-based approach (Kang et al., 2021). A series of model experiments was carried out using bolts with intentionally varied unbonded sections. Acoustic signals resulting from longitudinal and flexural impacts were recorded, and the frequency characteristics of these signals were studied. The unbonded zones were assessed by analyzing the dominant frequencies and evaluating the bolt flexibility. The study presents theoretical insights into rock bolt dynamics, including vibration modes, flexibility, and resonance, followed by a detailed description of the experimental design, instrumentation, results, interpretation, and final conclusions.

2 EXPERIMENTAL STUDY

2.1 Experimental setup

A concrete block was used to simulate the rock mass in which the rock bolt was embedded (Fig. 1). The concrete block was 3.0 m in length, 1.1 m in width, and 1.1 m in height. Rock-bolt specimens were prepared by employing the rock-bolting process.

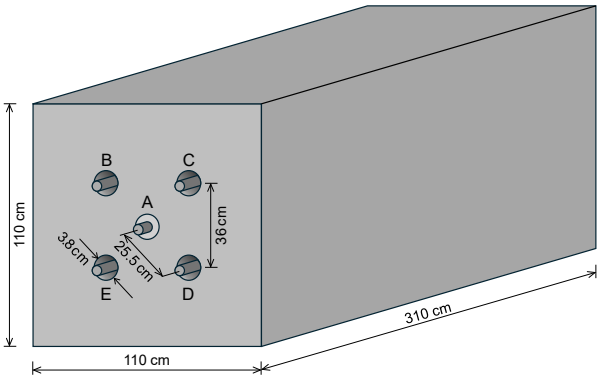


Fig. 1. Schematic drawing of concrete block.

First, five holes (holes A–E in Fig. 1) having a diameter of 3.8 cm were drilled. The distance (center-to-center) between the holes was set to be 25.5 or 36 cm. Second, D25 rebars having a diameter of 2.54 cm and a length of 301 cm were inserted into each hole. Third, grouting concrete was injected with the inserted rebars. Thus, the embedded and protruded lengths of the rock bolts were 300 and 1 cm, respectively. Rock-bolt specimens having unbonded lengths (L) of 0 (fully grouted), 60, 90, 120, and 150 cm were prepared by varying the amount of grouting concrete poured, as shown in Fig. 2. The injected grouting concrete was cured for 28 days, with a compressive strength of 35 MPa after 28 days of curing. Note that the water-cement ratio of the grouting concrete was set as 0.5, which is a typical value (Hoek and Karakas, 2008; Wyllie, 2017).

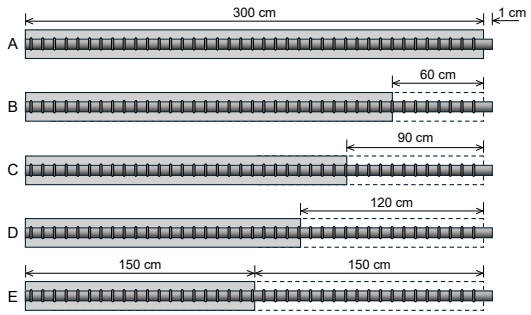


Fig. 2. Rock bolt specimens with different unbonded lengths.

2.2 Measurement system

As illustrated in Fig. 3, acoustic waves were produced by delivering axial impacts to the protruding sections of the rebars. To ensure consistent impact energy, a hammer (PCB Piezotronics, 086D05) was lifted at a 45° angle and released, generating axial excitation (Kang et al., 2022; Kang et al., 2024b). The axial impact was applied at the center of the rebar's cross-sectional area (Fig. 3). Each impact's force signal was captured using a load cell integrated into the hammer. Since the stiffness of the hammer tip can influence the resulting frequency content of the impact, a steel tip with a 0.6 cm diameter was employed to minimize impedance mismatch between the hammer and the rebar.

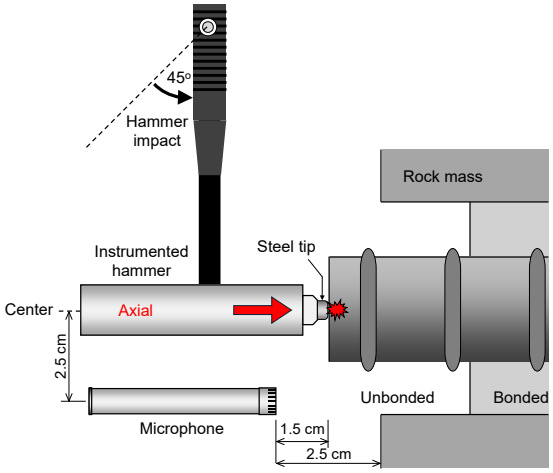


Fig. 3. Measurement system.

To assess the resulting acoustic waves, a condenser microphone (PCB Piezotronics 378B02) was utilized. This microphone is highly responsive to low-frequency signals within the 7–10000 Hz range and maintains a flat frequency response within ±1 dB across this spectrum (PCB Piezotronics, 2017; Kang et al., 2022). During measurements, the microphone was positioned 2.5 cm below the center of the rebar's cross-section for axial impacts and 1.5 cm from the rebar's tip (2.5 cm away from the concrete surface) for transverse impacts. A signal conditioner (482C05, PCB Piezotronics) was used to enhance the quality of the acquired signals by improving the signal-to-noise ratio for both the hammer and microphone outputs.

2.3 Experimental results

Fig. 4 presents the microphone responses recorded for unbonded lengths (L) of 0, 60, 90, 120, and 150 cm under axial impacts. The arrival times of the recorded signals were aligned with the corresponding hammer impact time, defined as 0 ms. Across all cases, a prominent acoustic wave was detected immediately following the hammer strike. For the specimen with L = 0 cm, the signal diminished sharply after the initial peak under both axial and transverse impacts. In contrast, the signals for other unbonded lengths decayed more gradually and persisted beyond t = 10 ms.

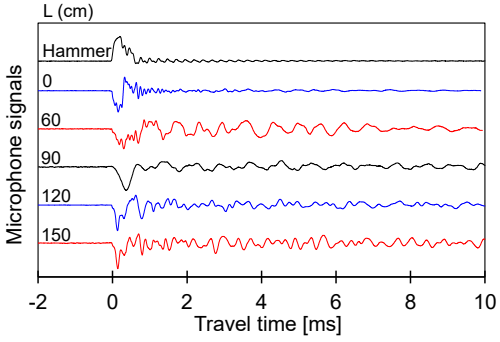


Fig. 4. Measured acoustic waves.

Geotechnical analysis of the applicability of modern pile technologies for construction in the historical part of St. Petersburg and in the reconstruction of buildings

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ABSTRACT

The importance of pile foundations on weak soils cannot be overestimated. Today, this is practically the most common method of fundraising in conditions of dense urban development in terms of minimizing the impact on buildings of the surrounding development. Due to their adaptability and high degree of reliability, they are widely used in both domestic and foreign foundation construction practice. In many construction situations, pile foundations are the only way to build foundations in difficult construction and geological engineering conditions. The practice of monitoring the precipitation of buildings during construction in the built-up part of the city confirms that when using pile foundations for new construction, the impact on the occurrence of additional precipitation of neighboring buildings is significantly reduced. In this decision, the choice of pile technology is important. The article presents the results of a qualitative analysis of structural and technological solutions for the installation of pile foundations during construction in dense urban areas. Based on the conducted research and practical experience, the technological features of the manufacture of drilling and packed piles in dense urban areas in weak soils have been analyzed, the risks of the impact of pile foundations on the surrounding area have been assessed, and recommendations for their effective use in various engineering and geological conditions have been given. The article provides a rating compiled by the author on possible technical solutions for foundations, taking into account the design and technology of pile manufacturing. This rating allowed us to decide on the choice of a constructive and technological solution that meets the established criteria and their weights.

Keywords: pile foundation, technological impact on soil, deformation of neighboring buildings, soft soils, safety of geotechnical work,

1 INTRODUCTION

The history of pile foundation construction, as well as the evolution of the use of piles in our city, has not been immune to any round of development and improvement of pile technologies. And this story began, as it did everywhere, with simple driven wooden piles. But as the centuries passed, technological progress also came to this branch of construction, and in the late 19th century, technologies were invented that have also survived to this day, and we know them as drilled and bored piles. Currently, there is no unified, universally accepted classification of piles.

2 TAKING INTO ACCOUNT THE INFLUENCE OF VARIOUS FACTORS IN GEOTECHNICAL CONSTRUCTION ON WEAK SOILS

It is proposed to determine the influence of geological and design parameters and technological factors on the modeling of the geotechnical construction process on weak soils using a functional dependence with the generalized indicator Og, according to the expression

$$O_g = f(G, E, D, T, R, S), \quad (1)$$

where G is the component of geological influence;
E is the component of external loading of the system;
D is the geometric characteristics of the foundation system;
T is the technological parameters of the foundation device;
S is the deformation values of the foundation system, including the maximum allowable deformation values.

When building in dense urban areas, an important condition for ensuring safety is the prediction of possible risks of additional uneven deformations based on the results of numerical modeling based on geotechnical forecasting (justification) with an assessment of the zone of influence of geotechnical construction on the buildings of the surrounding area. Such calculations are performed using software packages based on the finite element method (FEM), which makes it possible to evaluate the interaction of the soil mass and the construction site.

3 RESEARCH ON THE PERFORMANCE OF DISPLACEMENT PILES IN DENSE URBAN AREAS

The operation of such a pile is determined by the loading conditions of the pile foundation, and the factors that determine the bearing capacity are friction on the lateral surface and soil resistance at the tip of the pile. The technology itself does not involve any special sealing effect on the soil mass around the pile, which results in low calculated values of the pile's bearing capacity.

The use of factory-made piles, driven piles, and cast-in-place piles has become widespread in the construction of buildings in dense urban areas. Drilling-injection piles are actively used for small loads on foundations and for strengthening the foundations of buildings, including during reconstruction. Special attention is paid to the new and effective DDS (Drilling Displacement System) cast-in-place pile design, which we have implemented at several sites in St. Petersburg. They are made using a spiral projectile, which is a cone or cylinder of variable cross-section with a spiral blade. Wells for such piles are performed without removing the soil, by displacing it to the sides and upwards. The formation of a well during the manufacture of a pile using this technology is described using the classical Lamé equation for loading a cylindrical cavity in an elastic medium with internal pressure. As a result of additional compression of the well walls during the manufacture of a DDS pile with cast concrete, the radius of the well increases. The volume of excess concrete relative to the nominal size of the well serves as an indirect indicator of the degree of compaction of the soil mass in the area affected by the drilling rig. Pressing the well with high concrete mixture pressure helps to increase the bearing capacity by forming a contact layer between the pile concrete and the soil mass.

In order to assess the impact of the DDS pile manufacturing process on the change in the soil mass's stress-strain state, field studies were conducted to determine the safe zone of influence on the foundations of neighboring buildings. The studies were conducted in the central part of St. Petersburg using the RIG 204 mobile static probing system, which allows for the measurement of pore pressure.

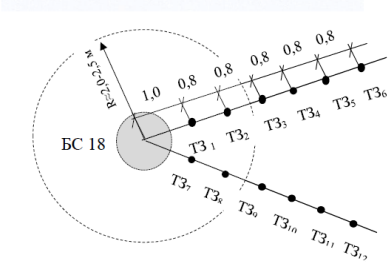


Fig.1 The CPT –point for investigation of the compaction soil around the pile

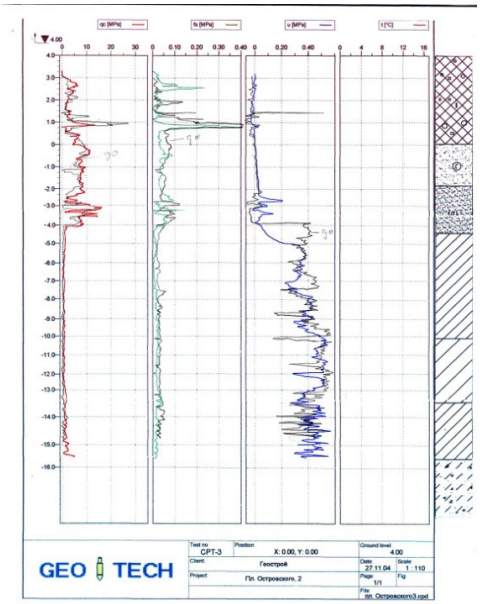


Fig.2 The view of diagram of CPT –parametrizes

4 CONCLUSIONS

In dense urban areas with weak soils, the use of prefabricated piles, driven piles, drilling piles, cast-in-place piles, and drilling-injection piles has become widespread in construction practices. An analysis of the characteristics of these technologies and an expert assessment have revealed that the technologies of piles protected by clay mortar and protected by an инвентарная casing pipe have the lowest impact on the foundation soils. The continuous hollow auger (CHA) technology has a high level of impact. In some cases, the technology of making cast-in-place piles (with soil displacement) leads to the lifting of the surface layers of soil and the foundations of existing buildings. Subsequently, sedimentary deformations develop, followed by the relaxation of the resulting stresses caused by anthropogenic influences. This circumstance requires technological and organizational regulation of the procedure for performing pile works in the affected area.

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Special Lecture Ⅲ:

Construction of vertical cut-off walls for reducing exposure to floods

Mr. Waldemar Kwiatkowski

Construction of vertical cut-off walls for reducing exposure to floods

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ABSTRACT

Climate change and growth of the population worldwide increase the risk of floods occurrence in urban areas, as was experienced in Kazakhstan in 2024. Flood prevention needs a complex approach which consists of monitoring, rivers regulation, dams and soil flood embankments engineering, construction and maintenance as well as international agreements.
This article presents the current state-of-the-art methods of cut-off walls construction for new projects and improvement of existing hydrotechnical and flood protection structures and gives a compact overview of available geotechnical methods to help reduce population exposure to floods. Selected geotechnical technologies are presented. The final choice of design solution and proper geotechnical technology depends on many aspects discussed in this work.

Keywords: cut-off walls, flood protection, filtration, soil flood embankments

1 INTRODUCTION

Climate change has been observed worldwide in recent years. One of its many consequences is an increased risk of flooding. However, uncontrolled water release can also result from other factors, such as poor maintenance, inadequate monitoring, and lack of river regulation. These events consistently lead to environmental damage and economic losses.
Extensive floods hit Kazakhstan in 2024, a dam failure occurred at the Sardoba Reservoir in Uzbekistan in 2020. These are only examples of widespread flooding that devastated surrounding landscapes and affected many residents.
One method to reduce flood risk is the use of vertical low-permeability barriers, also known as cut-off walls. These can be implemented in existing structures to enhance safety or incorporated into newly designed projects.

2 DEFINITION AND ROLE OF CUT-OFF WALLS

Vertical cut-off walls are low-permeability barriers constructed underground to control or limit groundwater flow. Cut-off walls are commonly used in civil engineering projects as well as environmental ones to control the movement of contaminants. Various construction methods of cut-off walls are presented in this article.
The main advantages of cut-off walls installed in the earth dams are:
a) decrease of the water flow speed (m/day) and volume (m^3/day) at the land side;
b) increase of earth dam stability;
c) extended time of first leakages on the land side.
To illustrate the influence of cut-off walls installed in the earth dam, simple comparison of a steady-state situation is presented for cases with and without cut-off wall. Calculations were done for one of the already completed projects for Vistula River in Poland (Topolnicki, 2005). Cut-off wall has been executed with soil mixing technology. The results are presented in table 1 and schematic cross-section is shown in figure 1.

Table 1. Comparison earth dam calculation results.

Case	Dam stability safety factor	Max. flow speed (m/day)	Volume (m^3/day)
Without cut-off wall	0.92	9.5	6.0
With cut-off wall	1.64	5.3	0.6

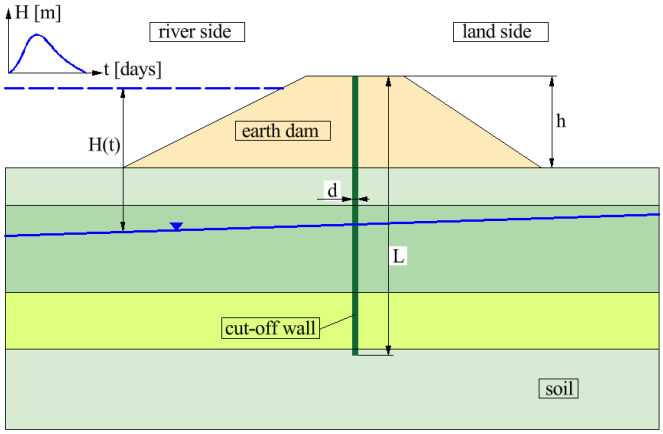


Fig. 1. Schematic cross section of the earth dam with vertical cut-off wall.

3 CLASSIFICATION OF CUT-OFF WALLS

Cut-off walls can be constructed in various configurations to control the flow of groundwater. The vertical barrier can either be keyed into a low-permeability layer or be hanging, where the wall does not extend into such soils. The appropriate geometry and location of the cut-off wall should be determined by consultants based on a hydrogeological analysis. Advanced software and models are nowadays available and can deliver precise results; however, accurate geotechnical soil parameters must be obtained and provided. This often is an issue during the engineering process.

The role of cut-off walls is to reduce seepage and potential washing effect, thereby reducing the risk of flooding. To achieve this, the appropriate cut-off wall technology must be selected. Several geotechnical technologies are available for use:

a) slurry walls (clay, soil-bentonite, cement-bentonite, soil-cement-bentonite),
b) soil mixing walls,
c) grouted barrier walls (jet grouting, permeation grouting),
d) sheet pile walls,
e) other types of cut-off walls.

The choice of proper technology depends on many aspects, e.g., type of soil, wall depth, geometry of the dam, project requirements.

4 GEOTECHNICAL TECHNOLOGIES FOR CUT-OFF WALLS EXECUTION

4.1 Grabbed slurry walls

The grabbed slurry cut-off wall means that excavation is done with the slurry which after hardening process creates a wall body (single phase cut-off wall) or slurry has supporting function only and is later replaced by the concrete or other material (double phase cut-off wall). Slurry used for single phase cut-off walls is usually mixture of cement (usually CEM III/B), bentonite, water and chemical additives. The UCS is typically less than 1MPa, filtration coefficient $k<1x10^{-8}$ m/s and Young Modulus is expected to be low to assure joint work of cut-off wall and surrounding dam body.

In general, there are three main types of slurry cut-off walls: soil-bentonite, cement-bentonite, and soil-cement-bentonite. Depending on the type, the backfill material contains a mixture of bentonite, cement, clay, fly ash, and ground granulated blast furnace slag. Typical equipment used for the construction of slurry wall is shown in Figures 2.



Fig. 2 Equipment for grabbed cut-off slurry wall (1).

4.2 Soil mixing cut-off walls

Soil mixing cut-off walls are constructed by mixing the existing soil with the slurry, what creates the cut-off wall body. The choice of construction method depends on the available equipment and project requirements.

There are three main methods for constructing soil mixing cut-off walls, depending on the equipment used:

a) cutter soil mixing method with mixing tools rotating about horizontal axes (Figure 3),
b) vertical trenching method (Figure 4),
c) soil mixing method with mixing tools rotating about vertical axes (Figures 6 and 7).



Fig. 3 Cutter soil mixing with mixing tools rotating about horizontal axes (1).



Fig. 4 Equipment for vertical trenching method (12).

The soil mixing method with mixing tools rotating about vertical axes, popularly called DSM (Deep Soil Mixing), became very common in CIS countries recently. DSM technology is widely used across various construction sectors under foundations, road embankments, for cut-off walls and retaining structures.

The DSM cut-off walls consist of overlapping columns created by rotating augers that mix in-situ soils with a low-permeability slurry. The slurry is injected through nozzles during the mixing process, producing a hardened mass (a mixture of soil and slurry). A continuous cut-off wall is formed by overlapping individual columns (Figure 5). The column diameter, pattern, and slurry types vary depending on site conditions and specific design requirements. Common slurry materials include bentonite, cement, lime, fly ash, among others.

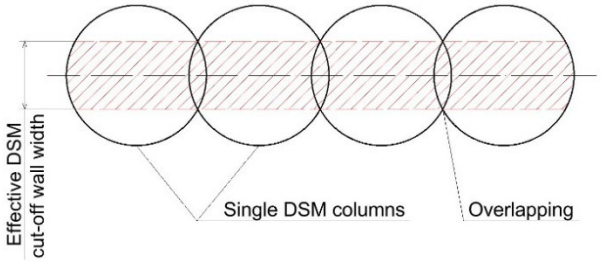


Fig. 5 DSM cut-off wall created by overlapping DSM columns

Execution of DSM cut-off wall typically requires the use of a rig fitted with either one, or a multiple mixing augers. Figures 6 and 7 show typical equipment for DSM cut-off wall production.

4.3 Grouted cut-off walls

The application of grouting techniques in geotechnics has limitations depending on the soil type and its grains size. The general limits are shown in figure 8.

Permeation grouting and jet grouting techniques are sometimes used for creating cut-off walls, however due to their high costs and relatively low productivity, those methods are not very popular. The application of these methods is common practice in combination with other methods in rock formations, soils which contain boulders as well as in case of any underground obstacles (e.g., underground pipelines or concrete structures).



Fig. 6 Equipment for DSM technology – single mixing tool.



Fig. 7 Equipment for DSM technology – multiple mixing tools (12).

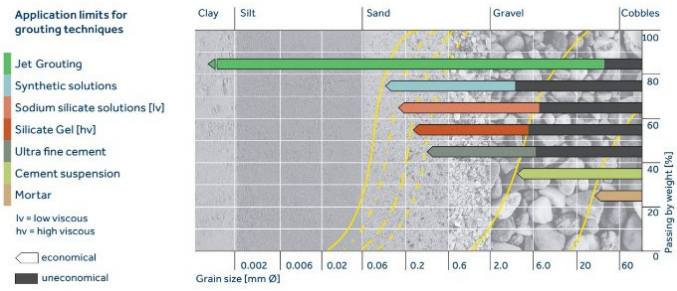


Fig. 8 Application limits for grouting techniques (5).

Jet grouting is one of the most flexible soil treatment methods to create structures for sealing and load bearing in geotechnics. In general, this technique injects the

grout mixture at very high pressure and velocity directly into the pore spaces of the soil or rock. The grout being injected cuts, replaces, and mixes the in-situ soil, destroying the soil structure and forming a homogeneous mass.

Permeation grouting is a low-pressure method, in general, a hole is drilled into the ground and a fluid is injected under pressure into the soil or rock, permeating the voids.

Type of grout depends on the soil type and the design requirements. Synthetic materials, silicate gels, ultra fine cements are often used in fine soils (grain sizes less than 2 mm), what makes grouting methods economically less attractive compared to other ones.

The advantages of grouted cut-off walls are flexibility in creating different grouted mass shapes as well as small size and weight of the equipment.



Fig. 9 Example of small-size drilling rig for jet grouting method (5).

4.4 Sheet piles walls

Steel sheet pile walls are commonly used in the construction sector mainly as excavation or slopes stability protection and are not good solution as a cut-off wall. This is because of high costs, its potential corrosion and leakages of the joints. However, relatively new and good to mention alternative for steel sheet pile walls is vinyl sheet piling. Vinyl sheet piles are an environmentally friendly, lightweight and extremely durable and advanced alternative to traditional materials such as steel. The commonly used method of vinyl sheet piles installation is the use of vibratory hammers and pre-installed templates. The type of equipment depends on the soil type, the depth of the installation and the durability of the sheet pile. In the case of hard, dense surfaces, or when long elements need to be hammered, steel mandrels are additionally used. Mandrels are special guide bars of shape and length reflecting the used PVC elements.



Fig. 10 Example of already installed vinyl sheet piles (10).

4.5 Other types of cut-off walls

There are other methods of cut-off walls construction available in different regions depending on local practice, experience, available equipment and material.

Application of ball-clay (instead of cement-bentonite slurry), in the grabbed slurry cut-off wall method is common practice in CIS countries. As material cost comprises the majority of all cut-off wall cost, this approach is very popular in the regions where the specific ball-clay is naturally present. Besides financial advantage, this type of cut-off wall has low deformation modulus what assure joint work of cut-off wall and surrounding dam body. However, the application of this method at higher depths leaves many doubts as to the quality of construction and the continuity of the cut-off wall.

Vibro methods using either the so-called “wing vibrator” or a suspended H-beam are also popular in some parts of Europe. This technique involves installing the wing vibrator or suspended H-beam into the dam body. During the extraction process, the resulting space is filled with slurry material. This is a quick method for creating cut-off walls, though it has limitations related to the depth and dam’s geometry. The impact of vibration on the dam body and surrounding area must also be taken into account. Examples of these vibro methods are shown in Figures 11 and 12.

5 CONCLUSIONS

Cut-off walls are constructed on river earth protection dams and retention reservoirs to enhance flood protection safety. The maintenance of existing flood protection structures is often at a low level, leading to their degradation, and in the event of a flood, the risk urban areas of flooding increases. The use of cut-off walls is a common global practice for renovating and

improving safety of flood embankments.
It is worth noting that anti-filtration barriers are also used to control groundwater flow in areas where underground contamination occurs. These barriers help control and limit the migration of contaminated groundwater into undesired areas.
A brief overview of available geotechnical methods for constructing cut-off walls is presented in this article. The choice of appropriate technology always depends on soil conditions, design requirements, and the local availability of materials. Recent floods in Kazakhstan and the failure of a retention reservoir in Uzbekistan have shown that implementing complex appropriate protective measures in this area is a necessary task.



Fig. 11. Example of vibro method using wing vibrator (Gluszkiewicz, 2002).



Fig. 12. Example of vibro method using hanged H-beam (Gluszkiewicz, 2002).

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Invited Lecture Ⅱ:

Assessment of Soil Contamination in Incheon Harbor Area

Prof. Eun Chul Shin, Dabeen Kim
, Jeong Ku Kang

Assessment of soil Contamination in Incheon Harbor Area

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ABSTRACT

Incheon Harbor, opened in 1883, includes six terminals handling various imports and passenger traffic. This study analyzes soil pollution in the harbor area using data obtained from the Korean soil monitoring network and local investigations over the past five years. Key pollutants identified are copper, lead, zinc, and fluorine, with high contamination levels in the north harbor due to industrial activities. Preventive measures are included strengthening the pollution prevention systems, environmental expertise among officials, and alongside establishing a soil monitoring network for better management.

Keywords: harbor area, soil pollution, high contamination level, key pollutant, soil monitoring network

1 INTRODUCTION

The Incheon Harbor has been opened the door to the world since year 1883. Now, it has 6 harbors such as Incheon Inner Harbor (main terminal, handling bulk of grain, sand, steel), Incheon North harbor (importing wood, used steel), Incheon Domestic Passengers Terminal (travelers to domestic islands), Incheon South Port (importing coal), Incheon New International Passengers Terminal (Travelers to China and International), and Incheon New Port (Cruise Ship and Container Terminal).

The usage of land around Incheon harbor area belongs to general industrial area and semi-industrial area with the facilities of harbor and commercial facilities. This paper presents the trend of soil pollution in Incheon harbor area through the results of the Korean soil monitoring network and soil contamination investigation history for the recent 5 years. The soil samples are obtained from the accessible 13 sites of Incheon harbor and surrounding areas. Based on the analyzed data, the concentration of each pollutant is identified by drawing up the soil contamination plume by using ArcGIS.

In order to identify the characteristics of the ground in the target areas, the overall ground pollution data of Incheon Metropolitan city are collected. The soil monitoring network data is provided by the Ministry of Environment and the soil contamination investigation data conducted by local governments (existing data survey) are readjusted and analyzed. The soil contamination data of heavy metal and oil concentration data are collected in the areas of Incheon harbor and surroundings, which are not sufficient enough for field survey data. Based on these results, major pollutants, those are concerned about cause of future soil pollution in the area of Incheon harbor and surrounding areas, are selected. In addition, the basic data for soil remediation and potential soil pollution are proposed by showing 2D soil contamination plume for each contaminant using the readjusted data.

2 GENERAL SPECIFICATIONS

Major pollutants derived by integrating existing data and field survey of 6 districts adjacent to Incheon harbor area are appeared to be copper (Cu), lead (Pb), zinc (Zn) and fluorine (F). But fluorine is distributed across all 6 districts, so its correlation with Incheon harbor area and surrounding areas is not considered significant. The results of soil contamination plume by using ArcGIS show high pollution indices for Cu, Zn, hydrargyrum (Hg), nickel (Ni) and TPH. It is because of the characteristics of the north harbor of Incheon, which handles the cargo of industrial materials including metal metallurgy, alloys and smelting.

As the preventive measures, company that has soil pollution-causing facilities should be required to strengthen their own prevention system against pollution. And it is necessary to enhance expertise in handing soil

affairs through the recruitment of highly qualified professional officials. Moreover, it is required to strengthen preventive system through the establishment of a soil monitoring network among officials.

Invited Lecture Ⅲ:

Design of the First Metro Line in Almaty Crossing Tectonic Fault Zones

N.S. Utegenov, A.A. Urazbayeva

Invited Lecture IV:

Engineering Solutions and Case Studies in Disaster Mitigation

Dr. Anil Joseph, Ashitha Joseph
and Dhanya J.S.

Engineering Solutions for Disaster Mitigation and controlled demolition techniques for buildings - Case Studies

Anil Joseph [i], Ashitha Joseph [ii] and Dhanya J.S [iii]

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ii) Geotechnical Engineer, Engineers Diagnostic Centre Pvt Ltd, Pullepady, Cochin, 682018, Kerala, India.
iii) Deputy Chief Engineer, Engineers Diagnostic Centre Pvt Ltd, Pullepady, Cochin, 682018, Kerala, India.

ABSTRACT

Engineering resilience is essential to sustainable development, allowing infrastructure to withstand, adapt to and recover from both natural and human-induced disasters. This paper presents three case studies that demonstrate the role of resilient engineering and controlled demolition in disaster mitigation and urban planning. The first case study examines a high-end resort project in Coorg, Karnataka, which suffered severe damage from landslides during the August 2018 floods. Triggered by intense monsoon rains, the landslides disrupted roads, foundations, retaining structures, and surrounding vegetation. Practical and cost-effective stabilization measures were implemented, including slope reinforcement using sandbags and geotextiles, structural retrofitting, deep well foundations, reconstruction of retaining walls and additional methods which helped to ensure long-term safety in the landslide-prone region. In urban areas and for structurally compromised high rise structures, demolition by controlled delayed detonation offers a safer, faster and less disruptive alternative to traditional demolition methods. It enables the removal of unsafe buildings after events such as deterioration, earthquakes, floods or fires, supporting rapid site clearance and improved public safety. The second case study focuses on a G+26 residential tower complex in Kochi, Kerala, built between 2013 and 2018, which was ordered for controlled demolition in 2025 due to severe structural deficiencies. Corrosion from chloride-laden water and multiple code violations led to structural instability, prompting a court-mandated implosion to prevent a potential disaster. This case highlights how proactive demolition can serve as a form of disaster risk reduction. The third case explores the demolition of a six-storeyed, 18-year-old building to enable commercial redevelopment. Both implosion-based demolitions emphasize the role of engineered interventions in ensuring safety and enabling urban renewal. Collectively, these studies underscore the importance of site-specific engineering strategies and controlled demolition in building resilient, sustainable infrastructure systems.

Keywords: Disaster mitigation, landslides, resilience, deterioration, controlled demolition

1 INTRODUCTION

The construction industry plays a vital role in disaster resilience through building design, emergency response, and long-term reconstruction, supporting sustainable urban development by mitigating risks from natural and human-made hazards. Communities depend on critical infrastructure to sustain daily functions and enable future growth (1). Disasters, while part of the natural environment, cause widespread disruption, impacting lives, ecosystems, infrastructure, and national economies (2). Rapid urbanization, increasing building density, and climate change have heightened structural vulnerabilities to geotechnical failures, seismic activity, and deterioration (3). As a result, innovative technologies and proactive engineering interventions are essential to enhance urban resilience and public safety (4). Disaster mitigation involves preventive and corrective measures before, during, and after disasters to reduce damage and speedy recovery. Geotechnical solutions such as retrofitting, slope stabilization, deep foundations, and load redistribution are commonly used to address instability (5,6).

The paper presents a case study of restoration of a resort project nestled in the lush, hilly terrain of Coorg, Karnataka, which faced a severe setback during the torrential monsoon rains of August 2018, when devastating landslides wreaked havoc across the state. A suite of practical yet cost-effective stabilization strategies were swiftly implemented at site. These measures not only restored structural integrity but also laid the groundwork for long-term resilience in this vulnerable terrain. While rehabilitation is often the preferred solution, some structures due to severe damage or redevelopment needs require complete removal. In such cases, controlled demolition by implosion presents a viable alternative to traditional methods. Demolition of existing structures is often a critical first step in the process of redevelopment, particularly in urban areas where space is limited and infrastructure must evolve to meet modern needs. It involves the systematic dismantling or destruction of old, outdated, or unsafe buildings to make way for new construction projects that can better serve current economic, environmental, and social demands (7).

2 GEOTECHNICAL CHALLENGES FACED IN CONSTRUCTION OF A RESORT PROJECT AT COORG

2.1 Project Overview



Fig.1. Structural and geotechnical damages caused to the resort project due to landslides

The case study focuses on a luxury resort project in Kodagu district, Karnataka, spanning 35 acres with 50 villas and 70 apartments. The site is characterized by steep slopes, an average annual rainfall of 2800 mm, and vulnerable soil strata comprising loose to medium dense silty sand up to 6 meters in depth. The August 2018 rainfall triggered multiple landslides, causing significant construction setbacks. A site inspection revealed several forms of structural and geotechnical damages including Collapse of road access due to slope failure (Fig.1.a), Toppling of foundations under construction (Fig.1.b), Failure of beam-column junctions (Fig.1.c) and base plate connections (Fig.1.d), Collapse of retaining walls (Fig.1.e), Widespread erosion and vegetation loss (Fig.1.f). These issues highlighted the urgent need for comprehensive rectification and stabilization strategies.

2.2 Mitigation and Rectification Measures

Given the impracticality of constructing a traditional retaining wall on steep terrain, a temporary slope-stabilizing solution using sandbags filled with locally available soil was implemented. Plastic sheets, geotextiles, timber poles and PVC pipe-based drainage systems were used to enhance stability and durability as shown in Fig.2. Toppled isolated foundations were reconstructed using well foundations, chosen over pile foundations due to the logistical challenges of deploying heavy equipment on steep slopes. This approach ensured stability against future scour and erosion. Beam-column joints and base plate connections that failed due to lateral soil movement were retrofitted with additional steel sections and replacement of damaged sections, restoring structural integrity.



Fig. 2. Slope stabilization using sand bags

New retaining walls were built using rubble masonry and Reinforced Cement Concrete (RCC), depending on location and load requirements. These were designed to protect slopes and support adjacent structures. As an additional preventive measure, Rubble masonry packing were provided around RCC footings (Fig.3.a), diagonal bracing (RCC and steel) for slope-facing buildings (Fig.3.b,c), stepped construction to distribute loads and reduce erosion (Fig.3.d), use of lightweight porotherm blocks to reduce structural loads (Fig.3.e), Protection and replanting of vegetation (Fig.3.f) to prevent further soil erosion were adopted. The completed project is shown in Figure 4.



Fig. 3. Additional preventive measures adopted



Fig. 4. The project after rectification and completion

3 PLANNING FOR DEMOLITION OF 26 STOREYED RESIDENTIAL TOWERS AT KOCHI DUE TO STRUCTURAL INSTABILITY

3.1 About the project and Background

The project site is located in Kochi, consists of three towers: Tower A, Tower B, and Tower C, housing a total of 264 apartments. Tower A features a stilt floor plus 14 additional floors, while both Tower B and Tower C are designed with a basement, stilt floor, and 26 floors each. The superstructure of the towers is an RCC framed structure with shear walls, supported by deep piles for stability. The project commenced in April 2013, and by March 2018, the first residents began moving into the completed sections of the building. However, in June 2018, after the floods, serious deterioration of the structures was noted with severe concrete spalling at random locations. In August 2020, following a series of inspections, a subcommittee of engineers recommended repairs and further evaluations to assess the structural safety. This was followed by a scientific report from Bureau Veritas in October 2022, which flagged severe corrosion and instability within the structure. In November 2023, a report from IIT Madras confirmed these findings, indicating a growing risk to the building's integrity. Later that same month, two residents filed a writ petition, urging immediate intervention due to the deteriorating condition of the building. By March 2024, the builder took action by calling for the evacuation of residents and initiating retrofitting efforts. However, in June 2024, experts from IISc Bengaluru conducted a detailed study and recommended the immediate demolition of the towers, citing the advanced level of structural degradation.

3.2 Issue and Mitigation Plan



Fig. 5. Structural issues of the towers noted during inspection

Investigations revealed that the reinforced concrete in the towers had suffered severe corrosion, primarily caused by the high chloride content in the water used during construction as shown in Figure 5. This corrosion compromised the integrity of the concrete, which, in turn, accelerated the degradation of the steel reinforcements within the structure, weakening its overall stability. Finally, in February 2025, after a thorough legal and technical assessment, the High Court issued an order for the demolition and complete reconstruction of the building to safeguard the lives of its residents.

4 DEMOLITION USING IMPLOSION OF SIX STOREYED BUILDING FOR REDEVELOPMENT

4.1 Project Overview and Pre-demolition Works

Mindspace is a techno-campus and IT park located in Madhapur, Hyderabad. As part of redevelopment plans, the owner decided to demolish Buildings 7 and 8, six-storeyed buildings that were 18 years old in order to make way for new commercial construction. Prior to the demolition, a pre and post-blast structural integrity survey, including the assessment of potential damage to adjacent buildings was studied. Rapid visual assessment and non-destructive tests such as rebound hammer, Ultrasonic Pulse Velocity tests and carbonation tests were conducted. The rebound hammer tests done on Buildings indicated the compressive strength of the elements are ranging from 44MPa to 67MPa. The study documented the existing deterioration and cracks in the adjacent buildings before and after demolition. The photographs taken before demolition as part of Rapid Visual Assessment were compared with post-demolition data and photographs.

4.2 Vibration measurement during demolition

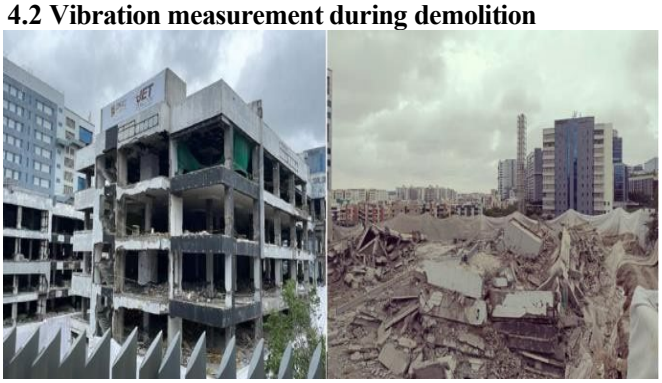


Fig. 6. Buildings 7 and 8 before and after demolition

Prior to the implosion, the structure was pre-weakened by removing hazardous elements, and carefully planned to collapse in a predetermined direction, minimizing debris dispersion. On September 23, 2023, at 8:30 AM, the building was safely brought down through controlled demolition by delayed detonation technology. The buildings before and after demolition is shown in Figure 6. Ground vibrations and dust emissions were recorded using seismographs and dust meters placed at eight strategic locations, in areas of building 6,7,9 and power house as shown in Figure 7.



Fig. 7. Locations of Seismographs and dust meters

In accordance with DGMS guidelines outlined in Technical Circular No. 7 of 1997, (8) the highest vertical peak particle velocity, measured at Location 4, was 2.096 mm/s were well below the allowable limit of 20 mm/s. The maximum air overpressure, also recorded at this location, was 85.3 Hz, which remains within the permissible range. A representative response obtained at Building 9 during the demolition is presented in Figure 8.

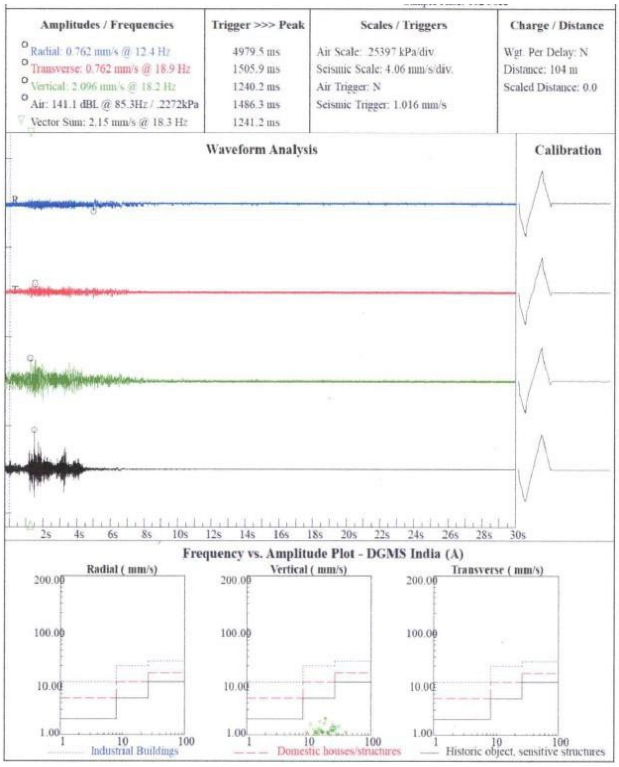


Fig.8. Representative response during demolition

4.3 Post Demolition Works

Following the demolition, a post-blast structural audit was conducted to assess damages if any. A comparative analysis with pre-blast records revealed no damages. As part of sustainable practices, a significant portion of the demolition debris were repurposed, utilizing it as recycled aggregates in the construction of access roads. This approach not only reduced waste but also conserved natural resources. Additionally, recoverable steel components from the debris were carefully segregated and sent them for recycling, ensuring that valuable materials were given a new life and minimizing the environmental impact of the demolition process.

5 CONCLUSIONS

This paper presents case studies illustrating structural rehabilitation and controlled demolition as key strategies in disaster mitigation and urban resilience. With natural disasters intensifying due to climate change and urbanization, the construction industry plays a critical role in both emergency response and proactive risk reduction through geotechnical solutions and structural retrofitting. The restoration of a landslide-affected resort in Coorg demonstrates how cost-effective stabilization can ensure long-term safety. In contrast, controlled implosions of structurally unstable buildings such as a 29 storeyed residential towers in Kochi and implosion of 6 storeyed building at Hyderabad, highlight the importance of precise, controlled delayed detonation techniques for bringing down structurally compromised towers and redevelopment in dense urban areas. These examples underscore the value of combining engineering expertise, technology, and regulatory coordination to manage disaster risks. Both restoration and demolition, when guided by scientific assessment, are essential tools for enhancing resilience, reducing exposure, and enabling sustainable development. Future strategies must incorporate life-cycle planning, early warning systems, and adaptive designs to build safer, future-ready cities.

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Thematic Lecture I:

Statistical and regression analysis of sand and clay stiffness in triaxial tests

Dr. Rafael Sharafutdinov

Statistical and regression analysis of sand and clay stiffness in triaxial tests

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ABSTRACT

Triaxial tests are the common laboratory methods for studying the mechanical properties of soils. According to international practice, it allows determining the reliable strength and stiffness properties. This paper describes the results of statistical analysis of the deformation parameters of sand and clay soils, obtained from triaxial tests. The research focused on sand and clay deposits of the Quaternary, Cretaceous, Jurassic and Carboniferous periods of diverse genesis. The results of 2000 consolidated isotropic drained triaxial tests on sand and clay soils in Russia (Moscow) and Belarus (Minsk) were analyzed. More than 50% of the tests were carried out under unloading/reloading conditions. As a result, empirical equations enabling evaluation of the effects of physical properties and stress state on stiffness of soils with different age and genesis were proposed. The proposed equations can be applied for provisional FEM calculation of the stiffness parameters and can be used in geotechnical models that consider variability, horizontal and vertical distribution of stiffness. Generally, geotechnical engineers may utilize the obtained results to apply them to design of complex soil models.

Keywords: laboratory tests, triaxial test, statistical analysis, stiffness non-linear modelling

MATERIALS & METHOD

Experimental data comprised the results of triaxial tests on soil specimens collected from 16 sites located in Moscow (Russia) and one site located in Minsk (Belorussia). Soil was classified pursuant to GOST 25100. Investigations of clayey soils were conducted on the base of 992 consolidated drained (CD) triaxial tests results. Data for sand investigation were taken from the 962 CD triaxial tests. Samples were collected from the depth of 0.6…99.6 m. On the base of triaxial test results, the following parameters were analyzed: secant modulus of deformation at 50% of strength E_{50}; deformation modulus under unloading–reloading E_{ur}; $k_E = E_{ur}/E_{50}$; Poisson's ratio ν; unloading–reloading Poisson's ratio ν_{ur}.

RESULTS

The correlation analysis revealed that:

- the stiffness of clays E_{50} and E_{ur} considerably depended on σ_3 and *RRS*, as well as on the total amount of clay particles, and on density and moisture content; the stiffness of sand depended on confining pressure and void ratio.

- the stiffness E_{50} and E_{ur} were closely related, however.

- Poisson's ratio ν moderately depended on physical properties of clay, and did not depend on those of sand; Poisson's ratio ν_{ur} was independent from physical and mechanical properties of soil.

- the deformation moduli E_{50} and E_{ur} slightly depended on deformation properties of soil.

- the relative radial stress *RRS* the most appropriately described the relationships between stiffness and stress.

On the base of statistical analysis, a relationship between E_{50} and the relative radial stress *RRS* and physical characteristics of soil was proposed:

- in clay:

$$E_{50} = a_1 RRS + a_2 e + a_3 I_p + a_4 I_L + a_5 \quad (1)$$

- in sand:

$$E_{50} = (ae^b) \cdot RRS^c \quad (2)$$

The empirical coefficients are presented in tables 1 and 2 respectively. The results calculated with equations (1) and (2) well agreed with the data received by external researchers in Russia, Europe, USA, India, and Thailand.

Deep soil layers are subjected to the impact of the variety of stresses. The dependence of stiffness on the stress state of soil is best described by equation:

$$E_{50} = E_{50}^{ref} \left(\frac{c \cot \varphi + \sigma_3}{c \cot \varphi + p_{ref}} \right)^m \quad (3)$$

The results of statistical analysis of power-law coefficient *m* are presented in tables 3 and 4 for clays and sands respectively.

Table 1. The empirical coefficients of the regression equation (3) for E_{50} of clays, in relation to the age and genesis

Coefficients (3)	Q		J	C
	a, f	g		
a_1	14.2	8.6	4.3	9.2
a_2	-6.2	-38.6	-3.5	44.8
a_3	-23.9	-144.6	-6.5	-162.2
a_4	-11	-3.5	-25.7	-76.1
a_5	7.8	38.7	20.7	19.4

Age: *Q* – Quaternary (*a* – alluvial, *f* – fluvioglacial and glaciolacustrine, *g* – glacial), *J* – Jurassic, *C* – Carboniferous

Table 2. The empirical coefficients of the regression equation for E_{50} of sands, in relation to the age and genesis

Coefficients (4)	Q				preQ
	C	M	F	S	F and S
a	31.1	8.1	11.0	12.0	7.76
b	0.05	-1.9	-0.95	-0.69	-1.16
c	0.30	0.73	0.77	0.80	0.43

Age: *Q* – Quaternary; *preQ* – pre-Quaternary
Sand type: C – coarse; M – medium; F – fine; S – silty

Table 3. Power coefficient *m* of clays

Age	Type of soil		*m*
Q *a, f*	Clay		1.13
	Lean clay		1.00
	Silty clay		0.94
g	Lean clay	$I_L \le 0.25$	0.68
		$0.25 < I_L \le 0.50$	0.54
	Silty clay	$I_L < 0$	0.77
		$0 \le I_L \le 0.25$	0.72
J	Clay and Lean clay	$I_L < 0$	0.24
		$0 \le I_L \le 0.25$	0.61
		$0.25 < I_L \le 0.50$	0.62
C	Clay and Lean clay	$I_L < 0$	0.93
		$0 \le I_L \le 0.25$	0.62

Age: Q – Quaternary (a – alluvial, f – fluvioglacial and glaciolacustrine, g – glacial), J – Jurassic, C – Carboniferous

Table 4. Power coefficient *m* of sands

Q				preQ
C	M	F	S	F and S
$e \le 0.55$			$e \le 0.60$	
0.20	0.75	0.63	0.66	0.47
$0.55 < e \le 0.75$		$0.6 < e \le 0.75$	$0,6 < e \le 0,80$	–
0.41	0.85	0.66	0.83	–

Age: Q – Quaternary; preQ – pre-Quaternary
Sand type: C – coarse; M – medium; F – fine; S – silty

It was obtained that k_e of clays was in the range of 2 to 10 and depended on physical properties, *e*, I_P and I_L. For practical application, k_e can be defined with the formulae:

$$k_E = b_1 e + b_2 I_p + b_3 I_L + b_4 \quad (4)$$

Empirical coefficients b_1, b_2, b_3 and b_4 taken in relation to the age and genesis are given in table 5.

It was observed that in sand k_e did not depend on the age and dispersity and might be recommended as $k_E \approx 3.8$.

Statistical analysis of the Poisson's ratio *v* defined its average values as the range of 0.29 to 0.36 for clays and the range of 0.28 to 0.3 for sands. The parameter was slightly variable and slightly depends on the age, type and variations of soil. The same was observed with v_{ur}, which ranged on average from 0.17 to 0.20 in clays and sands.

Table 5 The empirical coefficients of the regression equation for k_E in relation to the age and genesis of deposit

Coefficients (5)	Q		J	C
	a,f	g		
b_1	-2.5	-4.8	-4.5	1.9
b_2	-2.2	-52.4	2.5	-17.3
b_3	1.3	2.1	7.2	2.7
b_4	7.8	13.1	7	6.5

Age: Q – Quaternary (a – alluvial, f – fluvioglacial and glaciolacustrine, g – glacial), J – Jurassic, C – Carboniferous

DISCUSSION

The defined dependencies develop the conception of the mechanical behavior of dispersed soils as a generalized function of soil stress state and physical properties. The dependencies allow to evaluate the stifffness of deepen soil layers and can subsequently become the basis for analytical solutions.

The vertical deformation of the arbitrary soil layer can be calculated with respect to dependency of the stiffness on stress state by formulae:

$$\varepsilon_z = (1 - 2\nu\xi) \int_0^{\sigma_{zp}} \frac{d\sigma_{zp}}{E_{ref} \left(\frac{c \cdot \cot\varphi + p'}{c \cdot \cot\varphi + p_{ref}} \right)^m} \quad (5)$$

where, $\xi = \Delta\sigma_h / \Delta\sigma_v$ is an at-rest pressure coefficient.

The following dependence represents the relative strain with respect to changes in stiffness under applied stress:

$$\varepsilon_z = aD \frac{\left(b + \chi\sigma_{zp}\right)^{1-m} - b^{1-m}}{\chi(1-m)} \quad (6)$$

where $D = (1 - 2\nu\xi); H = c \cdot \cot\varphi; a = \frac{\left(H + p_{ref}\right)^m}{E_{ref}}$;

$b = H + \sigma_{zg} \frac{(1 + 2K_0)}{3}; \chi = \frac{(1 + 2\xi)}{3}; K_0 = \frac{\sigma_{hg}}{\sigma_{zg}}$; σ_{hg} and σ_{zg} is horisontal and vertical earth pressure in soil mass.

The approach (6) is capable of predicting foundation settlement with respect to the changes in soil stress state under load. An analytical solution for calculating foundation settlement with taking into account the dependence of soil stiffness on the stress state was proposed. It was shown that in the comparison between the numerical solution and pile test data or long-term observations of high-rise building and bridge pile foundation settlements the discrepancy was less than 10-20 %

Thematic Lecture Ⅱ:

Hazardous Geological Processes and Their Impact on the Condition of Architectural Monuments

Prof. Fayzulla Abdullaevich Ikramov

Hazardous Geological Processes and Their Impact on the Condition of Architectural Monuments (Samarkand, Uzbekistan)

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Abstract

Monitoring and observation of landslides have been actively conducted and significantly developed over the past 20–25 years. Research is being carried out in collaboration with the state service of the Samarkand Observation Station (Uzbekistan) for hazardous geological processes. Slope instability (landslides, rockfalls, subsidence, suffusion, karst) is widespread in the mountainous and foothill regions of the Zeravshan Oasis and also in the city of Samarkand, home to unique world heritage architectural monuments. Landslides cause serious complications, pose dangers to human life, and destroy cultural heritage sites. In recent years, due to heavy rainfall, earthquakes, and human intervention in the natural environment, emergency situations have arisen, accompanied by significant material losses and human casualties.

Keywords: landslides, stability, seismic hazard, risk, monitoring.

Introduction

The influence of earthquakes on landform development, slope deformation, and rockfalls has been studied by V.P. Solonenko (1973) and N.I. Nikolaeva (1975). The nature of residual deformations caused by earthquakes was addressed in the works of E.V. Popova (1974), E.V. Dedova (1966), and A. Khodjaev (1985). The formation of modern landslides has been studied by G.S. Zolotaryov (1986), B. Saifiddinov (2002), R.A. Niyazov (2009), among others.
In Uzbekistan, earlier studies describing the effects of major earthquakes in mountainous areas mainly focused on building deformation or destruction, as well as various fault ruptures, and rarely addressed the landslide or rockfall phenomena triggered by earthquakes. This is primarily due to the fact that many of the destructive earthquakes that caused landslides occurred in the 17th and early 19th centuries, when timely and thorough surveys in mountainous regions were not possible.

Hazardous Geological Processes and Their Impact on the Condition of Samarkand's Architectural Monuments

The engineering and geological conditions of the Samarkand region and Zeravshan Oasis are characterized by natural variation in geological factors from south to north. The central part of Samarkand, covering an area of 25 km², contains the largest historical architectural monuments. The terrain is relatively flat, with an elevation of 690–750 meters above sea level and a general slope of 0.005°. The aeration zone consists of sandy loams and loess-like clays.
In the city center, the thickness of anthropogenic soils ranges from 4 to 14.5 meters. Beneath these lie loess subsidence soils of type II. The groundwater depth near the Tilla-Kari Madrasa is about 17 meters. The porosity of loess-like clays is 37.0–51.6%. Naturally, high-porosity loess soils undergo significant subsidence deformation when saturated.
The constant and temporary water flows of ancient canals (Dargom, Siabcha, Siyob, Obimashat, Zhar, etc.) in the floodplains of the Zeravshan River contribute to active erosion. With vertical canal banks reaching heights of 15–35 meters, this erosion leads to large collapses and landslides.

Monitoring and observations have been conducted at numerous unique architectural monuments, minarets, nearby residential buildings, and areas inhabited by people in ravines and near springs:

1. **Ulugbek Madrasa** (1417–1430): The building's height from the square level is 23–25 m, width 8–10 m, with majestic pointed arched portals. Minarets at the corners reach 30–35 m, with high hujras measuring 5 m in height and 3–5 m in length and width. Monitoring results show no hazardous geological processes were recorded.
2. **Sherdor Madrasa** (1619–1636): Built by order of Samarkand ruler Bahodir Yalangtush. Externally, Sher-Dor Madrasa resembles Ulugh Beg

Madrasa. A comprehensive study confirmed the absence of hazardous geological processes.

3. **Tilla-Kari Madrasa** (constructed in 1646): Built 10 years after Sher-Dor by the same ruler. "Tilla-Kari" means "decorated with gold." Besides functioning as a madrasa, it also served as a congregational mosque.
In March 1993, vertical cracks appeared on the western exterior corner of the gallery entrance: 0.8–1.5 m long, 2–4 cm wide and 2 cm deep.
During renovation, serious mistakes were made: the clay-saman roofing was replaced with galvanized metal; the asphalt around the building was removed and replaced with burnt bricks on sand bedding.
As a result, rainwater infiltrated under the foundation. Heavy rainfall caused significant subsidence deformation. Monitoring was conducted using markers and sensors.
Between 1993–1995, the total length of the crack reached 2 m, with a width of 6 cm and visible depth of 4 cm. Foundation support was reinforced with cantilevered bored piles.
At present, the monument is under monitoring and is operating without risk.

4. **Mausoleum of Saint Daniel** – One of the oldest architectural monuments.
Located at an elevation of 700 m above sea level, on the left bank of the Siabcha Canal.
A 10–30 m high vertical slope composed of loess-like clays, sandy loams, and yellow sand lines the roadside near the entrance.
At the top of the slope are orchards and agricultural lands.
Rainfall, irrigation, slope instability, and excavation during road construction in 2009, 2012, 2017, and 2020–2022 caused landslides and rockfalls.
A cave was dug into the slope near the shrine entrance, above which two landslide blocks (250–300 m³) are suspended, posing a real threat of collapse.

Conclusion

It can be concluded that the development of mountainous and foothill areas without prior research to assess slope stability and without following expert protective recommendations has become one of the main factors in landslide formation.

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Thematic Lecture Ⅲ:

Geoinformational zoning in addressing hazards from natural disasters

Dr. Daulet Sarsenbayev

Geoinformation zoning in solving problems of countering dangerous natural processes

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ABSTRACT

Geographic information zoning is a continuous process aimed at identifying areas most at risk of destruction in the event of a sudden natural disaster.
As part of the solution to the tasks set, the work examines methods for collecting, summarizing, and modeling spatial data using geographic information systems (GIS) and spatial analysis of relevant data to create vulnerability maps of territories.
In particular, the use of methods for visualizing observed seismic and geophysical data, the results of their modeling, and forecasting natural disasters using intelligent geographic information systems and artificial intelligence (AI). It also involves the use of remote sensing monitoring data on the geodynamic state of the Earth's surface and structures.
An important place is given to the integration of GIS into the emergency warning system for the population and the adoption of operational management decisions.
The analysis will provide a more complete picture of narrowly focused GIS research and the construction of corresponding maps of earthquake-prone and flood-prone areas, as well as mudslides and landslides in mountainous areas.

Keywords: geoinformation system, forecasting natural disaster, earthquake-prone, remote sensing

1 INTRODUCTION

All dangerous natural phenomena are the result of the natural evolution of continuous geodynamic processes of an endogenic, exogenic, and technogenic nature. Therefore, the fight against natural disasters in general and in a particular country should be systematic, aimed at creating a sustainable economy that takes into account possible dangers and is capable of preventing them.

According to estimates by the German insurance company Munich Re Group (2007), the number of natural disasters in the world is steadily increasing (Figure 1). At the same time, the amount of economic damage is growing twice as fast, which indicates a relatively low level of protection of both infrastructure and the population itself [1].

Another objective reason for this situation may be the rapid growth of the world's population, which has doubled over the past 100 years and stood at ~4 billion people in 1975. Such intensive population growth objectively leads to the development of unsuitable territories that require well-thought-out engineering solutions.

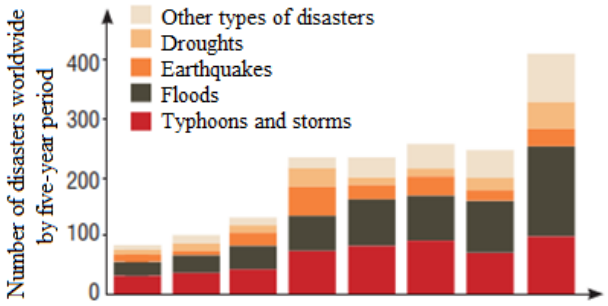


Fig. 1. Number of disasters worldwide by five-year period

As Osipov (2010) notes, industrial development "has come into conflict with the natural environment, as it has ceased to take into account the real limits of the biosphere's sustainability" [1].

However, since technological progress cannot be stopped, the task of preventing natural disasters and mitigating their negative consequences has come to the fore. Solving this problem requires assessing the probability of a particular event and, as a result, calculating the vulnerability index of the territory and the objects located on it. Vulnerability is expressed in relative units or percentages. One of the methods for reducing the risk of earthquake consequences in cities is well-thought-out engineering solutions in the

planning of infrastructure facilities and construction itself.

One of the strategies aimed at preventing the harmful consequences of geocatastrophes is the creation of a geoinformation base in the form of interactive maps of hazardous natural processes, which provide an idea of the potential danger of territories where economic facilities are already located or are planned to be built. The creation of a GIS aimed at forecasting emergencies would reduce material losses by 15 times [2].

According to estimates given in the work of Kuzmin (2019) [3], Kazakhstan is one of the countries with a low level of protection against natural disasters (protection coefficient V_c=2.05, environmental stress coefficient E=0.8), which is ~ (10 - 40) times lower than the level of developed countries in Europe and the world as a whole. Protection against natural disasters is understood here as "the potential ability of management structures to withstand dangerous natural processes, which is composed of political, economic, social, medical-ecological, organizational, geopolitical, demographic, and other indicators" [3].

A significant part of Kazakhstan's territory and its industrially developed city of Almaty are located in a potentially seismic zone in the foothills of the Tien Shan mountains. Therefore, for the southern region of Kazakhstan, earthquakes are the most dangerous type of geological disaster with difficult-to-predict consequences.

As an example, we can compare the consequences of two earthquakes of equal intensity that occurred in the 1980s – in Spitak (Armenia) and San Francisco (California, USA). In the first case, 40,000 people died and the cities of Spitak and Leninakan were significantly damaged, while in the second case, only 40 people died.

The reason for this is the difference in the quality of the building structures used and in the organization of preventive measures [1].

2 GEOGRAPHIC INFORMATION SYSTEMS, AS A SYSTEM FOR COLLECTING DATA, PROCESSING IT, MODELING AND FORECASTING NATURAL PROCESSES

Geographic information systems (GIS), as a tool for spatial analysis of large amounts of geospatial information, are successfully used to solve problems related to emergencies in all their diversity.

The emergency situation geographic information system (ES GIS) allows data to be collected and linked to specific locations, and enables analysis and modeling of emergency situations with forecasts of possible consequences and options for their prevention. Thus, ES GIS provides the opportunity to make the necessary management decisions and create your own comprehensive cartographic support.

2.1 Structure of the emergency situation geographic information system

One of the indisputable advantages of GIS is its ability to process large arrays of diverse digital coordinate-linked data, which can be used to detect adverse or dangerous processes occurring in the natural environment, followed by the development of recommendations to reduce possible adverse consequences.

ES GIS, as an automated information system, combines a number of technologies known as automated scientific research systems (ASRS), automated design systems (ADS), automated reference and information systems (ARIS), etc. [4].

All these technologies allow for various thematic analyses of databases with subsequent full visualization provided by a map. This circumstance distinguishes GIS from other information systems and provides unique opportunities for their application in solving problems related to ES prevention and response.

Currently, a variety of software based on geoinformation technologies is actively used to improve the effectiveness of ES prevention and response measures.

Based on this, the GIS structure consists of the following blocks [5]:

– a database block containing cartographic and semantic data in the form of various maps. This block is responsible for the rapid acquisition and systematization of data for the purpose of organizing it for subsequent analysis;
– an output data and documentation block;
– an analytical block for assessing consequences and developing risk reduction measures.

The block provides analysis of source data and generates qualitatively new resulting information in a form that is optimal for preparing management decisions. This predetermines the presence of two sub-blocks in the emergency situation forecasting system, which can be spatially and organizationally distributed between organizations that perform data analysis and processing:

– a sub-unit of data processing methods and algorithms related to the regulatory, legal, and methodological support of forecasting and eliminating the consequences of emergencies;
– a sub-unit of modeling systems that actually process data based on specialized software and hardware complexes that allow for forecasting the situation and assessing the danger of natural and man-made impacts.

Finally, due to the need for prompt management decisions, the ES GIS includes a support system (block) designed to reduce the time required to develop and adopt optimal decisions on ES response, minimizing casualties and environmental damage.

The system allows for the coordination and management of departments and ensures information exchange between all participants in ES response operations.

Given the variety of types of ES, GIS can include specialized analytical models of geographic information systems that allow for the rapid assessment of the actual state of the situation, the prediction of development trends, the analysis of the possible consequences of management decisions, and the creation of maps of areas that may be affected by certain factors.

3 EARTHQUAKES AND THE USE OF GIS TECHNOLOGIES

An earthquake is a geodynamic process of instantaneous release of stress accumulated during the evolution of the Earth's crust.

To date, all studies based on empirical data in the search for seismological, statistical, biological, ionospheric, remote sensing, and other patterns of precursors to a possible earthquake have not proven effective.

3.1 Goals and objectives

To reduce the risks of suddenness and mitigate the consequences of a possible earthquake using GIS technologies, it is necessary to continuously collect, process, and analyze data, followed by dynamic modeling of the development of possible events using numerical and mathematical methods [6,8,9].

In general, the modeling task involves [6]:

1) defining the basic system of differential equations characterizing the state of a natural phenomenon with the corresponding initial and boundary conditions;
2) selecting a numerical or analytical (if possible) method for solving the given boundary value problem;
3) identification of the main limitations of the developed model and verification of its adequacy.

The main requirements for the developed dynamic model of a natural phenomenon are the consistency of the obtained results with real data (compliance with physical laws and correct representation of the essence of the phenomenon under study), as well as the accuracy, reliability, and speed of the numerical calculations performed.

3.2 Risk assessment

Risk assessment is a numerical characteristic of the probability of an earthquake of a given intensity and the possible damage in a particular area.

To solve this problem, GIS should use deterministic and probabilistic models and methods with various indicators that allow assessing the scale of possible damage [6].

With a unified intensity assessment scale based on a set of indicators, digital thematic maps will be created that reflect the degree of earthquake hazard depending on the depth of the epicenter and its location relative to the object.

The latter is a multifactorial task requiring additional research on natural disasters with the creation of global databases, without which it is difficult to implement an early warning system for earthquakes.

On this basis, the task of identifying the moment of transition of the system to a new geodynamic equilibrium with all the ensuing consequences must first be solved.

To achieve this goal with the help of GIS technologies, it is necessary to use not only seismic, geophysical, and geodynamic data, including data from radar monitoring of the territory or object and remote sensing in general, but it is also desirable to use intelligent support for computational processes. That is, to involve intelligent geographic information systems focused on modeling and predicting natural disasters.

4 ARTIFICIAL INTELLIGENCES

Artificial intelligence (AI) is currently very popular as a tool for geoinformation zoning due to its ability to process large amounts of data and analyze complex models.

The use of artificial intelligence in natural disaster management significantly improves the efficiency and accuracy of forecasting, as well as contributing to more effective risk management and reducing the potential damage from such events.

The main tasks of artificial intelligence in natural disaster management include:

- predicting the occurrence and development of natural disasters based on the analysis of historical data, meteorological and geological parameters;
- assessing potential threats and risks associated with natural disasters;
- developing models and scenarios to assess the consequences of natural disasters and determine the best ways to prevent or mitigate them;
- monitoring and analyzing data in real time to respond quickly to natural disasters;
- automating decision-making processes and optimizing resources in natural disaster management.

4.1 Predicting natural disasters

Predicting natural disasters is one of the most important areas of AI application (A1) in managing the risks of natural disasters, including earthquakes.

AI algorithms are adapted to analyze large amounts of data, including data from seismic stations, in order to identify signs preceding natural disasters.

This analysis allows:

– creating models and algorithms capable of predicting the probability of a disaster occurring
– and its possible consequences.
– warning the population and taking measures to mitigate risks.

For example, with the help of AI, it is possible to:

– develop early warning systems and automatic evacuation systems.
– optimize resource allocation and rescue operation coordination processes.

However, the accuracy of predictions may depend on the availability and quality of data, as well as the algorithms and models used by AI.

4.2 Disaster prevention and mitigation

Using machine learning algorithms and big data analysis, AI can identify and predict natural disasters, as well as suggest measures to prevent them and reduce their impact on people and the environment.

Together, this allows for the creation of probabilistic models that can predict the occurrence of disasters and forecast the scale of potential damage.

Such predictions enable preventive measures to be taken to minimize losses and save human lives.

However, despite all its advantages, AI cannot completely replace the human factor in natural disaster management.

Human understanding and expertise are still necessary for analyzing and interpreting data, making decisions, and coordinating actions. Therefore, the effective use of AI requires close interaction between people and technology.

4.3 Automation of natural disaster forecasting processes using artificial intelligence

Thanks to its ability to process large volumes of data, AI can analyze information from various sources and make predictions based on statistical models, as well as assess their potential consequences.

AI enables the development of systems for monitoring and detecting both earthquakes and other natural disasters. The automation of data collection and analysis processes allows it to respond quickly to changes in the situation and take the necessary measures to prevent or reduce damage.

Optimizing natural disaster management with AI involves developing systems for forecasting and modeling various scenarios, followed by an assessment of the effectiveness of the proposed measures.

AI systems allow analyzing data on previous disasters, assessing their consequences, and proposing optimal strategies to prevent similar situations in the future. For example, AI can help in planning urban infrastructure, taking into account possible natural disasters.

4.4 Prospects for the use of artificial intelligence in combating natural disasters

AI offers new perspectives in the fight against natural disasters. With its help, it is possible to predict and reduce the risks associated with phenomena such as earthquakes, floods, hurricanes, and fires.

AI improves the accuracy, speed, and probability of predicting natural disasters. It can also help reduce risks and manage the consequences of disasters by providing real-time information on the spread of disaster routes.

This enables effective measures to be taken to evacuate the population, deploy rescue services, and prepare regions for possible consequences.

AI's ability to analyze and process huge amounts of data in real time allows for rapid response to changes and informed decision-making.

Overall, the use of artificial intelligence in combating natural disasters has great potential to improve the safety and protection of the population.

However, it is necessary to consider the ethical and legal aspects associated with the use of AI to ensure fair and reliable decision-making in complex situations related to natural disasters.

5 GEODYNAMICS, CARTOGRAPHY, AND GEOGRAPHIC INFORMATION SYSTEMS

Earthquakes and their consequences are the final stage of geodynamic processes occurring in the Earth's crust.

Geodynamic processes are shifts in points on the Earth's surface, the monitoring of which requires cartographic support and cannot be implemented without the use of an appropriate GIS.

The created GIS structure “Geodynamics” [7] is the optimal method for cartographic research of geodynamic problems. It is focused on creating maps that allow identifying spatial patterns that may be indicative of geodynamic processes and provides the opportunity to carry out both fundamental and operational cartography.

The following functions are implemented in the Geodynamics GIS:

– storage and updating of source data;
– computational processing and analysis of data obtained using various methods of space geodesy;
– interpretation of the results obtained and mapping (graphical constructions of varying degrees of complexity) [7].

It includes three series of cards:

1) earthquake maps;
2) maps of velocities of points on the Earth's surface (for different methods of space geodesy and joint);
3) maps of velocities and discrepancies for the Eurasian and North American plates.

Thus, GIS provides an opportunity to visualize the location of earthquake hypocenters in specific deep zones and allows highlighting certain features of their spatial localization both in terms of plan and depth.

In addition to the distribution and location of hypocenters by depth, GIS allows you to create maps of the spatial distribution of hypocenters by magnitude. These maps allow you to trace the process of formation and subsequent destruction of linear structures of earthquake hypocenters at different magnitudes and identify the features of this process in different seismic zones.

It should be emphasized that two directions should be developed in geodynamic mapping: fundamental and operational.

In the first case, the content of the maps reflects the most stable, mainly qualitative characteristics. They are created on the basis of geological and geophysical information with partial use of data from space geodesy systems and do not require systematic updating.

Maps of the second direction provide monitoring of geodynamic processes; operational seismological information, data from space geodesy systems, and the results of geodynamic modeling are used to create them. The content elements of these maps mainly have quantitative estimates [7].

The full development of geodynamic mapping, especially its operational direction, is impossible without the use of GIS technologies. Only under these conditions can, in particular, periodic mapping of velocity indicators be organized, allowing changes to be recorded [7].

6 MATHEMATICAL MODELING OF GEODYNAMIC PROCESSES AND GEOGRAPHIC INFORMATION SYSTEMS

The overview would be incomplete without mentioning such an important area of geodynamics as modeling. Since all maps are essentially the result of modeling certain data, the question of reliability arises.

Modern information and instrumental capabilities, as well as the acquisition of new experimental data, make it possible to test existing models, propose algorithms for their improvement, and develop the most visual forms of representation using GIS software technologies [8].

In particular, the introduction of satellite technologies into geodetic practice has made it possible to significantly expand the area coverage of the Earth's surface for the organization of geodynamic monitoring of crustal deformations.

The IGS (International GNSS Service) network is a network of continuously operating base stations located on various tectonic plates, which allows their movements to be continuously recorded. This makes it possible to compare the results of measurements obtained by other methods [8].

Modeling of geodynamic processes is carried out everywhere, although this is a rather complex problem, since it involves modeling physical phenomena that usually lead to the solution of incorrect mathematical problems [8].

As an example, we note the algorithm created by the authors of [8] for modeling modern vertical movements of the Earth's crust and software that allows the results of modeling measurements of the vertical displacement of points on the Earth's surface to be presented using a GIS project, the source data for which was geodetic, geophysical, seismological, and geological data.

In this way, a comprehensive multidisciplinary approach can be implemented in solving problems of geodynamics and ES in general.

7 CONCLUSIONS

GIS are a means of accumulating, processing, analyzing, presenting, and displaying geospatial data.

Today, they are the most promising methods for processing and assimilating large amounts of information about the state of the natural environment.

The tasks of forecasting and eliminating ES have so far remained outside the scope of GIS technology development. Specialized GIS are developed by narrow-profile specialists, which leads to the development of separate special programs for each type of ES [9].

Modern geoinformation systems for monitoring and forecasting hazardous natural events and their consequences are multifaceted information systems that include [9]:

- means of monitoring hazardous phenomena (sensor networks);
- communication channels and equipment;
- powerful databases and knowledge bases containing information about the consequences of various emergencies;
- mathematical models of hazardous phenomena;
- descriptions of the state and vulnerability of risk elements;
- data on the distribution and capabilities of sources of danger;
- interconnected computing resources, etc.

GIS significantly facilitates the process of making management decisions.

GIS uses a raster base, i.e., a scanned map or fragments thereof. The basis for the effective functioning of any geographic information system is the availability of reliable source data. The accuracy and reliability of the GIS data obtained depends on the quality of the source cartographic and geodetic information.

Collecting data to populate the GIS is the most important stage in creating a GIS. The effectiveness and quality of a geographic information system depends on the accuracy, reliability, and relevance of the data collected [9].

GIS is not a tool for making decisions, but a means of accelerating and improving the efficiency of the decision-making process, providing answers to queries and functions for analyzing spatial and attribute data, and presenting the results of the analysis in a clear and easy-to-understand form.

When using GIS, the information required for decision-making can be presented in a concise cartographic form with additional text explanations, graphs, and diagrams.

The availability of information that is accessible for perception and generalization allows responsible employees to focus their efforts on finding a solution without spending significant time collecting and interpreting heterogeneous data. Several solution options can be reviewed fairly quickly and the most effective one can be selected. In addition, the visualization of spatial data using GIS allows for the evaluation of large amounts of data as a whole [9].

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Special Lecture IV:

Recent advancement in “Press-in” piling technology: contribution to disaster prevention and mitigation, and attempts for a rational project cycle using piling data

Yukihiro ISHIHARA, Koichi OKADA
, Yukinori TODA, Atsushi MORI
, Masashi EGUCHI and Naoki SUZUKI

Bright Spark Lecture I:

Environmental assessment of pyrolyzed industrial mixed plastic waste

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, Assem Askarbay

Environmental assessment of pyrolyzed industrial mixed plastic waste

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ABSTRACT

Recently, plastic-related wastes became significant problem worldwide, especially industrial mixed plastic. In this study pyrolyzed industrial mixed plastic waste is considered and its environmental properties are investigated. Environmental characteristics of man-made geo-materials are very important in order to utilize for geotechnical purposes. Especially, leaching characterization is crucial when thermally treated waste material is used as geo-material that has direct contact with soil and groundwater. The aim of this research was to analyze short-term leaching behavior of a sample of pyrolyzed nonrecyclable industrial plastic waste. Concentrations of environmentally problematic leached elements As, Cd, Cr, Cu, B, Pb, Se and Zn were analyzed using the method of the Japan Environmental Agency Notification No. 46: JLT-46. According to batch-leaching test results using inductively coupled plasma mass spectrometry (ICP-MS), concentrations of these elements, except As and Se, are below JLT- allowable limits. Cr and Pb were not detected in the mixed-plastic sample. Overall results suggested that the amount of the selected elements in the final leachates of the test was lower than the limit values of landfill regulations.

Keywords: pyrolysis, leaching behavior, industrial plastic waste, batch-leaching test

1 INTRODUCTION

Plastic waste is a significant problem of modern Japanese society, especially nonrecyclable industrial mixed plastic. Home appliances, vehicles, furniture and innumerable other goods use various polymer combinations, additives, metals and fillers to produce plastics with desired properties, but the highly variable compositions exclude these plastics from recycling since recycling requires separation by single-polymer type.

Landfilling vast volumes of plastic waste is impractical since it requires significant land space. Therefore, thermally treating such plastic waste and putting the treated waste to beneficial geotechnical use is a potentially practical solution.

Pyrolysis of mixed-plastic waste for production of liquid oil – with solid char as a byproduct – has had limited success, because compositional variability in batches of mixed plastics results in variable decomposition temperatures within a single batch (Haruon, 2013; Pandey et al., 2020). On the other hand, pyrolysis of mixed-plastic waste for geotechnical use does not have the goal of recovering liquid oil. Rather, it uses long-duration thermochemical decomposition purely for the production of char.

2 MATERIALS AND METHODS

2.1 Industrial mixed plastic

A sample comprising a mixture of numerous types of plastics collected from multiple sources was received from an industrial waste disposal company. The sample of mixed plastics was batch-crushed into smaller pieces of predominantly 5-15 mm diameter. According to previous studies, priority of ash production can be obtained by longer exposure (minutes to hours) combined with a lower temperature range of 400-600°C (Moharir et al., 2019). The sample was treated pyrolytically at 600°C for 3 hours in the absence of oxygen to produce pyrolytic char. General representations of original and pyrolyzed mixed-plastic waste material are shown in Figure 1.

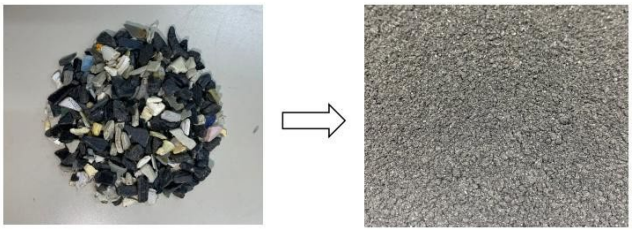


Fig. 1. General representation of original and pyrolyzed mixed plastic waste material

2.2 Batch leaching test

Leaching characteristics of pyrolytic ash were determined following the standard Japanese leaching test (JLT-46). In 500 ml of polyethylene bottle, 20 g of treated sample was mixed with 200 ml of distilled water to form a 1:10 solid-liquid ratio. The mixture was shaken for 6 hours at 200 rpm at room temperature and atmospheric pressure using a reciprocating horizontal shaker to dissolve water-soluble chemical species. After shaking, liquids were separated from solids at 3000 rpm for 20 minutes in a centrifuge. The separated liquid was filtered through a 0.45µm membrane filter. Separated extract was diluted as needed. Concentrations of aforementioned elements (As, Cd, Cr, Cu, B, Pb, Se, Zn) were analyzed by inductively coupled plasma-atomic emission spectrometry (ICP-AES) and inductively coupled plasma-mass spectrometry (ICP-MS). The results were compared with the limit leaching values according to the JLT-46 Soil elution standard. Pictorial representation of typical steps for short-term leaching test is shown in figure 2.



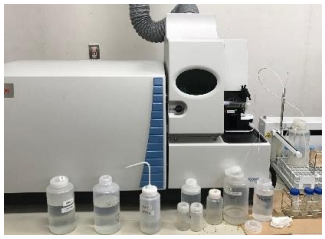


Fig. 2. Batch leaching test steps: shaking, separating with centrifuge and ICP- AES and ICP-MS analysis

3 RESULTS AND DISCUSSIONS

Results of batch leaching test of the sample are presented in Table 1. Leached concentrations of elements except As and Se are less than JLT-46 limit values. Cd, Cr and Pb leached concentrations are very low. Cr and Pb were not detected in the sample using ICP-MS. However, As and Se leaching behavior in the sample of mixed plastics and disposable diapers are over the limit concentrations. In the past, geotechnical applications have employed attenuation layers to prevent As leaching. Compared to previous studies, our pyrolysis ash is safer than incinerated ash materials except for As and Se leaching. Heavy metals including As and Se pose a contamination threat to soil and underground water, therefore pretreatment of pyrolyzed plastic ash and the use of additives can play a crucial role in avoiding environmental contamination. Several techniques such as site grading, compaction, use of an attenuation layer, and controlling surface-water exposure can be used to prevent heavy-metal leaching.

Table 1. Batch leaching test results for mixed plastic and disposable diapers

Elements	Short-term leaching test results (mg/L)	JLT-46 (mg/L)	Analytical method
Cd	0.5*10-3	0.01	ICP-MS
Cr	ND	0.05	ICP-MS
Se	0.55	0.01	ICP-AES
Pb	ND	0.01	ICP-MS
As	0.04	0.01	ICP-MS
B	0.77	1.00	ICP-AES
Cu	0.02		ICP-MS
Zn	0.46		ICP-MS

4 CONCLUSIONS

Pyrolyzed non-recyclable mixed-plastic waste materials were investigated for suitability for geotechnical use. The thermally treated waste material was studied for its short term leaching characteristics. From leaching test, it was found that leached heavy-metal concentrations are below JLT-46 leaching limit values except for As and Se in the sample. Therefore, countermeasures for preventing leachate of these two elements, As and Se, should be considered with using techniques such as site grading, addition of an attenuating layer, controlling contact with surface water, etc.

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Bright Spark Lecture Ⅱ:

Stabilization and Ecological Restoration of Tailings: Addressing Key Challenges and Future Opportunities

Ph.D. Bharat Rattan, Ankit GARG

, Sreedeep S, Chen RUI

Effects of Water-Absorbing Polymers and Vegetation on the Behavior of Unsaturated Soils

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ABSTRACT

It is well known that water retention in unsaturated soils plays a critical role in bioengineered slopes, urban green infrastructure, and agricultural water-use efficiency. Enhancing water retention properties through unique soil conditioners (such as water absorbing polymer)—particularly those capable of storing and gradually releasing water during water scarcity. However, a comprehensive investigation of WAP influence on soil properties and plant response is remains limited. The main objective of this study is to investigate the interactions between soil, WAP, vegetation, and the atmosphere under drought conditions. Although there is ample evidence that plant physiological parameters, such as stomatal conductance (SC) and photosynthetic yield (PY), change under water stress, the relationships between these changes and unsaturated soil properties remain unclear. We evaluate drought stress progression by correlating plant physiological characteristics with soil suction (SS) to address this gap. Results indicate that water-absorbing polymers (WAPs) can effectively mitigate the adverse effects of water stress as a soil amendment. Plant physiological parameters (including SC and PY) declined more slowly in WAP-amended soils than in untreated soils, indicating a reduction in plant water stress under drought conditions. These findings highlight the potential for WAP-amended soils in various applications, including urban green infrastructure, bioengineered slopes, and agriculture in arid and semi-arid regions.

Keywords: water-absorbing polymer (WAP), WAP-soil-vegetation-atmosphere interaction, SWCC, stomatal conductance, photosynthetic yield

1 INTRODUCTION

Water scarcity, driven by global warming associated with climate change, is a key challenge, impacting countries' economic and environmental aspects. These conditions often affect the physical and water-holding properties of soils. The water-holding capacity of unsaturated soils is known to play a crucial role in slope bioengineering, urban green infrastructure, and agricultural soil water-use efficiency. It is well known that water retention of unsaturated soil plays an important role in the bioengineering of slopes, urban green infrastructure, and water use efficiency of agricultural soil. The water retention properties of unsaturated soil were quantified by the soil-water characteristics curve (SWCC), which is the relationship between water content and soil suction. The knowledge of SWCC is central to understanding how easily water is lost from the soil during drying. Additionally, SWCC was used in this study to understand the contribution of water-absorbing polymer in enhancing the water retention/ storage within soil pores. It is understood that the water retention characteristics can be improved by adding innovative soil conditioners that can store more water and use it during dry spells.

The water-absorbing polymer (WAP) is one of the viable soil amendments capable of improving the water storage in soil pores and releasing it during water stress conditions. To establish its utility, it is important to investigate its impact on soil properties and plant response thoroughly. The main objective of this research is to explore soil-WAP-vegetation-atmosphere interaction under drought or water stress conditions systematically. The WAP interaction with soil and external ionic materials (such as fertilizers) significantly affects their water-absorbing capacity and overall performance. Therefore, the combined interaction of WAP-fertilizers and WAP degradation may inhibit the functionality of WAP, which needs to be thoroughly investigated by observing the changes in the SWCC of WAP-amended soil. It is well-known that plant physiological parameters (stomatal conductance (SC) and photosynthetic yield (PY)) undergo changes during the period of drought stress.

However, there is a lack of understanding on how these changes (SC and PY) can be linked with the unsaturated soil properties. Therefore, this study investigated the water stress by establishing the relationship between plants' physiological characteristics (SC and PY) and soil suction (SS).

2 METHODS

Pot experiments were conducted to evaluate the efficacy of fly ash-based water-absorbent polymer and commercial WAP on vegetation. This study was conducted at the Indian Institute of Technology Guwahati using locally available soil. FA-WAP was synthesized in-house, and commercial WAP was procured from the local market. Based on the literature, the WAP concentrations were selected as 0.2% and 0.4%, and a control group (no WAP addition) was set up for experimentation.

The SWCC is an effective tool to quantify PAWC in the root zone by measuring the soil suction (ψ) and volumetric water content continuously under water stress conditions (drying). The soil suction and water content were monitored using TEROS 21 and EC5 sensors installed in the pot. The collected data were analyzed to determine soil water content (SWC) and plant available water (PAWC).

Measurement of Stomatal conductance (SC): The Leaf Porometer measures SC by placing the conductance of a leaf in series with two known conductance elements and comparing the humidity measurements between them. It is a battery-operated, menu-driven device used to measure stomatal conductance of leaves. *Measurement of photosynthetic yield:* The photosynthetic yield (PY) analyzer MINI-PAM-II has been designed for the highly sensitive saturation pulse analysis of photosystem II (PS II) for field and laboratory conditions. MINI-PAM-II determines how environmental factors modulate the efficiency of the conversion of measuring light into fluorescence.

3 RESULTS

Figure 1 depicts the effect of different combinations of vegetation and WAPs on the SWCC during the drying curve. Monitoring started once the plant reached the vegetative stage to ensure proper plant root growth and development. The results showed that vegetation and WAP improved water retention properties more than the control soil. Water retention capacity obtained from the estimated SWCC is listed in the following order: CS < CS_WV < Com-WAP < Com-WAP_WV < FA-WAP < FA-WAP_WV. The presence of vegetation improves the SWCC for all treatments, as shown in **Figure 1**. The increase in SWCC in vegetation remains consistent across the entire suction value range. Water retention properties are enhanced due to the soil pore space being filled with roots, reducing the size of the soil pore space and improving the soil's water holding capacity. It is clear that the roots influence the water holding capacity of the soil, even with the addition of WAP. Furthermore, the SWCCs of both WAP-amended soils were significantly improved compared to control soil. FA-WAP exhibited higher water retention than Com-WAP. This improvement was primarily attributed to the WAP filling of soil pore space. At low suction range, this enhancement was primarily attributed to capillary effects, while at high suction range; it was influenced by increased surface area.

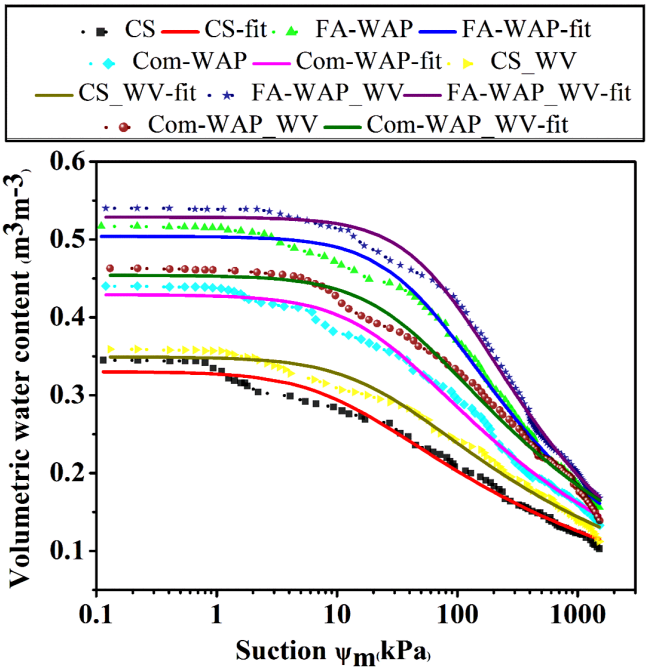


Figure 1: Influence of WAP and vegetation on SWCC in the soil

Compared to the control soil, the maximum PAWC increased twofold in the vegetation and the FA-WAP-amended soil. The WAP-amended soil improved the PAWC by 1.75 times. Previous literature also reported that vegetation improves the PAWC. As a result, the combined effect of WAP and vegetation would improve the PAWC, which helps plant growth during water stress conditions. This clearly states that the WAP and vegetation store water during irrigation and provide water when the plant is stressed, acting as a supplementary reservoir. The study clearly demonstrated the usefulness of WAP-amended soil that has its utility in bioengineered slope, green infrastructure/urban green planning, and arid region agricultural practices.

4 CONCLUSION

This study investigates the interaction between WAP, vegetation, and soil and how it affects water availability and survival time. With the presence of vegetation, the SWCC of both WAPs was determined. This study has produced the following conclusions:

- The FW was found to have higher water retention than CW due to the aluminosilicate contribution.
- The plant available water content (PAWC) of FW amended soil with vegetation was higher than that of CW and control soil.
- The rate of decrease in stomatal conductance (SC) and photosynthetic yield (PY) was found to be less in WAP amended soil, indicating less water stress on plants under water-deficit conditions.
- The addition of FW in the soil enhanced the yield more than 1.5 times under water stress

Bright Spark Lecture Ⅲ:

Brittle Failure Mechanism and Evaluation Model of Reef Limestone under Multi-Factor Regulation of Mineral-Pore-Interface Interactions

Dr. Yang Liu, Dong-sheng XU

, Askar ZHUSSUPBEKOV

Brittle Failure Mechanism and Evaluation Model of Reef Limestone under Multi-Factor Regulation of Mineral-Pore-Interface Interactions

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ABSTRACT

Reef limestone (RL), formed through multi-level coupling of biological skeleton and diagenetic cementation develops a discontinuous pore network. Its brittle characteristics are synergistically controlled by mineral crystal strength, fracture toughness of cementation interfaces, and spatial distribution of porosity. Traditional brittleness evaluation methods aredifficult to capture cross-scale brittle fracture mechanisms. On this basis, using drilling cores from an island in the South China Sea and focusing on three typical diagenetic minerals (aragonite, calcite, and dolomite), we innovatively propose a cross-scale evaluation method integrating "microscopic interfaces, meso-structures, and macroscopic responses." First, nanoindentation tests on different mineral components quantitatively characterize the fracture toughness of mineral crystals (K_{IC1}) and cementation interfaces (K_{IC2}), revealing that dolomite crystals possess significantly higher fracture toughness compared to calcite and aragonite. Second, fracture mechanics experiments calibrate the macroscopic fracture toughness (K_{IC}), establishing a cross-scale mapping relationship $\Delta K_{IC} = f(\Delta K_{IC}, \varphi)$, where ΔK_{IC} (K_{IC1}- K_{IC2}) quantifies the mechanical difference between crystals and interfaces, and φ represents porosity. Finally, a comprehensive brittleness index (BI) is defined, and uniaxial experiments establish a quantitative relationship between BI values and mineral composition/porosity. The proposed K_{IC} threshold-based evaluation method effectively reduces misjudgment rates in brittleness assessment, bridging the critical gap between the scale separation between microstructural characterization and macro-mechanical responses in rocks.

Keywords: Reef limestone; Nanoindentation; Fracture toughness; Brittleness evaluation; Multiscale coupling

1 INTRODUCTION

Reef limestone, serving as the primary load-bearing stratum in island-reef foundations, differs fundamentally from conventional terrigenous geomaterials. This unique lithology exhibits a loose and porous structure, high porosity, and pronounced heterogeneity (Xu et al., 2024a; Zhu et al., 2016). Owing to frequent geological dynamics and microbial activities in marine environments, RL develops distinct sedimentary features with stratified zoning phenomena at varying burial depths (Xu et al., 2024b; Zhao et al., 2015). The resultant heterogeneity in mineral composition and microstructure leads to complex macroscopic mechanical responses, which are collectively governed by the micromechanical behavior of constituent phases and diagenetic mineral heterogeneity (Zhao et al., 2013). Consequently, mineral composition, pore architecture, and cementation interface characteristics emerge as critical determinants of multiscale damage mechanisms in RL.

Leveraging advanced micro rock mechanical characterization techniques, this study employed TESCAN Integrated Mineral Analyzer (TIMA) and nanoindentation to systematically investigate the mineralogical composition, micro structural evolution, and micro rock mechanical properties of RL. Energy-based fracture analysis was applied to evaluate the failure characteristics of diagenetic mineral phases. By integrating these microscale findings with macro mechanical experimental data, we established multiscale correlations and developed a brittleness evaluation framework based on fracture toughness thresholds.

2 METHODOLOGY

2.1 Microscale rock mechanics experiment system

The microscale rock mechanics experiment system, as shown in Figure 1, integrates a fully automated

mineral analysis system (TIMA) with a nano-indentation device for microscale mechanical characterization. First, TIMA was used to rapidly and accurately determine quantitative information of RL, including mineral composition, mineral crystal interface, crystal size, structural configurations, and elemental occurrence states. Subsequently, indentation tests were conducted using a grid-based nanoindentation method, and the mechanical properties of the material were analyzed through load-indentation depth curves.

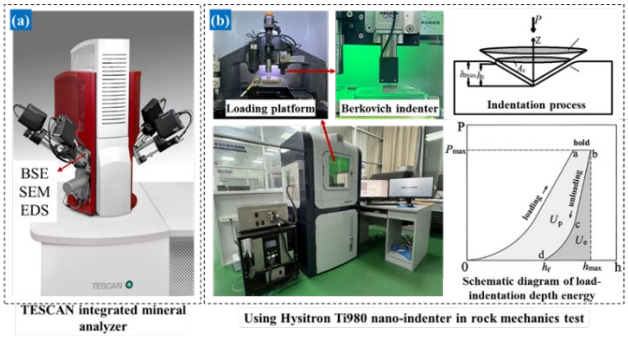


Fig. 1. Microscale rock mechanics experiment system

2.2 Data process

Fracture toughness is one of the critical parameters for assessing rock materials' resistance to crack propagation, is often evaluated through energy-based analytical methods due to the inherent challenges in directly measuring crack lengths. As shown in Figure 1(b), the total energy input (U_t) during testing consists of two components: pure plastic energy (U_p) and elastic recovery energy (U_e) during unloading. The fracture energy is denoted as U_c, and the plastic energy is U_{pp}. Therefore, U_c can be determined using equations (1)(2).

$$U_t = U_e + U_p = U_e + U_{pp} + U_c \tag{1}$$

$$\frac{U_{pp}}{U_t} = 1 - \left[\frac{1 - 3\left(\frac{h_f}{h_{max}}\right)^2 + 2\left(\frac{h_f}{h_{max}}\right)^3}{1 - \left(\frac{h_f}{h_{max}}\right)^2} \right] \tag{2}$$

The fracture toughness of the tested material is calculated according to the following equation.

$$K_{IC} = \sqrt{G_c E_r} \tag{3}$$

3 EXPERIMENTAL RESULTS

3.1 Brittle failure characteristics of reef limestone

Statistical analysis revealed significant positive correlations between fracture toughness (K_{IC}) and both Young's modulus (E) and hardness (H). The analysis of 100 indentation measurements demonstrated consistent clustering of data points within 95% confidence ellipses for both K_{IC} -E and K_{IC} -H relationships (Figure 2).

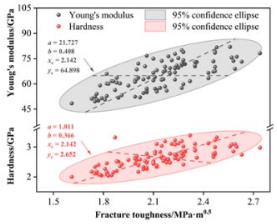


Fig. 2. The relationship between KIC and E/H

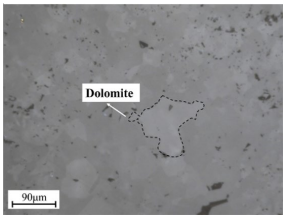


Fig. 3. Dolomite crystals and mineral interface

The 95 % confidence ranges of the E and H with K_{IC} are expressed by the following equations.

$$K_{IC} = (472.063 - 2835.824E^2)^{0.5} \tag{4}$$

$$K_{IC} = (1.022 - 7.632H^2)^{0.5} \tag{5}$$

3.2 Correlation between Mineral-Pore-Interface and mechanical properties

Through micro structural characterization of porosity and the Young's modulus of Mineral crystal interface (E_{int}), we employed a homogenization approach to establish quantitative correlations between these micro structural parameters and the macroscopic mechanical properties of RL (Figure 4).

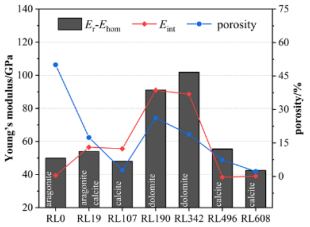


Fig. 4. The relationship between Mineral-Pore-Interface and E

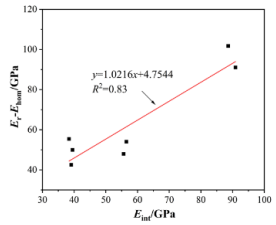


Fig. 5. The fitting curve of E_{int} and E_r

As shown in Fig. 5, The equivalent macroscopic Young's modulus (E_{hom}) can be estimated from the E of diagenetic minerals and mineral boundaries.

$$E_{hom} = E_r - 1.0216E_{int} - 4.7544 \tag{6}$$

4 CONCLUSIONS

(1) K_{IC} values: dolomite > calcite > aragonite. The relationships between E, H, and K_{IC} were better characterized through 95% confidence ellipses. This approach enables quantitative brittleness evaluation of RL while overcoming the experimental challenges associated with direct microscale K_{IC} measurement.

(2) Both porosity and crystal interface significantly influence the mechanical properties of RL. The strong correlation between the E_{int} and the macroscopic Young's modulus of RL provides compelling evidence supporting the validity of the Mineral-Pore-Interface coupling mechanism for investigating RL's mechanical behavior.

Poster session

Calculation of building sediments taking into account the creep of the base soils composed of dislocated Proterozoic (Cambrian) clays

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ABSTRACT

In the suburbs of St. Petersburg, there have been cases of large and uneven settlement of residential buildings on a natural basis. The total settlement from the nine-floor building (Pushkin) was 105 cm. The building received heavy uneven settlement for 14-15 months in cause of settlement is the deposition of soil from under the foundations throughout the building site. At the same time, the flogging began during the construction of the 8th floor, that is, at loads less than the design one. Due to the state of disrepair, the building was dismantled. A similar situation developed in Petrodvorets.

An analysis of the engineering and geological situation jointly with S.N. Sotnikov, allowed us to conclude that the causes of deformation are the loss of the bearing capacity of the base, composed of dislocated Proterozoic clays. Soil moisture decreases quite naturally with a depth from 0.28 to 0.20-0.19. Such a distribution of humidity was also noted in Petrodvorets. (S.N. Sotnikov, H.Z.Bakenov, L.G. Zavarzin).

Keywords: foundation settlement, dislocated Proterozoic (Cambrian) clays, entropy criterion for soil destruction, soil destruction, soil creep, finite element method

INRODUCTION

Investigation of rheological properties Cambrian clays (N.N. Maslov, A.Y..Budin, S.N. Sotnikov, H.Z.Bakenov and others), indicates that the eluvium of the indigenous clays has pronounced creep properties. Taking into account the above, the calculation of sediments on dislocated Proterozoic clays must be carried out taking into account the creep and destruction of the base soils with the preliminary allocation of a transitional layer both in depth and in strike.

1. CONSTRUCTION SITUATION OF THE RESEARCH OBJECT

Total settlement of the nine-story part of the building (Pushkin) was 105 cm. Settlement developed most intensively at the completion of the construction stage. After the suspension of construction, settlement levels decreased significantly, during the finishing works, the speeds increased again. The building received heavy uneven settlement. The reason for such settlement is the prolonged discharge of soil from under the foundations throughout the building site, while the discharge began during the construction of the 8th floor. Such situations are extremely rare in construction practice. Due to the state of disrepair, the building was dismantled. A similar situation developed in Petrodvorets, the house was saved [6].

In the weathering zone Proterozoic clays underwent changes, significantly decompressed and saturated with water, while retaining the layered texture characteristic of Proterozoic clays in the bedrock (S.N. Sotnikov, L.G. Zavarzin).

2. RESEARCH OF THE RHEOLOGICAL PROPERTIES OF SOIL FOR PURE SHEAR

Here are shown the results of the study of the Cambrian clay of undisturbed structure were determined by: τ_o and τ_∞ at vertical pressure σ_i= 0.1, 0.2 and 0.3 MPa; strength parameters for peak and residual strength: (c_o, φ_o) and: $(\ _\infty, \varphi_\infty)$ c_o=39 kPa; φ_o = 5-6°; c_∞, = 34 kPa; φ_∞ = 4°; τ_o=59.6 kPa; τ_∞ = 49 kPa at σ_i= 0.2 MPa. For Cambrian clays at I_l = 0.3 − 0.32 and Natural humidity W= 0.276- 0.280; creep thresholdτ_τ= 0.32τ_o= 19 kPa; viscosity coefficient η_i= 1.53-1.58 10^{13}Pa s, which corresponds to experimental data.

3. METHOD OF CALCULATING THE SEDIMENT TAKING INTO ACCOUNT THE GROUND AND SOIL DESTRUCTION

For the analysis of the stress-strain state by the finite element method, taking into account the rheological properties of soils, we adopted the Bingham-Shvedov model [1,3]. The reaction of the medium to volumetric deformations is elastic and the shear is elastic-viscoplastic, obeying the law of plastic flow, with the criterion of Coulomb

strength, the viscous shear flow is assumed to be equal-volume. The rate of creep shear strain is proportional to the difference between the acting one and the maximum voltages. In order to ensure such a deformation while its stiffness matrix of the system remains unchanged, additional tangential stress must be applied to it: $\Delta\tau = \frac{1}{T_p}\Delta\tau_i\tau^{п}$. Here T_p is the relaxation time. The direction $\Delta\tau$ may coincide with the direction of the maximum tangential stress in the element. The main additional stresses are converted into axial stresses, $\{\Delta\sigma\}$ then the additional nodal forces are calculated using the vector $\{\Delta F\}$. The following solution of the problem gives an answer about the state of the region at time $t_{i-1} + \Delta t_i$ etc. The procedure for obtaining an elastic-viscoplastic solution is similar to obtaining an elastic-plastic solution.

$$\{\Delta\sigma_{1,3}\} = \{\xi\tau^{п}, -\xi\,\tau^{п}, \alpha\}^{т}, \text{ где } \xi \leq \frac{G\Delta t_i}{\eta}. \quad (1)$$

We have adopted an increasing cycle by cycle time step. The considered EVP solution procedure was implemented by us in the Geomechanics GREEP program.

4. CRITERIA OF SOIL DESTRUCTION

Shear tests of soils [1,2,5,7], as well as our studies showed that when describing creep and fracture, the most common condition for fracture is the entropy criterion. According to this, destruction occurs when the entropy density reaches the critical level $\Delta S^* = const$ [1,2,5]. In the case of mechanical dissipation and under isothermal conditions ($\Theta = 0$) and a homogeneous temperature field, for the rheological model implemented in the program: $\tau_i = \tau_T + \eta\dot{\gamma}_i$ when $\tau_i > \tau_T$ the entropy criterion is written as:

$$\Delta S^* == [(\tau_i - \tau_T)^2 t^*] / \theta\eta_i, [\text{Pa} \cdot \text{grad}^{-1}]. \quad (2)$$

Here θ is the absolute temperature, τ_i maximum shear stress, τ_T the ultimate shear stress.

The time to destruction $t^* = \frac{\Delta S^* \theta\eta}{(\tau_i - \tau_T)^2}$ (3). The change in the entropy density $\Delta S_i$is calculated for the predicted time and the coefficient of accumulation of damage to the "characteristic" volume of soil is determined $\rho = \Delta S_i / \Delta S^*$. При $0 < \rho > 1$, the state is stable; $\rho \to 1$ and $t_i \to t^*$.

5. PREDICTION OF FOUNDATION SEDIMENTATION

The forecast is completed for settlement of the research object. The pressure at the base of the foundation, σ =0.2 MPa, the soil characteristics are given in [6]. The coefficient of viscosity and long-term strength for semi-solid clay adopted according to our research and sources [1,4,8] for the same soil. η=6,48 10^{10} PA·s, long-term strength test results σ_∞ = (0,6-0,7) σ_o, Here σ_o and $\sigma_{ст}$ -conditionally instantaneous and standard strength. As calculations have shown, the extrusion of soils from under the foundations with the formation of prisms occurred at $\sigma_{ср} < P_{пр}$. According to the creep curves, the time to destruction, at $\tau_3 = 0.8\ \tau_0$ is t_3^*=36 36 days; viscosity coefficient η_3= 2.82· 10^{12}Pa*s; critical entropy density $\Delta S_i^* = 3.01\ \text{Pa} \cdot grad^{-1}$ The predicted sample destruction time at $\tau_1 = 0.675\tau_0$ according to formula (3) will be: $t_1^* = 3.02 \cdot 10^7$ sec= 386 days. At the end of the experiment t_1= 80 days, the increase in entropy density is ΔS_i = 07÷ 0.8 $P\text{a} \cdot grad^{-1}$, the coefficient of accumulation of damage ρ =0.23-0.27- stable condition.

CONCLUSIONS

The forecast of settlement, taking into account the creep of the soils of the finite element method, increases the reliability of calculations, and the use of the entropy criterion allows, according to short-term tests, to assess the bearing capacity of structures for a long period.

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Application of airborne laser scanning for monitoring deformations of the Nadym–Salekhard highway under permafrost conditions

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ABSTRACT

The article presents the results of comprehensive aerial phototopographic surveys using airborne laser scanning (ALS) from a VT-30E unmanned aerial vehicle (UAV) to assess the intensity of deformations of the Nadym–Salekhard highway, located in permafrost conditions. The study included the creation of a geodetic monitoring system using GNSS equipment at 15 sites and ALS at 10 sites, with a total length of 143 km. The VT-30E helicopter-type UAV with an AGM-MS3 v.2 airborne laser scanner provided a digital elevation model resolution of 2–4 cm². The data obtained made it possible to identify the main factors of deformation in the roadbed and pavement, as well as to create a geodatabase for long-term monitoring of the cryogenic dynamics of the transport infrastructure in the Yamalo-Nenets Autonomous Okrug.

Keywords: airborne laser scanning, permafrost, highway, unmanned aerial vehicle, deformation, cryogenic dynamics

1. INTRODUCTION

The considerable length and continuity of transport infrastructure in the Russian Far North make them especially vulnerable to various exogenous processes, often activated by changes in local hydrological regimes, geocryological conditions, and climate change. The Nadym–Salekhard highway, connecting the western and eastern parts of the Yamalo-Nenets Autonomous Okrug and providing the capital with access to the main road network of Russia, is a key object in the region. The road is 344 km long and includes 53 bridges and 46 reindeer crossings.

Degradation of permafrost soils due to thawing of the road foundation, climate change, and other factors leads to intensified deformations, negatively affecting the stability and quality of the pavement (Syromyatnikov et al, 2024). Traditional monitoring methods do not always provide the required accuracy and timeliness for assessing the condition of extended transport facilities. Therefore, the use of modern remote sensing technologies, including airborne laser scanning, is especially relevant (Isaev et al, 2021).

2. METHODS

2.1 Equipment and Software

The study used a VT-30E helicopter-type UAV with a maximum takeoff weight of up to 30 kg and a payload of 8 kg. The UAV is equipped with a 10 kW electric motor and provides a maximum flight duration of 1.5 hours at a range of up to 40 km. The airborne laser scanning system AGM-MS3 v.2 included:

- A scanning unit with a scan rate of 680,000 pulses/sec and a range of 0.5–200 m;
- An AGM-PS.M inertial navigation system with a high-precision GNSS receiver;
- A Sony Alpha ILCE-6000 camera synchronized with the navigation system.

Geodetic referencing was performed using Pentax G6Ti and EFT M1 Plus2 GNSS equipment, with post-processing in the AUSPOS service. ALS trajectory adjustment was performed in AGM ScanWorks Pro. Phototopographic data were processed in LiDAR 360 and Agisoft Metashape.

2.2 Geodetic Referencing

A geodetic monitoring system was established at 15 sites along the highway. Benchmarks were installed every five kilometers on stable sections without visible deformations, using anchor bolts in the asphalt and on road elements. Static GNSS observations were also conducted at state geodetic network points and at intersections with gas pipelines.

Coordinates were determined by the PPP (Precise Point Positioning) method using AUSPOS, with observation times exceeding six hours. This ensured a mean planimetric error of 7–8 mm and an elevation error of 15–20 mm.

2.3 Airborne Laser Scanning Parameters

Aerial phototopographic surveys were carried out at a relative altitude of 100 m and a speed of 12–15 m/s. The distance between flight lines was 70 m, allowing coverage of a 400 m wide corridor along the highway. The average density of unclassified point clouds was 60–70 points/m², and for the "ground" class—5–7 points/m².

3. RESULTS AND DISCUSSION

3.1 Work Characteristics

During the 2024 field season, ALS was performed on 10 sections of the Nadym–Salekhard highway, totaling 143 km (Erofeyev et al, 2024). The data provided a high spatial resolution digital elevation model (2–4 cm), enabling analysis of road irregularities and deformations down to several tens of centimeters. The average coordinate error for photo centers was: X – 1.37 mm, Y – 0.17 mm, Z – 0.62 mm.

3.2 Geomorphological Analysis

The geomorphological analysis identified various exogenous processes affecting the highway:

- Ngarka-Veloyakha River valley section: Development of sheet and gully erosion, ground subsidence, areas of excess moisture accumulation, and existing subsidence were identified.
- Polygonal-block terrain section: A system of residual blocks and separating depressions, which become temporary watercourse valleys, was observed. Fine material removal along depressions was noted.
- River crossing sections: Dominated by channel and floodplain processes, sometimes combined with cryogenic ones. Free meandering and oxbow lake formation were noted.

3.3 Deformation Detection

ALS enabled detailed mapping of roadbed deformations. At the 1014 km + 82 m section, horizontal stretching of the pavement was detected, with uplift in the central part and depressions at the edges. Analysis of 10 cm interval contour lines clearly showed deformation vectors.

3.4 Monitoring Database

A GIS and database (Shkurenko et al, 2024) was created, including:

- Point clouds in LAS format for each flight;
- Georeferenced aerial photographs;
- Vector geographic files in Shape format;
- Surface deformation analysis results;
- Morphometric maps and difference relief maps.

4. CONCLUSIONS

The study demonstrated the high efficiency of UAV-based airborne laser scanning for monitoring highway deformations under exogenous geodynamic processes in permafrost regions.

ACKNOWLEDGEMENTS

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GIS-based landslide risk assessment in Karo Regency, North Sumatera, Indonesia

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ABSTRACT

In Indonesia, the convergence of the Eurasian Plate, the Indo-Australian Plate, and the Pacific Plate generates significant volcanic and tectonic activity. This geotectonic configuration substantially increases the nation's vulnerability to geological hazards, including earthquakes, tsunamis, and landslides. Karo Regency, situated in a hilly region with high annual rainfall, is particularly susceptible to landslides. This study adopts a Geographic Information System (GIS)-based approach that integrates hazard, vulnerability, and capacity components to evaluate landslide risk in the region. Spatial analysis was conducted using QGIS version 3.30.2, applying a scoring and overlay method in accordance with the Technical Guidelines for Landslide Disaster Mitigation (Minister of Public Works Regulation No. 22/PRT/M/2007) and the General Guidelines for Disaster Risk Assessment (BNPB Head Regulation No. 2 of 2012). The resulting risk classification identifies five levels of landslide risk: very low (636.320 km²), low (506.095 km²), moderate (901.106 km²), high (121.461 km²), and very high (13.284 km²). Model accuracy was evaluated using the Receiver Operating Characteristic (ROC) curve, which produced an Area Under the Curve (AUC) value of 88.6%, indicating a high level of predictive performance.
Keywords: GIS, landslide, hazard, vulnerability, capacity

1 INTRODUCTION

Landslides are geological hazards caused by the destabilization of slopes, frequently leading to substantial damage to infrastructure and loss of life. Landslides are a worldwide concern, especially in emerging nations with tropical climates like Indonesia. In these places, landslides regularly occur, particularly in areas with steep topography and elevated rainfall intensity. According to data from Indonesia's National Disaster Management Authority (BNPB), there were 591 documented landslide incidents in 2023, leading to 132 fatalities, 17 individuals reported missing, 767 injuries, and damage to 2,054 residential units and 128 public structures.

Indonesia's location at the convergence of several tectonic plates renders it highly prone to geological hazards, including earthquakes, tsunamis, volcanic eruptions, and landslides. The tectonic and volcanic activities along the Sunda Arc, part of the Pacific "Ring of Fire," create complex geological settings that enhance landslide susceptibility. North Sumatra's Karo Regency, in particular, represents a critical area for landslide risk assessment. The region experiences high rainfall and is dominated by highland terrain, including two active volcanoes, Mount Sinabung and Mount Sibayak. Moreover, Karo is traversed by major segments of the Sumatra Fault Zone, namely the Tripa and Renun fault segments, increasing its vulnerability to earthquake-induced landslides.

A comprehensive understanding of landslide risk is essential for effective disaster mitigation planning. Geographic Information System (GIS) technology offers powerful tools for landslide risk analysis by integrating various geospatial factors influencing susceptibility and vulnerability. Previous studies have demonstrated the effectiveness of GIS in landslide hazard and risk mapping.

In this study, landslide risk assessment in Karo Regency will be conducted by developing a GIS-based landslide risk map. The approach integrates various factors contributing to landslide susceptibility and vulnerability, employing a weighted scoring and overlay technique. The resulting landslide risk map will be validated using the Receiver Operating Characteristic (ROC) curve method to ensure the model's accuracy and reliability. Ultimately, the findings are expected to contribute significantly to disaster risk reduction efforts in Karo Regency and offer a reference model for other regions with similar geological and climatic characteristics.

2 RESEARCH LOCATIONS

The study area is Karo Regency, North Sumatra Province, encompassing 17 sub-districts, 10 urban villages, and 259 rural villages over 2,127.25 km². Geographically, it lies between 2°50'–3°19' N latitude and 97°55'–98°38' E longitude.

3 RESEARCH METHODS

3.1 Data Processing

This study utilized secondary data from verified sources. Data processing was performed using scoring and overlay techniques on hazard, vulnerability, and capacity maps, based on the Regulation of the Minister of Public Works No. 22/PRT/M/2007 and the BNPB Head Regulation No. 2/2012. GIS processing was carried out in QGIS 3.30.2, and the model outputs were validated using ROC curve analysis to ensure reliability.

3.2 Data Analysis

Hazard mapping involved the analysis of seven parameters: slope steepness, soil conditions, lithology, rainfall intensity, land use, slope hydrology, and seismicity. Vulnerability mapping used two main parameters: population density and housing distribution. Capacity data were sourced from Inarisk BNPB North Sumatra, with scoring and weighting based on official guidelines.

$$Risiko = Hazard \times \frac{Vulnerability}{Capacity} \quad (1)$$

4 RESULT

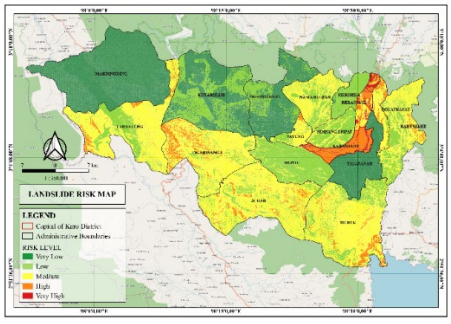


Fig. 1. Risk map of Karo Regency

The analysis reveals that 42.36% of Karo Regency is classified within the moderate risk (Fig. 1), followed by very low risk (27.51%), low risk (23.79%), high risk (5.71%), and very high risk (0.63%). Slopes, particularly those exceeding 45°, are a major contributing factor to increased landslide risk. The prevalence of ultisol soils, which are characterized by low bearing capacity and high erodibility, exacerbates the region's vulnerability. Additionally, the presence of fragmented volcanic rock formations further destabilizes slopes. The region's high annual rainfall, ranging between 1501-2000 mm, coupled with notable seismic activity, significantly intensifies the risk by increasing the load on the slope and decreasing soil cohesion. Furthermore, land-use changes, such as deforestation and the conversion of land for agriculture, further compromise slope stability, thus contributing to the heightened susceptibility to landslides.

5 VALIDATION

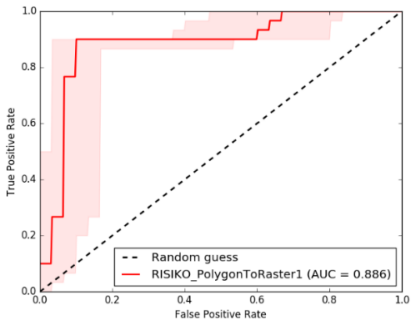


Fig. 2. Validation result using ROC method

The ROC curve analysis showed an AUC value of 0.886 (88.6%), indicating good model performance and high mapping accuracy (Fig. 2).

6 CONCLUSIONS

The results indicate that Karo Regency is highly vulnerable to landslide hazards, driven by a complex interplay of geological, geomorphological, climatic, and anthropogenic factors. The region's slopes, combined with unstable soils and high rainfall, exacerbate the susceptibility to landslides. Additionally, anthropogenic activities such as deforestation and land conversion further compromise slope stability. To address these challenges, effective landslide disaster mitigation must adopt an integrated approach that encompasses prevention, risk reduction, and preparedness strategies.

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Computational justification of the use of protective measures in the construction of excavation pits in dense urban areas on weak soils

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ABSTRACT

In such megacities as St. Petersburg, the development of urban areas is not possible without the construction of existing historical buildings. For the construction of foundations and underground parts of buildings, deep excavation pits have to be designed. In such circumstances, the main task of the successful implementation of the project is to ensure the safety of neighboring buildings. For St. Petersburg, for example, the search for a solution to the conditions of safe construction is associated with an assessment of the deformability of the foundation soils under various kinds of technological construction impacts. We have analyzed various methods of geotechnical protection of buildings and structures based on practical experience, depending on a combination of factors. Numerical calculations performed using the PLAXIS software package make it possible to predict with sufficient accuracy the degree of impact of excavation of the excavation pit and its loosening on nearby buildings, taking into account the proposed protective measures. The article discusses practical examples of the use of various active and passive geotechnical protective measures in the groundwater conditions of St. Petersburg, as well as possible consequences in the event of their absence. For the construction of a geotechnical barrier, it is necessary to use modern materials that reduce the time of standing and accelerate the strength gain of the fixed soil mass.

Keyword: weak soil, deformation of building, excavation pit, dense urban area, protective geotechnical measures

1 INTRODUCTION

When constructing in historical areas, the most important condition for preserving the architectural expressiveness of existing buildings and monuments, as well as the tourist attractiveness of St. Petersburg, is the scientific and technical justification and support of geotechnical work in terms of predicting possible uneven deformations of buildings that are affected by such construction, designing protective measures, and conducting continuous geotechnical monitoring at all stages of construction.

The development of underground space in residential buildings and courtyards in the early 19th and late 20th centuries was carried out by creating basements for storage and technical purposes, leading to the emergence of so-called "suspended courtyards" that were used for storing coal and firewood for heating the houses. These courtyards were underground spaces of varying heights (2.0 to 2.4 meters) located beneath the courtyard surface.

2 ANALYSIS OF ENGINEERING AND GEOLOGICAL CONDITIONS

The geological structure of the areas under consideration, which include underground parking lots, is characterized by layers typical of the central part of the city. According to drilling and static sounding data, up to a depth of 45.00 m, modern man-made formations (tIV), represented by bulk soils (IGE-1); biogenic sediments (bIV), represented by inhibited soils (IGE-2); lacustrine-marine sediments (lmIV), represented by dusty, dense sands with sandy loam layers, are involved. (IGE-3), sandy loams, with interlayers of fluidplastic loams (IGE-4); upper quaternary deposits (QIII) of lacustrine-glacial (lgIII) genesis, represented by flowing loams, with interlayers of fluidplastic, ribbon and layered (IGE-5), sandy loams, plastic (IGE-6); upper quaternary deposits (QIII) of glacial (gIII) genesis, represented by sandy loams, plastic (IGE-7, 8), semi-solid loams with layers of refractory (IGE-9); middle Quaternary deposits (QII) of lacustrine-glacial (lgII) genesis, represented by plastic loams (IGE-10), and Vendian deposits of the Kotlinsky horizon (V2kt), represented by hard clays (IGE-11).

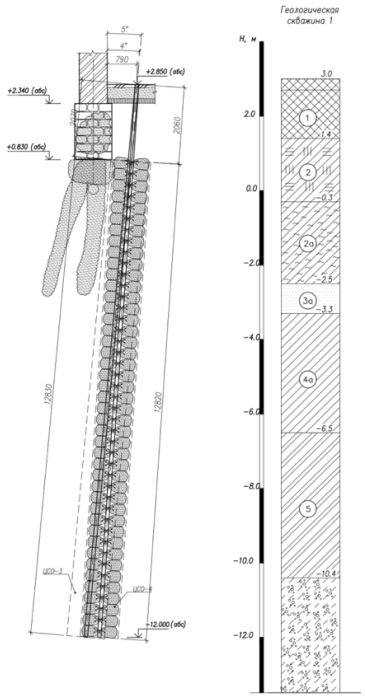
3 FEATURES OF THE FOUNDATIONS OF OLD BUILDINGS IN THE CENTRAL PART OF ST. PETERSBURG

The depth of the surveyed foundations ranged from 1.5 m to 5.0 m, depending on the soil conditions, the location of the building, the presence of a basement, and its functional purpose. It is worth noting a number of patterns in the design of foundations that were typical for buildings constructed between the 18th and early 20th centuries. These patterns were identified based on the results of a technical survey of 64 buildings from this period in the central part of Saint Petersburg. For foundations on natural ground for stone buildings with 2-6 storeys, the bearing layer was sands of different sizes (56%) and sandy-clay soils (44%). The foundations of 82% of the buildings were made of rubble, granite, and limestone stones, and 18% were made of burnt bricks. Wooden logs were found under the foundations of 22 buildings (34%), and wooden piles were found under 9 buildings (14%). The width of the base of the inspected buildings varies from 0.5 to 2.8 m for foundations on a sandy base and from 0.8 to 2.3 m for sandy-clay soils, depending on the number of storeys in the building. The most common foundation depth is between 1.5 and 2.0 m (44% of foundations on sandy bases; 53% on silt-clay soils).

Comparison of the actual average pressure p acting on the footings of the inspected buildings with the permissible pressure values on the foundation soil regulated by modern standards, in particular the value of R, revealed the following features for the foundations of buildings built in the central part of St. Petersburg: in 62% of cases, when the foundation soil consists of sandy soils, the pressure on the foundation footings exceeds the permissible normative pressure R, and when the foundation soil consists of sandy clay soils, the pressure on the foundation footings exceeds the permissible normative pressure R by 34%.

4 SAFETY OF GEOTECHNICAL CONSTRUCTION IN DENSE URBAN AREAS)

Reinforcement of the foundation base using the cuff technology by drilling a 112 mm ø well to the design mark (fig. 5), filling it with a casing solution, and allowing it to stand for 3 days before the main injection stage begins. the injection is performed with an injection solution based on a cement binder with an additive of $CaCl_2$ and a water-cement ratio of 0.8, from bottom to top, with a height difference of 80 mm. the initial injection pressure is set at 1.5 MPa to break the clamping ring, and then the pressure is reduced to the working pressure of 0.3 MPa. the injection is stopped when the failure pressure reaches 0.6 MPa, and the solution flow rate should be adjusted to 5 l/min.



5 CONCLUSIONS

Regular instrumental monitoring and operational management of the construction process, known as geotechnical monitoring, allow for the identification of adverse trends during the construction phase and the first period of operation.

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Modeling of stress-strain state of water-saturated weak soil when injected along the lateral surface of the pile to increase bearing capacity of the bored pile

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ABSTRACT

During the construction and reconstruction of buildings and structures in conditions of weak soils and dense urban development of the city of St. Petersburg and the installation of piles, despite the use of the most effective, from the point of view of safety and their bearing capacity, drilling and injection piles, it happens that in order to achieve the required bearing capacity it is necessary to significantly increase the length or diameter of these piles. This is due to the insufficient efficiency of this type of piles on the side surface in conditions of weak soils. In this connection, work has begun on the development of a new method for installing a bored pile using injection of fixing solutions along the side surface to qualitatively increase the bearing capacity by fixing and compacting the soil mass of the near-pile space. This article is devoted to numerical and analytical calculations of a new type of bored pile using injection of cement mortar along the lateral surface, as well as to the study of the effect of injection on the stress-strain state of weak soil in the near-pile space. In particular, the following aspects were investigated: the dependence of the crack opening width on the applied injection pressure in the range from 150 to 500 kPa; a change in the porosity coefficient of the soil mass around the pile during its hardening due to compaction; a change in effective stresses in the soil due to the injection of soil around the pile

Keywords: drilling pile, injection fastening using collar cementing columns, weak soils, reinforcement, soil stabilization

1 INTRODUCTION

During the reconstruction and construction of buildings and structures in the central part of St. Petersburg, difficulties most often arise precisely because of the large thickness of weak soils. In this regard, the most common methods of foundation construction and reinforcement using bored piles become ineffective due to the need to increase the diameter or length of such a pile to achieve the required bearing capacity. To increase the efficiency and cost-effectiveness of the drilling pile device, it is proposed to use injection along the side surface. In order to study the effect of injection on the stress-strain state of the soil around the pile, a number of calculations were performed in the Plaxis 2D software package.

2 RESEARCH METHODOLOGY

The hardening (sealing) of water-saturated loam during injection using cuff technology (hydraulic fracturing) was considered.
Characteristics of the compacted soil:
• soil density – 1.84 g/cm^3
• density of dry soil – 1.33 g/cm^3
• filtration coefficient – 0.05 m/day
• modulus of deformation – 2639 kN/m^2
Note: the value of the deformation modulus is determined by the compression curve (Fig. 1) for a depth of 3.5 – 4 m. At a depth of 3.5 m, the stress from the ground's own weight is 0.064 MPa, at a depth of 4 m – 0.074 MPa. Then for the range 0.05 – 0.1 MPa

The groundwater level coincides with the

ground surface. The type of soil behavior is undrained. At a depth of 4.5 m, it was assumed that there was a strong, almost incompressible soil that extends to a great depth. The distance between injection wells is 1.2 m. Thus, an area with a radius of 0.6 m is considered, along the axis of which the injector is located. Injection is performed from the bottom up. The first gap (horizontal) is at a depth of 4 m, the second one is at a depth of 3.5 m, etc. The gap is filled with a mortar under pressure, as a result of which the cavity expands, while the soil is compacted. It was assumed that the pressure of the mortar, and, consequently, the crack opening, decreases with distance from the injector. The solidification time of the mortar is approximately 2 hours.

Two sealing schemes were considered. In scheme №. 1 (fig.1), it was assumed that the injection of the mortar into each gap occurs for a long time (2 hours).

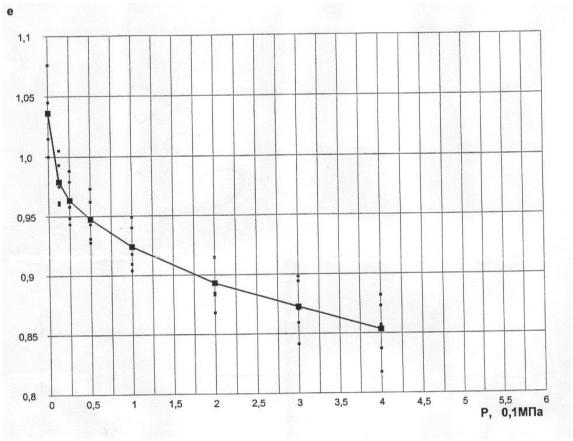


Fig. 1. Compression curve.

In scheme № 2, the instantaneous filling of the first gap with a mortar, then the second one, was assumed. After that, the supply of the mortar stops, excessive pore pressures arise in the soil, which gradually dissipate.

An axisymmetric problem using 15 nodal triangular elements was considered. The Mohr-Coulomb model was used as a model of compacted soil.

A series of calculations were performed for the values of the solution pressure 150, 200, 250, 300, 400, 500 kPa.

To quantify the increase in the bearing capacity of a bored pile, a numerical experiment was conducted to study the effect of increasing the bearing capacity of a bored pile with a diameter of 300 mm and a length of 2.1 m when it was injected with a fixing mortar along the lateral surface. As a fixed soil, the characteristics of heavy, dusty, flowing, ribbon, thixotropic loam were used according to the test results of report of LenTISIZ CJSC with a deformation modulus E = 1.8 MPa, specific adhesion c = 8 kPa, internal friction angle φ = 7º and filtration coefficient kf = 0.05 m/day.

3 CALCULATION RESULS

At a depth of 3.5 – 4 m, natural total average stresses are 70-75 kPa, effective average stresses are 30-35 kPa, pore pressures are 35-40 kPa.

When the mortar is injected into the cavity at a depth of 4.0 m, the maximum instantaneous displacement when the crack opens is from 4.2 to 28.0 mm, depending on the pressure.

With the instantaneous opening of the crack at a depth of 4 m, the total average stresses increase significantly. Already at a minimum mortar pressure (150 kPa), the outlines of the so-called sealing zone are observed, in which stresses increase by more than 30% (more than 100-105 kPa).

The bearing capacity of the bored pile without reinforcement was 268.6 kN. After strengthening, its bearing capacity was 824.0 kN. Thus, the increase in load-bearing capacity was 3.07 times.

CONCLUSIONS

When injecting near-pile soil, compaction zones occur, which contributes to an increase in pressure on the perimeter of the pile, thereby increasing the contribution of friction forces to the overall bearing capacity;

With a higher injection pressure, the sealing effect increases, which can be observed based on the values of additional effective stresses in the soil mass between cracks after injection;

As shown by preliminary calculations for injecting soil around a short pile by injection in 30 cm increments along the trunk, the increase in total load-bearing capacity can reach up to 3 times.

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Information modelling in geotechnics

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ABSTRACT

Currently, in the Russian Federation, when designing the foundations of buildings and structures, a predominantly deterministic approach is used. The latter assumes both the constancy of the soil properties within the geological element, and the immutability of the external load on the foundations.
In fact, there is considerable uncertainty both in the properties of soils due to the natural variability of natural conditions. This uncertainty creates a problem when developing a reliable geological model and, consequently, complicates the design process. The article considers an integrated design approach that combines a quantitative assessment of the uncertainty of input parameters with traditional design methods to ensure the selection of the optimal model based on both safety and cost. This approach is implemented within the framework of the developed information modeling technology (BIMGeo). A key feature of BIMGeo is the integration of engineering surveys and foundation design into a single technological process. Digital models contain different data and can interact with other objects, which makes it easy, fast and efficient to make the necessary updates.

Keywords: information modeling, engineering surveys, slope design, uncertainty soils

1 INTRODUCTION

Despite significant interest in the use of information technologies in the construction industry, their implementation in conducting engineering and geological surveys is still insignificant both in the Russian Federation and in foreign practice (Boldyrev et al. 2023, Boldyrev 2023, Brennan, 2021). At the same time, in recent years, several standards have been put into effect in the Russian Federation regulating the general rules for the development and use of information technologies in construction.

2 METHODOLOGY

The proposed integrated information technology combines engineering surveys and base design into a single production process. All information, starting with geodetic surveys, conducting research, determining soil properties and calculating foundations using analytical and numerical methods, is carried out in a single digital stream with open data formats. This approach eliminates the stage of transferring information from geologists to designers, which reduces decision-making time.

3 PROBABILISTIC APPROACH

Currently, in the Russian Federation, when designing the foundations of buildings and structures, a predominantly deterministic approach is used. The latter assumes both the constancy of the soil properties in the area under study and the immutability of the external load on the foundations of buildings and structures.

In fact, there is considerable uncertainty about the properties of soils due to the natural variability of natural conditions, as well as knowledge-based sources of uncertainty, including limited availability of information about soil conditions and errors made during the stage of engineering and geological surveys. This uncertainty creates a problem when developing a reliable geological model of a survey site and, consequently, complicates the design process.

The simplest and most understandable method of probabilistic modeling is the Monte Carlo method. As a result of the simulation, we get results with different safety coefficients, unlike the traditional approach, when there is one safety coefficient. The solution of the problem in this formulation can be performed using one of the probabilistic finite element methods or analytical solutions.

4 PRACTIAL EXAMPLE

Let's consider the procedure of engineering and geological research and slope design with an assessment of the probability of failure. The procedure includes the following steps.

1. On the situational plan, the soil mass under study is highlighted with its boundaries in the plan. The method of cone penetration test (CPT) the soil parameters on the supporting workings necessary for calculating the slope stability. In this case, these are nine CPT workings

2. A statistical assessment of the sounding parameters and selected soil parameters is performed using samples from the support workings. The spatial variability of soil parameters is estimated: the average value of μ, the standard deviation σ, and the coefficient of variation COV.

3. The construction of a three-dimensional digital engineering and geological model (3D CGM) begins. To do this, the selected soil array is divided into finite elements of a given size (Fig. 1). Then, the soil parameters are determined in the space between the supporting workings. In this case, the Kriging method and the spherical variogram are used (Webster 2001).

4. A statistical assessment of soil parameters is performed using the values on the support workings and found in the space between the support workings. The spatial variability of soil properties is estimated: the average value of μ, the standard deviation σ, and the coefficient of variation COV.

5. The final construction of the 3D DEGM is being performed. The properties of soils are determined in each finite element of the soil mass under study.

6. Since the slope stability assessment is performed in a 2D formulation, one of the sections in the 3D DEGM is selected (Fig. 2) and the soil parameters in each of the finite elements of the section.

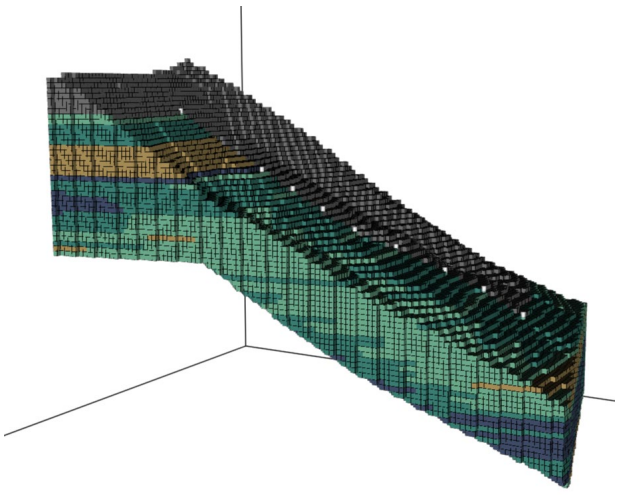


Fig. 1. Three-dimensional model of the slope. The grid elements are 1x1x1 m in size.

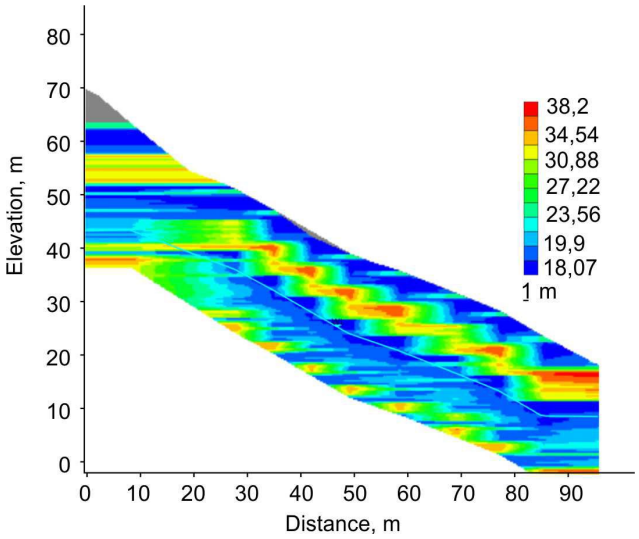


Fig. 2. An example of the distribution of the angle of internal friction along a section through workings on a three-dimensional model.

7. The slope stability coefficient is calculated using one of the limit equilibrium methods, for example, the Bishop method (Bishop 1955).

8. Using Monte Carlo simulation, the probability density function of the safety factor distribution and the probability of failure are found.

9. 3D CGM visualization is performed and the model is formed in the IFC format. The geometry of the 3D model with the display of finite elements and a list in an Excel spreadsheet with numbers, nodes of finite elements and soil parameters are displayed.

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Investigation of laboratory and field tests of piles installed by displacement technology

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ABSTRACT

The Drilled Displacement System (DDS) and Continuous Flight Auger (CFA) piles provide advantages over conventional pile systems, including better load carrying capability, faster installation times, and less spoil generation. This article describes tests conducted in a test tank filled with dirt on small piles constructed using the Continuous Flight Auger (CFA) and Drilled Displacement System (DDS) technologies. Model piles scaled at 1/20 and measuring 20 mm in diameter and 300 mm in length were used for the investigation. Static loading is applied to the model piles, and the relevant displacements are recorded at each stage of the loading procedure. The results of the investigation on load-settlement curves and estimations of ultimate bearing capacity were compared for both DDS and CFA piles. The study found that in terms of load-carrying ability, the DDS piles performed better than the CFA piles. In addition, DDS-drilled piles of 400 mm in diameter and 6 m in length underwent full-scale field testing with static stresses. The load-settlement response in the field test shows good agreement with the model tests. All things considered, the study's conclusions provide valuable knowledge regarding the behavior and effectiveness of DDS piles that may be applied to enhance the piles' installation and design in a variety of soil types.

Keywords: scaled model tests, model piles, bearing capacity, static load test

1. INTRODUCTION

The most common kind of foundation utilized at building projects in Kazakhstan is the pile foundation. Because they safeguard the pile's load-bearing capacity—which is drilled and filled due to the growing weight from high-rise buildings and structures—pile foundations are quite popular. New technologies and devices that have emerged in connection with the installation of drilled-filled pile foundations force regulatory document designers to improve them because there are currently no regulatory documents or recommendations for the installation of pile foundations using modern technologies [Zhussupbekov and ., Omarov, 2020, Chang and Utepov, 2020].

Drilled Displacement System (DDS) piles and Continuous Flight Auger (CFA) piles have been used in pile foundation construction recently. DDS piles are a type of displacement pile that is commonly used in urban environments where noise and vibration levels must be kept to a minimum. A steel casing is forced into the ground and a hole is created, all with the help of a hydraulic press. The area is then filled in with concrete and reinforcing steel to provide a solid foundation for the structure to be built. DDS piles are frequently used for small to medium-sized constructions and may be built in a range of soil types. On the other hand, to build a CFA pile—a type of auger cast pile—a continuous flight auger is drilled into the ground to the required depth. After that, the auger is removed, and the empty area it leaves behind is filled up by injecting concrete via its hollow stem. This process creates a continuous concrete column that acts as a solid foundation for the building of the

structure that follows. CFA piles are often used in large-scale building projects with challenging soil conditions, such as soft soil and rock.

With DDS technology, earthworks take less time, effort, and money while piles can support more weight at the same time. Drill-filled pile installation using DDS technology has become more and more common in the US and Europe during the last ten years. This method's main advantages are its high economic efficiency, its ability to reduce noise and vibration during pile installation, and its increased pile bearing capacity. [Zhussupbekov and Omarov, 2021, ., Temirova and Riskulov, 2021]. This technology has a disadvantage in that it may have an impact on the foundations of already-existing buildings and structures, notwithstanding the advantages mentioned above. It is not recommended to use this method in areas with dense urban construction because of this.

The procedures that comprise the sequence of work for installing drilling piles made using DDS technology are as follows: installation of drilling equipment at the drilling site; immersion of the drilling tool with a sealing system to the design mark; connection of the concrete pump followed by filling the well with concrete mixture and simultaneous extraction of the drilling tool; immersion in the well with concrete reinforcement frame to the design mark. One distinctive feature of DDS technology is the drilling instrument (Figure 1). The walls are compacted during drilling up because the well is being excavated concurrently with the drilling, and the soil is compacted radially without the need for excavation [Issakulov and Mussakhanova, 2023, Aldungarova and Abisheva 2022]. This method can be used to arrange heaps up to 30 meters deep and 0.6 meters in diameter. When measuring productivity, the following parameters should be considered: density (soil strength, compaction, and concrete pump power), pile diameter, torque and pressing force applied.

The current study aims to compare the load-settlement response by conducting scaled model tests on model test piles installed using DDS and CFA technology. To validate the results of the laboratory tests and to offer recommendations based on the findings, more field investigations were carried out.

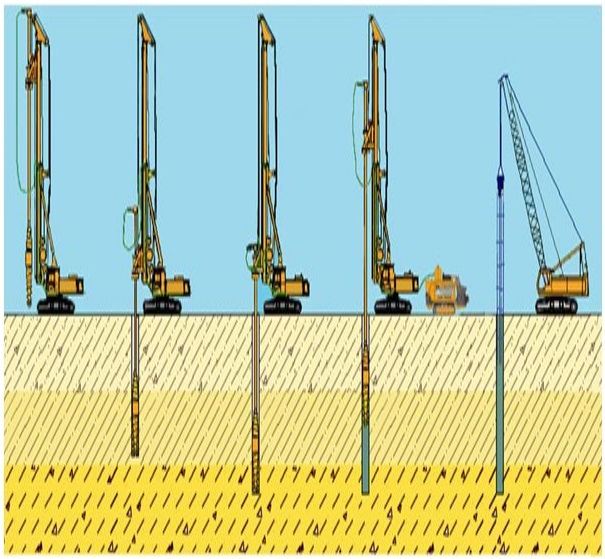


Fig. 1. Installation process of DDS piles and details of drilling shaft

2. LABORATORY SCALED MODEL STUDY

Model tests of piles was implemented on a volumetric stand setup which was developed and implemented in Geotechnical Institute of the Eurasian National University (Figure. 2). The test material used is poorly graded sand of angular shape with a specific gravity of 2.62 and unit weight of 1.7 g/c^3. Based on direct shear test carried out on the test specimen, the sand was found to have an angle of internal friction of 37^o. An equivalent material (97% sand and 3% spindle oil) was used as a base to achieve minimal cohesion of 0.9 kPa and elastic modulus of 0.24 MPa. The test bed was prepared in five layers and each layer was compacted using tamping to achieve a relative density of 75%.



Fig. 2. Volumetric stand

The augers were prepared for installation of model piles by DDS and CFA technologies with a diameter of 20 mm and a length of 300 mm considering a scale factor of 1/20. Figure 3 shows the augers used to install the model pile DDS (a) the model pile CFA (b) which performed by technologies .

, compliance with the verticality of drilling, immersion depth of the working body, and the pressure of concrete when filling the borehole. As a result, high-quality concreting is achieved with smooth and strong walls after unrolling, and pressurized concrete feeding through the hollow unroller.

3. CONCLUSION

Based on the results of testing model piles on a volumetric stand, a comparative analysis of the bearing capacities of model piles using DDS and CFA technologies was obtained. The results showed that the load-bearing capacity of model piles using DDS technology is higher than CFA piles. The results obtained served to recommend this technology for use at the Asyltas construction site. The results of the axial compression loading tests performed in soft to firm or stiff clays demonstrated the suitability of DDS technology pile foundations. The results of the loading testing program confirmed that the DDS bored pile is a viable deep pile foundation option for the construction site in Kazakhstan and demonstrated their advantages. The results of the static load tests were satisfactory, as the maximum test load on the pile was 1080 kN. The settlement was 27 mm These investigations are important for the understanding of soil-pile interaction on the problematical soft soils ground of by different technologies (DDS and CFA) in Astana, Kazakhstan.

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Thermodynamic parameters of earthquake swarms

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ABSTRACT

This article describes a method for recognizing earthquake swarms in seismicity and new data on their thermodynamic parameters. Earthquake swarms of the Northern Tien Shan and adjacent territories, recognized in the seismicity of this region for 2017-2024, are considered. Numerical values of thermodynamic parameters in the sources of swarms are obtained. This article is devoted to the first obtained results of calculating the thermodynamic parameters in the sources of earthquake swarms, which are made using a universal method. It is also shown that the behavior of thermodynamic parameters over time corresponds to the calculation model.

Key words: earthquake swarms, seismicity of the Northern Tien Shan region, thermodynamic parameters

1 INTRODUCTION

Currently, the Earth's seismicity is actively increasing on a global and regional scale. Strong earthquakes occur in seismically active regions of the Earth, such as Turkey, Syria, Afghanistan, China, Taiwan, Japan, Russia, etc. Strong seismic events, for example, in January 2024 in China, Aksu province, M = 7.1, numerous aftershocks had a magnitude of more than 4.0 [1]. Thus, on 03/04/2024 in Almaty there was a noticeable earthquake with M = 4.7. Along with historical catastrophic earthquakes: Vernenskoye , Keminskoye, Chilikskoye and others [2,3], the forecast and prevention of strong earthquakes is a priority task of seismology. The swarms of earthquakes recognized in the seismicity of the Northern Tien Shan region and adjacent territories for 2017-2024 are considered. [1-3, 4-7,10] The need for research is dictated by the current stage of seismicity activation in the entire region. Swarms are background, impulsively arising and also impulsively disappearing shocks, and vibrations of the earth's surface in certain areas of the study region

2 METHOD AND METHODOLOGY OF CALCULATION

Method the MGKl method [8,11] was used to recognize swarms in the seismicity of the region . More details about it can be found in [7, 8,11]. Calculations of thermodynamic parameters in the foci of weak earthquakes (M>1.6) were carried out using the computational method from [4, 9, 10, 13,14,15], which makes it possible to study in more detail the nature of the occurrence of earthquake swarms. For this purpose, swarms (with a magnitude greater than 1.6) of earthquakes were recognized using modern seismic data from the earthquake catalog [1] , in particular, an algorithm and method for recognizing swarms [4,5,7,8,10] were used, and their thermodynamic parameters were calculated [13,14,15] . The calculation was carried out using a universal method [2-4, 7-14], which resulted in obtaining numerical values of the thermodynamic parameters of earthquake swarms for 2017-2024. Note that earthquakes are sources of information about the physical parameters of the earth's crust and its stratification [2-4, 5, 6, 7, 9, 13, 14]. The initial data for determining the thermodynamic parameters of earthquake swarms are the magnitude and energy class of the earthquake. Table 1 shows the numerical values of some thermodynamic parameters of earthquake swarms. Thermodynamic parameters in

earthquake swarms are calculated using known and original equations from [2-4, 7-10, 13].

3 RESULTS

For each magnitude greater than 1.6, the values of the given thermodynamic parameters were calculated using the corresponding formulas from [2, 7, 13, 14]. In this case: the logarithm of the specific (volume) energy density of seismic waves (lgU), unlike (lgV), is in a linear dependence on the magnitude. The value of the potential energy of seismic waves depends on the volume of the source and is practically independent of the specific energy density U, which follows from the differences in the changes in V and U for the range of magnitudes. When considering the relationships between energy and temperature in the sources of earthquake swarms, in theoretical terms, the probability of a relationship between the energy and magnitude of an earthquake with the thermodynamic parameters of the source of the swarms can be assumed from the very nature of the accumulation of thermoelastic stresses in the upper shells of the Earth.

The relationships between the values of magnitude, seismic wave energy and shear modulus, on the one hand, and temperature and effective stresses, on the other, are in inverse proportions. With increasing temperature and, accordingly, effective stresses, the values of magnitude, seismic wave energy and shear modulus decrease. These relationships in the areas of seismically active orogens create a favorable geodynamic environment for the development of neotectonic deformations and partial melting of crustal matter. As can be seen from Figure 1, the contours of earthquake swarms are distributed unevenly throughout the study region in 2017-2024.

4 CONCLUSION

Based on the results of the conducted research on modern seismic data, numerical values of thermodynamic parameters in some sources of weak earthquakes (swarms) in the Northern Tien Shan region and adjacent territories for 2017-2024 were calculated. Thermodynamic parameters characterize the physical conditions in the sources of weak earthquakes, which gives a clearer idea of the nature of the occurrence of swarms. Table 1 Thermodynamic parameters in the foci of earthquake swarms in the Northern Tien Shan region and adjacent territories for 2017-2024 [1]

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Enhancing Slope Stability Under Extreme Rainfall Conditions Through Biomineralization Technology

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ABSTRACT

Intense rainfall is a primary trigger for slope failures and landslides, posing significant challenges to geotechnical stability. While various reinforcement measures have been extensively studied to mitigate rainfall-induced slope failures, innovative approaches are still needed to address these issues effectively. Recently, biomineralization technology has emerged as a promising technique. This approach leverages bacteria or enzyme-induce urea hydrolysis to induce calcium carbonate precipitation, which enhances soil strength and stiffness while reducing permeability, offering a low-carbon and environmentally friendly solution. In this study, the concept of using urease extracted from soybeans to activate biomineralization was proposed for mitigating rainfall-induced slope instability. Simulated rainfall experiments were conducted on slope models with multi-sensor monitoring system. The failure mechanisms and internal moisture migration characteristics of slopes before and after biotreatment under extreme rainfall conditions were studied and compared. The results reveal that rapid increases in moisture content and pore water pressure due to rainwater infiltration are the main causes of slope failure. The biotreatment effectively established a rainwater shielding system, suppressing rainwater infiltration. During simulated rainfall test, the biotreated slope exhibited slower increases in moisture content and pore water pressure compared to the untreated slope. Furthermore, the onset of slope failure was delayed in the biotreated model. This study provides valuable insights into the application of biomineralization technology as a novel strategy to mitigate slope failure under extreme rainfall conditions.

Keywords: Biomineralization technology, Slope stability, Extreme rainfall conditions

1 INTRODUCTION

In hilly areas with a lot of colluvium and leftover soil, rainfall-induced slope failures are common and can seriously endanger both human life and property (Zhuang et al., 2018). Intense rainfall events have been linked to slope failures in a variety of terrain types, including embankments, cut slopes, fill slopes, and natural slopes (Dai et al., 2003). Conventional slope stabilization methods frequently have disadvantages, such as expensive prices, substantial material usage, carbon-intensive construction, and possible environmental damage. Bio-mineralization technology is an emerging soil solidification

technique characterized by low cost, high efficiency, minimal construction disturbance, and environmental sustainability. Via Bio-mineralization process, soil shear strength can be improved, while the permeability of soil can be reduced (Wang et al., 2022).

The primary objective of this study is to experimentally investigate the feasibility of using urease extracted from soybeans to activate biomineralization as countermeasure to enhance slope stability under intense rainfall condition. To achieve this goal, simulated rainfall slope model tests were performed. Under conditions of intense rainfall, internal moisture movement properties of slopes the enhancement mechanisms and were examined and contrasted before and after biotreatment.

2 METHODOLOGY

In current study, 40 g sieved soybean powder was added into 1 L distilled water to obtain the SCUS with the concentration of 40 g/L. Cementation solution (CS) encompassing equal mole concentration (0.5 mol/L) of urea and $CaCl_2$ was applied in this study to assist the process of biomineralization. The test soil used to prepare the slope model is K-7 standard testing silica sand.

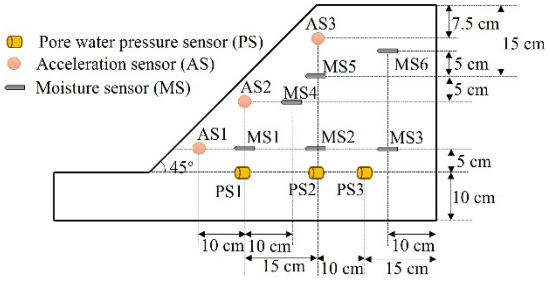
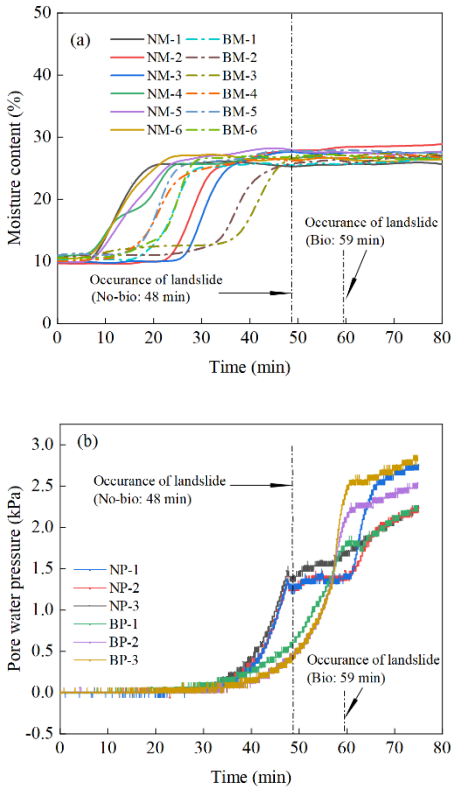


Fig. 1 Layout of sensors

The dimensions of slope models are 800 mm × 400 mm × 450 mm (L×W×H) with slope angle of 45°. The slope model was built with dry density of 1.4 g/cm^3. The initial volumetric moisture content for building the model is approximately 10%. The layout of sensors is shown in Fig. 1. The surface sprinkling method was chosen to complete the biotreatment of the slope. The simulated rainfall tests were conducted in the flume tank where the slope models were built. The rainfall intensity involved in this study was set to 100 mm/h, simulating extreme rainfall condition.

3 RESULTS AND DISCUSSION

The results of the simulated rainfall tests are presented in Fig. 2, showing the evolution of moisture content, pore water pressure, and tilt angle for both the untreated and biotreated slope models. Notably, despite similarities in the hydrological response and slope failure mechanisms between the untreated and biotreated models, the increase in water content and pore water pressure in the biotreated slope occurred with a noticeable delay. Consequently, under the same rainfall intensity, slope failure in the biotreated model was delayed by 11 minutes compared to the untreated model.



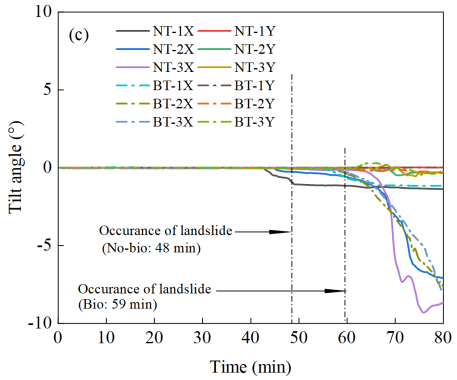


Fig. 2 Monitoring results under rainfall intensity of 100 mm/h : (a) Soil moisture content; (b) Pore water pressure; (c) Tilt angle

4. CONCLUSIONS

This study experimentally examines the effectiveness of soybean-extracted urease in improving slope stability, particularly under intense rainfall. The biotreatmnent measure of biomineralization process induced by soybean urease significantly enhanced the shear strength of soil and reduced its permeability, thereby effectively delaying slope failure.

ACKNOWLEDGEMENT

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Effect of rice husk ash geopolymer amendments on the strength of sandy soils

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ABSTRACT

The 2024 Noto Peninsula Earthquake of 1 January 2024 caused a lot of liquefaction damage. Liquefaction caused extensive damage to many houses, affecting the livelihoods of residents. In Japan, liquefaction countermeasures for detached houses are costly for residents, and this has hindered the introduction of such measures. The authors focused on geopolymer technology. Geopolymers have attracted attention as a cement-free concrete and have been used in buildings in Europe and elsewhere. However, there are few applications in the field of geotechnical engineering. Geopolymers have a mechanism to increase strength through a compound polymerisation reaction between active filler and alkaline solution. In Japanese research, blast furnace slag and fly ash are generally used as active fillers, but previous studies have shown that rice husk ash can also be applied. In this study, rice husk ash was used as the active filler and water glass was used as the alkaline solution. In Japan, one quarter of the rice husk ash discharged from rice cultivation is disposed of as waste. The use of these materials was expected to be cheaper and have a smaller environmental impact than conventional amendments. Strength comparisons were made between improved soils blended at a given mixing ratio. Needle penetration tests were used for strength tests. The reason for this was that the conventional uniaxial compression test, due to its characteristics, may not be able to accurately measure the strength of the soil because the shear plane is formed in a relatively weak part of the whole soil. The results of the tests showed that the target strength was exceeded in many cases.

Keywords: liquefaction countermeasures, rice husk ash, geopolymer, needle penetration test

1 INTRODUCTION

The Noto Peninsula Earthquake of 2024 caused liquefaction damage to many detached houses, and countermeasures against liquefaction are urgently needed. There are various liquefaction countermeasures, such as increasing the density of the ground and lowering the groundwater table, but this study focuses on ground improvement. Cement and water-glass chemicals are generally used as injection materials for ground improvement. However, cement contains carcinogenic substances such as hexavalent chromium and chemical solutions have problems in terms of permanence. To solve these problems, we focused on geopolymer (GP) as a material for ground improvement, which is produced by the condensation polymerisation reaction of an alkaline solution and active filler and has been used as an alternative to cement, but there are few examples of its application in the field of geotechnical engineering. As the application in geotechnical engineering does not require the same strength as that of building materials, a cost reduction can be expected by increasing the moisture content. Rice husk ash, of which approximately 25% is incinerated or disposed of[1)] , was used as the active filler. This is expected to further reduce costs and environmental impact. In this study, GP was used to produce GP-improved soil based on the existing GP mortar formulation. Needle penetration tests were conducted to investigate the effect of alkaline solutions on the strength of GP-improved soil.

2 SETTING TARGET INTENSITIES

Among the many liquefaction countermeasures, the improved soil in this study is considered for application to the permeation solidification method. In the permeation solidification method, it is common practice to set the uniaxial compressive strength to around 100 kN/m² based on the relationship between uniaxial compressive strength and liquefaction strength ratio shown in Fig. 1, so that the liquefaction strength ratio RL, which is the stress ratio at a repetition rate of Nc = 20, is around 0.4-0.6. However, when structural measures (settlement control) are also taken, it is said that the main body of the improvement requires a uniaxial compressive strength of 200 kN/m² or more as the surface improvement strength[2]. As this study targets detached houses and it is important to consider settlement control in liquefaction countermeasures for detached houses, the target strength is set at a uniaxial compressive strength of 200 kN/m².

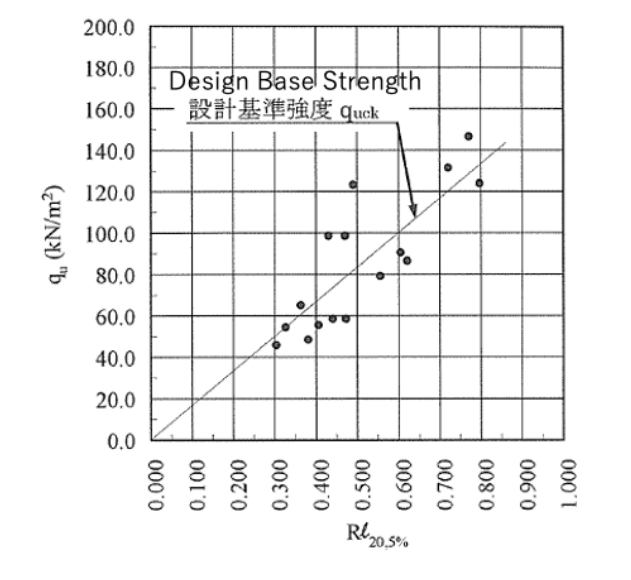


Fig. 1. Relationship between uniaxial compressive strength and liquefaction strength ratio used in the design of infiltration solidification treatment methods (Adapted from the Manual for Permeation Solidification Treatment (revised edition))[2]

3 TEST SPECIMEN

Tohoku silica sand No. 6 was used as soil sample for the GP improved soil. Rice husk ash from Toyama Prefecture was used as the active filler, after being crushed and passed through a 2 mm sieve. Rice husk ash was selected because it has a high content of silica and alumina, and because Japan is a rice-growing country and a stable supply can be obtained. Alkaline solutions were compared in two cases: water glass No. 3 and lime water with a concentration of 0.2%. A 5% concentration of sodium hydroxide solution was used as a reaction accelerator. The test samples were subjected to physical property tests in accordance with the Geotechnical Engineering Society. Table 1 shows the results of the physical properties tests on the sand. The grain size distribution of the sand is shown in Fig. 2. The figure shows that the sand used in this study is close to a single grain size.

Table 1. Physical properties of Northeast Silica Sand No. 6.

Soil particle density ρ_s(g/cm³)	2.61
Natural water content ratio w(%)	0.065
Maximum particle size D_{max}(mm)	0.85
Average particle size D_{50}(mm)	0.31
Maximum void ratio $emax$	0.84
Minimum void ratio $emin$	0.54
Coefficient of Equality Uc	3.50
Coefficient of curvature Uc	1.90

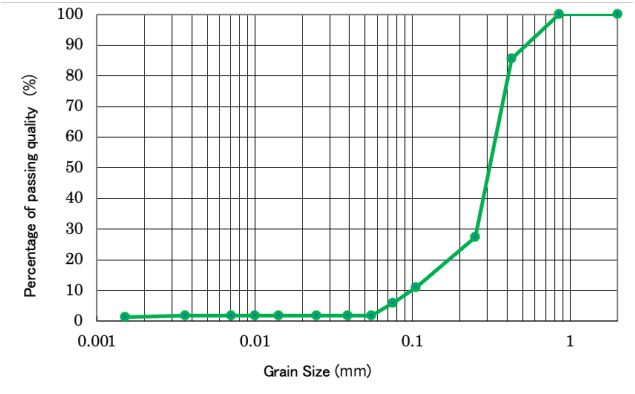


Fig. 2. Particle size distribution of Northeast silica sand No. 6.

4 TEST CONDITIONS AND PROCEDURES

4.1 Percentage of geopolymer improved soil

In this study, the mix proportions of GP mortar with fly ash as active filler by Terai[3], who aimed to develop GP materials using wasabi soil as aggregate, were used as a reference. Terai used fly ash as the active filler and water glass as the alkaline solution, and examined whether geopolymers using these materials could be applied to building materials. The authors followed this proportion and replaced the active filler with rice husk ash as the standard formula. In addition, the authors attempted to observe the effect on strength by adding water in arbitrary proportions. The standard formulations for this study are shown in Table 2.

Table 2. Percentage of GPs in this study (%)

Soil sample	Geopolymer modifier		
Tohoku Silica Sand No. 6	Active Fillers	Alkaline solution	
	Rice husk ash	Water glass (or Lime water)	Sodium hydroxide solution
49	34	12	5

4.2 Specimen preparation method and curing conditions

In a bowl, the materials in the proportions shown in Table 2 were mixed with an arbitrary proportion of water, and the mixture was kneaded by hand until uniform (shown Fig 3). After that, the material was packed into a plastic mold (φ=50mm, h=100mm) in three layers, and each layer was tamped to remove air and solidify the material. After the specimens were prepared, they were air-cured in a curing room (20°C, 70%RH) for a specified period (7~168days).



Fig. 3. Mixing of samples

5 NEEDLE PENETRATION TEST

The needle penetration test is a method to penetrate soil or rock with a needle, measure the penetration length and penetration load, and determine the needle penetration slope from the relationship between them. Soils and soft rocks including solidified soil that can be penetrated by a needle are the main targets of this method. As mentioned earlier, GP is a modified soil and is considered to be a highly disturbed sample, so there is a concern that the strength may be underestimated by uniaxial compression tests. The needle penetration test does not involve failure of the specimen and can be repeated on the same specimen. Therefore, the strength change over time can be confirmed with less error from specimen to specimen. The needle penetration tester used in this study is shown in Figure 4. Figure 5 shows the needle penetration test.

In the needle penetration test, the needle penetration length L (mm) and the penetration load P (N) are read when the needle penetration length reaches 10 mm or when the penetration load reaches the maximum of the testing machine. To derive the needle penetration gradient NP, substitute the needle penetration length L and penetration load P obtained from the penetration test into Equation-(1). The average value of the three tests is used as the needle penetration gradient NP of the specimen.

$$NP=P/L \quad (1)$$

where NP: Needle penetration slope (N/mm)
P: Penetration load (N)
L: Needle penetration length (mm)



Fig. 4. Full view of needle penetration tester

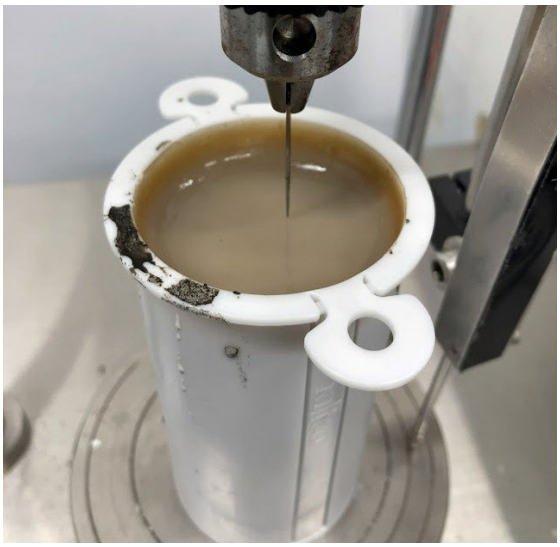


Fig. 3. Measurement in needle penetration test

Figure 6 shows the relationship between the needle penetration gradient NP obtained from the needle penetration test and the uniaxial compressive strength of the specimens prepared under the same conditions. The conversion formula shown in Equation-(2) was derived from the figure. The converted uniaxial compressive strength, quE, calculated from this equation is used as the strength hereafter. Equation -(2) was used for the limewater case as in the water-glass case.

$$\log q_{ue}=0.174*\log NP+2.57. \quad (2)$$

where, q_{uE}: converted uniaxial compressive strength (kN/m^2),
NP: needle penetration slope (N/mm)

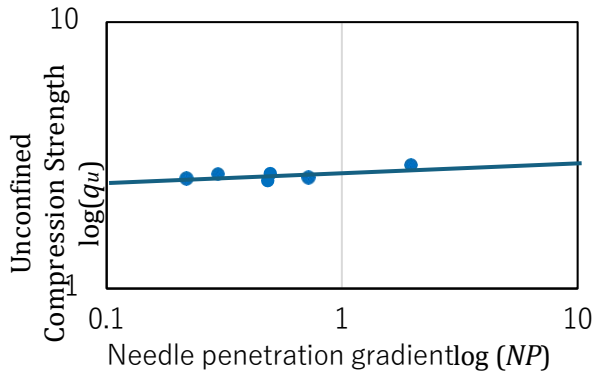


Fig. 4. Relationship between needle penetration gradient and uniaxial compressive strength

6 TEST RESULTS

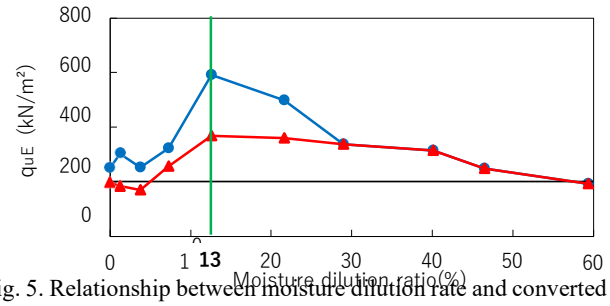


Fig. 5. Relationship between moisture dilution rate and converted uniaxial compressive strength at 28 days of curing

Figure 5 shows the relationship between the water dilution ratio and the converted uniaxial compressive strength, q_{uE}, at 28 days of curing. The water dilution ratio (%) is the mass of added water/total mass of GP amended soil. In all cases where water glass was used, the target strength was met. In all cases, the strength peaked at 13% water dilution. The reason for the lower strength at lower moisture dilution is thought to be the water absorbency of the rice husk ash, which inhibited the penetration of the alkali components. Sato et al.[4] reported that when cement-improved soil mixed with bamboo was stirred, the water absorbency of the bamboo prevented a sufficient hydration reaction and the soil became brittle, and the same phenomenon is thought to have occurred in this case. In addition, the decrease in strength at high water dilution is thought to be due to the decrease in the concentration of the alkaline solution.

Up to this point, the results have been organized using the water dilution ratio. However, the water dilution ratio is not versatile because it is based on a standard formula that mimics Terai's formula, and if experiments are conducted using a different standard, it will not be possible to compare differences in the type and concentration of alkaline solutions. Therefore, a new water-alkali molar ratio W/A (Equation-(3)), which is expressed as the ratio of the molar ratio of the alkali component to the molar ratio of water, was used to organize the results. By using the molar ratio, the properties of the solution can be quantitatively evaluated.

When the water dilution ratio is 13%, W/A is W/A=33 in the case of water glass and W/A=243 in the case of lime water.

W/A=Molar ratio of water / molar ratio of alkali Component (3)

In this section, the strength of the improvement after infiltration is estimated. In the case where rice husk ash was used, the strength peaked at a water dilution ratio of 13%. Since lower viscosity of geopolymer amendments is advantageous for penetration tests, we consider here the range of water dilution above 13% (W/A=33 for the case using water-glass and W/A=243 for the case using petroleum petrochemical water). Figure 6 shows the relationship between the water-alkali molar ratio and the converted uniaxial compressive strength of the alkaline solution in water glass, and Figure 7 shows the relationship between the water-alkali molar ratio and the converted uniaxial compressive strength in lime water at 7 and 28 days of curing. The figures show that the relationship is inversely proportional in both cases. Next, approximate equations are derived from the figures. In general, the strength is measured 28 days after penetration of the solution, but we also propose a case in which the curing period until solidification is short.

In the present case, the strength peaked at 13% water dilution. Since a solution with low viscosity is more advantageous for penetration tests, the cases with water dilution rates higher than 13% (W/A=33 in the case of water-glass and W/A=243 in the case of petrochemical water) were used in this study. In both cases, the converted uniaxial compressive strength decreased as the water-alkali molar ratio increased, i.e., as the alkali concentration decreased.

In Figure-6, the evaluation equations for the converted uniaxial compressive strength at 7 and 28 days of curing are shown in Equations (4) and (5).

$$q_{uE}=13430\,(A/W)+43.323\ (\mathrm{W/A}\geqq 33) \quad (4)$$
$$q_{uE}=11164\,(A/W)+262.39\ (33\leqq \mathrm{W/A}\leqq 85) \quad (5)$$

In Figure-7, the evaluation equations for converted uniaxial compression at 7 and 28 days of curing are shown in Equations (6) and (7).

$$q_{uE}=13430\,(A/W)+43.323\ (\mathrm{W/A}\geqq 243) \quad (6)$$
$$q_{uE}=11164\,(A/W)+262.39\ (243\leqq \mathrm{W/A}\leqq 428) \quad (7)$$

Normally, the 28th day when strength is stable should be used. However, if you want to design considering the initial strength, you can use the equation for the 7th day of curing. It is shown that the target strength can be met if the strength is in the range of $33\leq \mathrm{W/A}\leq 85$ for water glass and $243\leq \mathrm{W/A}\leq 428$ for lime water.

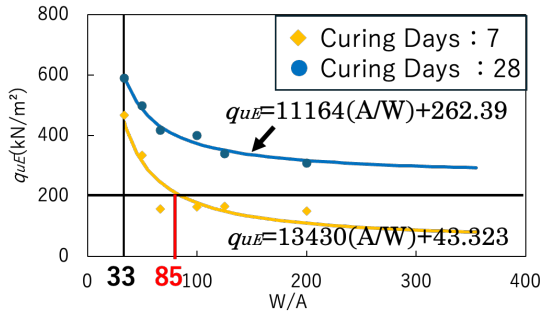


Fig. 6. Relationship between crown-pile uniaxial compressive strength and water-cement ratio in water-glass at 7 and 28 days of curing time.

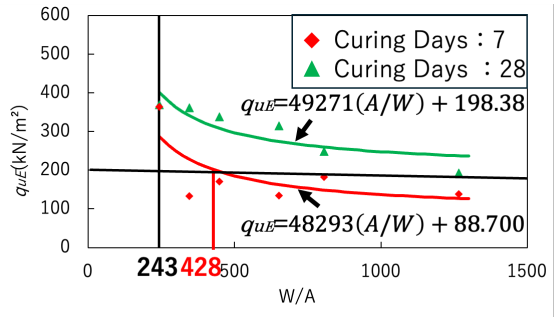


Fig. 7. Relationship between crown crest uniaxial compressive strength and water-cement ratio in lime water at 7 and 28 days of curing time

8 FUTURE ATTEMPTS

In the previous chapter, it was confirmed that the use of rice husk ash as an active filler can meet the target strength even when the moisture dilution ratio is increased from 0 to 22% from the standard formulation. However, when these samples are applied to actual ground, not only strength development but also permeability into the ground is important. Therefore, in this study, we attempted to conduct a three-dimensional permeation test using rice husk ash in a simple soil tank. Figure 8 shows a full view of the 3D penetration test apparatus.

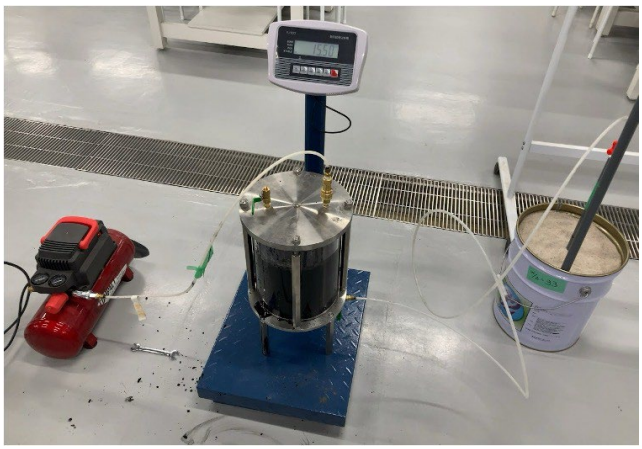


Fig. 8. Full Description of the 3D Penetrant Testing System

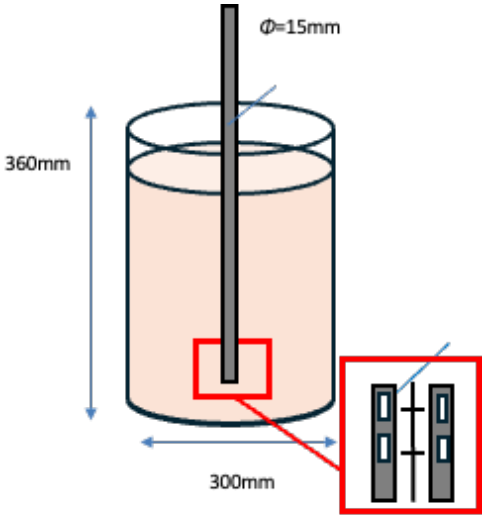


Fig. 9. Overview of soil tanks using pails

In this study, Tohoku silica sand was placed in cylindrical pail tubes with an inner diameter of 300 mm and a height of 360 mm. Commercially available vinyl chloride pipes (15 mm in diameter) were used as injection pipes. Two holes were drilled in the PVC pipe, one in the front and the other in the back, as injection holes. When the injection pipe was placed in the tank, sand was expected to flow in through the injection holes, preventing the solution from flowing, so a mesh wire mesh was wrapped around the pipe and secured with rubber bands. To control the injection volume by weight, the test was conducted with the stirrer on a scale.

A simple three-dimensional penetration test was conducted in the case of rice husk ash + lime water with W/A = 428 (the highest moisture content among those meeting the target strength). However, as shown in Figure 10, even when the wire mesh was wrapped around the injection pipe, sand flowed into the injection hole, causing clogging. Although the wire mesh was secured with rubber, some sand may have entered through gaps at the top and bottom when sand was injected. The viscosity of the GP solution was higher than expected, and additional water had to be added to improve permeability.

Since the injection holes were clogged and no improved material could be obtained, several remedial measures are listed below. First, the use of a wire mesh with a smaller mesh is considered to prevent the inflow of sand. Secondly, it is thought that water flow during infiltration can promote infiltration. Hashimoto[5] injected cement milk into Tohoku Silica Sand No.7 and Tohoku Silica Sand No.8 and measured the penetration distance using a one-dimensional penetration test. The cement milk tended to clog due to a decrease in the water-cement ratio (increase in cement concentration), resulting in a decrease in the infiltration distance. The authors report that the water-cement ratio of the cement milk was similar to that of the water-cement milk, but the water-cement ratio was lower than that of the cement

milk, and the penetration distance was clearly increased. The authors also found that the water flowed through the cementitious media during the process of permeation, and that the solution was able to spread to the edges of the cementitious media.



Fig. 10. Clogged injection pipe after penetration test

10 CONCLUSIONS

The results of needle penetration tests on geopolymer-amended soil with rice husk ash showed that the target strength could be met even with a diluted concentration of alkali solution . From the test results, an evaluation formula for uniaxial compressive strength using alkali concentration was proposed.

ACKNOWLEDGEMENTS

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The influence of engineering and geological conditions on the zoning of urban development

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ABSTRACT

This study examines the significance of zoning urban areas based on engineering and geological conditions, emphasizing its role in ensuring reliable and sustainable construction. Despite the necessity of accounting for soil types and geological features in urban development, these factors are often overlooked in design and planning, leading to suboptimal land use. The research focuses on the development of a geoinformation database (GIS) for analyzing geotechnical properties and creating specialized geotechnical maps. Based on field surveys and laboratory testing, the engineering and geological characteristics of soils in Pavlodar were classified, considering their physical and mechanical properties. Given the rapid pace of modern construction, the study highlights the need for efficient methods to determine soil characteristics. Errors in geotechnical assessments due to limited investigation data can lead to overestimation or underestimation of soil properties, particularly in fine-grained clayey soils, which are highly sensitive to moisture content and consistency variations. This research employs correlation analysis to establish relationships between strength and deformation properties of loams and clays of different genetic types based on liquidity index. The analysis is conducted using technical reports from various engineering-geological surveys in Pavlodar. The resulting equations expand the range of mechanical property estimations, facilitating more efficient geotechnical assessments. Given Pavlodar's status as an industrial hub, construction activities are increasing, necessitating more comprehensive engineering and geological studies. Long-term observations indicate that anthropogenic impacts on the geological environment contribute to issues such as erosion, waterlogging, and soil salinization. To mitigate these effects, scientifically based urban planning solutions must integrate geotechnical data into zoning methodologies. This study underscores the importance of geotechnical classification in optimizing engineering-geological surveys, improving economic efficiency, and enhancing construction reliability.

Keywords: zoning, engineering-geological conditions, geoinformation database, soil properties, geotechnical mapping,

1 INTRODUCTION

In modern construction practice and urban planning concepts, the defining components of successful project implementation are competent zoning of sites and detailed analysis of their geological characteristics. Zoning, as a methodological process of dividing territories according to their functional purpose, has evolved from primitive forms based on observations and traditions to scientifically grounded methods.

Urban planning currently struggles with fragmented stages of engineering-geological research, project development, and foundation design, which lowers construction efficiency and reliability. Soil classification by physical and mechanical properties is conditional—soils suitable for low-rise buildings may not support high-rises.

The Kazakh State Institute of Engineering Surveys (KazGIIZ) exemplified a systematic approach during the Soviet era, with branches across the republic. After the USSR's collapse, many important KazGIIZ documents were lost or privatized, restricting access to vital geological data (Zhussupbekov et.al., 2021).

This study focuses on Pavlodar, a rapidly growing city in northeastern Kazakhstan, where urban expansion highlights the need for geotechnically-informed

planning. Its geographic and geological context reflects common challenges in mid-sized post-Soviet cities, including data gaps and development pressures. Fig. 1 shows Pavlodar's location within Kazakhstan.



Fig. 1. Map of Kazakhstan with location of Pavlodar

2 METHODOLOGY OF RESEARCH

2.1 Integration of engineering and geological data into GIS

The geoinformation database developed in this study uses a two-level hierarchical structure for efficient management, processing, and visualization of engineering-geological survey data. The upper level handles administrative control and graphical workflow coordination, while the lower level processes input data and organizes its graphical display.

Input data is divided into two types: fixed datasets integrated into the software (such as base maps and coordinate references), and user-generated data from field surveys prepared in CAD software like AutoCAD. This includes creating detailed survey maps and compiling tabular data for each borehole (see Table 1) covering soil depth, geological age, classification, coded soil conditions, and alternating soil layers for stratigraphic interpretation (Mimura and Yamamoto, 2014).

The data includes deposit depth, soil age, and name to define the stratigraphic framework. Soil conditions are coded with zero for loose soils and codes 1 to 4 for various semi-rock and rock types. The spatial distribution of boreholes across the study area is presented in Fig. 2.

Table 1. Soil column data.

RB CODE	RB CODE	Lower Depth (m)	EGE	Soil_INDEX	Soil CODE	SOU
B-01	PVDRR001B001	0.4	1	tQ_{IV}	Pr	FS
B-01	PVDRR001B001	3.6	2a	adQ_{I-IV}	Lm Sn	RO SA
B-01	PVDRR001B001	6.5	6c	N	1Cl	CL SA
B-01	PVDRR001B001	7.8	5	N	3Cl	RO SA CL
B-01	PVDRR001B001	8.6	2a	N	1Cl	RO SA



Fig. 2. Borehole location map of Pavlodar

Thus, the developed geoinformation database ensures efficient management and processing of engineering-geological data, providing a solid foundation for further analysis and assessment of construction conditions. Based on the obtained data, an evaluation and detailed typification of the engineering-geological conditions of the territory was carried out.

2.2 Typification of Engineering-Geological Conditions in Pavlodar

Building on the created database and the collected initial information, the second chapter presents the evaluation and typification of the engineering-geological conditions of the city of Pavlodar. This process is a crucial step in assessing the suitability of the territory for various types of construction and enables identification of key zones with distinctive geological features, essential for rational planning and design.

3 CONCLUSIONS

This study highlights the crucial role of engineering-geological conditions in urban zoning and planning. The development of a geoinformation database and geotechnical mapping improves soil property assessments, essential for rapidly growing cities like Pavlodar. Integrating geotechnical data into urban planning helps mitigate risks from anthropogenic impacts and ensures sustainable infrastructure development. Overall, this approach optimizes engineering surveys, reduces costs, and increases the reliability of building projects.

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Environment - preservation and development of folk crafts in the settlement structure of Kazakhstan

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ABSTRACT

The study of modern trends and prospects for the development of craft centers, the experience of designing craft centers and educational institutions of applied crafts in the system of educational institutions and the construction of various universal professional lyceums and colleges, fair and tourist complexes in Kazakhstan and abroad, as well as our own research allow us to determine a number of principles for the formation of educational and production centers of folk crafts, to propose general theoretical models, basic regulatory indicators, to determine the optimal capacity taking into account the population and needs of a given region. The article proposes principles for the formation of the architecture of training and production centers of folk arts and crafts. A general theoretical model of the functioning and stage-by-stage development of centers and principles for their placement in the settlement structure of Kazakhstan are developed, proposals are given for the calculation and nomenclature of the network and types of training and production centers of folk arts and crafts.

Keywords: folk arts and crafts, training and production centers, theoretical model, nomenclature of building types, architectural planning structure

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1 INTRODUCTION

Currently, the implementation of promising programs has begun, during which a set of measures will be implemented aimed at the targeted and systematic development of the Kazakh handicraft industry by entering the international arena. In order to localize efforts to develop crafts in the regions, it is planned to create information and resource centers for crafts in the cities of Shymkent, Astana, Turkestan and Aktau. The project plans to provide technical equipment, organize craft exhibitions and fairs and conduct a series of professional technical trainings for artisans through these resource centers. In the future, it is planned to open a network of similar information and resource centers for artisans throughout Kazakhstan.

Craft resource centers will be equipped with craft equipment and office equipment to provide a wide range of services for local artisans. The program trainers will acquire and strengthen their experience in conducting trainings and master classes for artisans. The population of Kazakhstan will be informed about their crafts and achievements through craft exhibitions and fairs organized in the relevant regions. The general public of the regions will be informed about the problems of craft development and will be able to gain access to quality craft products. The youth of the regions will have the opportunity to get involved in crafts.

The foundation of a national network for the development of crafts will be laid. The development of new sales channels for their products by artisans of the two regions, such as local tourist sites and attractions, and as a result - an increase in product sales. Creation of new jobs and development of entrepreneurship in the region.

2 GENERAL SPECIFICATIONS

The above economic and social analyses, principles of sectoral and regional placement of crafts are necessary, but not the dominant factors determining the feasibility of the formation and development of craft centers of one type or another and their location in Kazakhstan. The artistic, historical value of the craft, its uniqueness and widespread use in a given region, its place and role in the architectural and urban planning level, as well as in the search for ways to develop them in modern society remain dominant. Architectural appearance of objects of folk applied crafts. It is practically impossible to single out specific features inherent in the architectural appearance of modern production and educational objects of folk applied crafts, as a special category of buildings. Nowadays, specially designed objects bearing all the features of modern typical public and factory architecture erected in our country in recent decades, as well as adapted premises in similar buildings are used as production buildings of

industries. Only in a few isolated cases, the objects of the scientific and industrial work are located in buildings that are cultural monuments (often reconstructed or surviving premises of historical and ethnographic value). The main requirements that decisions taken when designing scientific and industrial work centers must meet are the following: - functionality - ensuring the best conditions for the full implementation of all functions of a given production, working conditions conducive to obtaining the best results, taking into account the historically established forms and nature of the production activities of a given industry; - aesthetics - creation by means of architecture of a harmonious subject environment providing the best conditions for work, life and rest of people; compliance with the developed patterns that meet the concepts of beauty; ensuring the imagery of architecture that corresponds to the creative direction of the craft; - connection with the external environment - historically established architectural and spatial, natural and landscape, socio-cultural, socio-economic, in which the craft exists.

CONCLUSIONS

The value of this topic of the article is to increase the competitiveness of domestic enterprises of folk crafts (small and medium-sized businesses) based on the integration of education, production and business by creating a single network of unified architectural and planning measures (a general theoretical model of functional zoning and specific proposals for the comprehensive planning of educational and production facilities of folk crafts). Integration processes in this topic will develop in a certain architectural and planning, urban planning and functional-organizational form. It is assumed that the modernization of the architecture of the UPC NP through the integration of education, science and production will lead to an increase in the innovative potential of the Kazakhstan economy.

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Quantitative risk assessment of ground deformations in Azerbaijan Onshore Petroleum and Gas Fields

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ABSTRACT

Numerous productive oil and gas fields are located in the Absheron Peninsula of Azerbaijan. The primary aim of this study was to quantitatively evaluate the rates of ground deformation, including subsidence and uplift, within these fields. Additionally, the study sought to identify both natural and anthropogenic factors influencing these deformations and to forecast future trends in ground movement. Ground deformation caused by hydrocarbon extraction presents serious risks to both infrastructure and environmental stability in petroleum-producing regions. This study uses Persistent Scatterer Interferometric Synthetic Aperture Radar (PS-InSAR) to assess land deformation in the Absheron Peninsula, Azerbaijan—a region with intensive oil and gas activities. By integrating satellite radar data from 2015 to 2023 with spatial statistical methods, including Geographically Weighted Regression (GWR), the research identifies deformation hotspots and correlates them with influencing factors such as elevation and well density. Subsidence and uplift rates reach extremes of −26 mm/year and +23 mm/year, respectively. GWR reveals that up to 32% of subsidence variance can be explained by well concentration and elevation, underlining the importance of monitoring anthropogenic impacts on land deformation.

Keywords: instruction, proceedings, geotechnical engineering, symposium

1 INTRODUCTION

Ground deformation, including both subsidence and uplift, is a critical hazard in petroleum-rich regions due to extraction-induced pressure changes in underground reservoirs. This is particularly relevant in the Absheron Peninsula of Azerbaijan, where historical and ongoing oil and gas activities have left a significant environmental footprint (Feyzullayev & Ibragimov, 2014). The risks include damage to infrastructure, increased flooding potential, and long-term land instability.

Satellite-based Interferometric Synthetic Aperture Radar (InSAR) techniques, especially Persistent Scatterer InSAR (PS-InSAR), have emerged as powerful tools to detect surface deformation with millimeter precision over wide areas (Ferretti et al., 2001; 2007). Unlike traditional ground-based monitoring systems, InSAR offers consistent temporal monitoring without the need for in-situ installations.

The Absheron region's geodynamic complexity, including tectonic movement and mud volcano activity (Mellors et al., 2005), further complicates subsidence patterns. To distinguish between natural and anthropogenic influences, it is critical to integrate deformation measurements with spatial statistical techniques. The objective of this study is to provide a quantitative assessment of ground deformation trends and identify their controlling factors through a combination of PS-InSAR data and spatial regression models.

2 MATERIALS AND METHODS

This study employed PS-InSAR techniques to process Sentinel-1 radar imagery from 2015 to 2023 across the Absheron Peninsula (Ferretti et al., 2001; 2007). The technique isolates stable radar reflectors over time, enabling precise deformation velocity mapping. The vertical displacement trends were extracted and interpolated for regional analysis.

To identify deformation hotspots and evaluate controlling factors, spatial statistical techniques, notably Geographically Weighted Regression (GWR), were applied. Independent variables included elevation, slope, and oil well density, derived from digital elevation models (DEMs) and field datasets. Deformation rates were modeled as the dependent variable to quantify their spatial association with these factors.

Hotspot analysis using the Getis-Ord Gi* statistic was also employed to highlight clusters of significant uplift or subsidence. The workflow is presented in Figure 1.

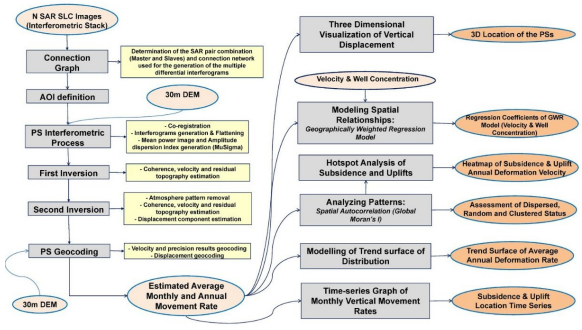


Fig. 1 Workflow of PS-InSAR, geospatial analysis and predictions of ground deformation processes

3. RESULTS

The PS-InSAR analysis detected three major deformation zones within the study area. Maximum observed subsidence reached −26 mm/year, while uplift reached +23 mm/year (Figure 2). These patterns were spatially uneven, with two zones closely associated with dense clusters of oil and gas wells, suggesting anthropogenic influence.

Spatial density analysis of deformation velocities revealed underlying natural patterns associated with tectonic processes of uplift and subsidence. The findings indicated that two oil and gas fields are predominantly influenced by anthropogenic activities related to exploration, while one field experiences effects from both natural tectonic and anthropogenic processes.

GWR analysis revealed that the combination of well density and elevation explained 32% of the spatial variability in subsidence, with localized areas showing stronger correlations. The inclusion of slope had minimal additional explanatory power. Hotspot analysis confirmed that deformation is significantly clustered in operational oilfields, particularly in Bibi-Heybat and Lokbatan.

Further analysis using linear regression (R^2) and ordinary least squares (OLS) demonstrated a temporal correlation in ground deformation processes for the aforementioned oil and gas fields, with a predicted monthly deformation rate ranging from −2.54 mm/month for subsidence to 1.89 mm/month for uplift.

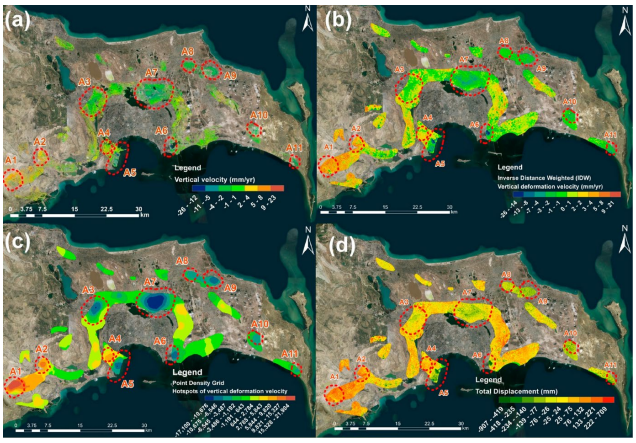


Fig. 2 (a) Deformation map of subsidence and uplifts of the Absheron oil and gas fields; (b) Inverse Distance Weighted interpolation of deformation values of persistent scatterers; (c) hotspots of vertical deformation velocity; (d) Inverse Distance Weighted interpolation of total deformation values of persistent scatterers

4. DISCUSSION AND CONCLUSIONS

This study demonstrates the effectiveness of integrating PS-InSAR and spatial statistics to assess and interpret ground deformation in petroleum-producing regions. Results confirm that human activities, especially oil extraction, are key contributors to land subsidence in the Absheron Peninsula.

The moderate but significant explanatory power of well density and elevation suggests that both anthropogenic and natural geodynamic processes play a role. However, the clustering of deformation around operational wells highlights the need for better management and monitoring practices.

The study showcased the effectiveness of a cost-efficient and safe technology for monitoring ground deformations in these fields. Furthermore, the results enhance the scope and reliability of observations, providing improved clarity regarding ground deformation trends, rather than focusing solely on isolated measurement points.

Overall, the research highlighted the significant role of Synthetic Aperture Radar (SAR) technologies in facilitating safe operational monitoring of petroleum and gas fields, eliminating the necessity for human ground-based measurements. This approach not only reduces costs but also enhances the reliability and scope of observations. The study successfully integrated Persistent Scatterer Interferometry with geospatial machine learning-based geostatistical analysis, leading to a more comprehensive understanding of ground deformation processes across the oil and gas fields of the Absheron Peninsula (Bayramov et al., 2020).

Given the potential risks to urban infrastructure and environmental systems, continuous satellite monitoring and the integration of ground-based validation are recommended. The approach developed in this study is scalable and can be applied to other oil and gas basins worldwide.

FUNDING AND ACKNOWLEDGEMENTS

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An experimental study of the year-to-year changes in the patterns of freezing and thawing of the pavement foundation

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ABSTRACT

A two-year monitoring program (2021–2023) on a roadbed in northern Kazakhstan recorded hourly temperature and moisture to depths of ~2.4 m. The freezing front reached 2.26 m in 2021–2022 and 2.51 m in 2022–2023. Shallow layers showed abrupt apparent moisture loss during freezing and rapid gains during thaw, while deeper layers changed more gradually. These patterns influence base compressive properties, stress–strain state, and pavement durability. The findings support measures such as drainage, groundwater control, and geosynthetic reinforcement, and emphasize monitoring over multiple winters to account for interannual effects in cold-region pavement design.

Keywords: road clothing, ground base, temperature, humidity, water-thermal regime, stress, deformation, durability, reliability, modulus of elasticity, bearing capacity

1 INTRODUCTION

In cold-region pavements, the stress–strain state depends on subgrade temperature and moisture. In northern Kazakhstan (West Siberian climatic zone), long, severe winters cause large interannual variations in the water–thermal regime, often leading to spring failures of asphalt-concrete. Reliable design and maintenance require explicit consideration of freeze–thaw and moisture variability (Chang and Li, 2021).

Key field study considerations:

a) *Interannual variability* – Seasonal severity, freezing depth, and moisture profiles differ; robust results require multi-year observations.

b) *Depth-resolved monitoring* – Measure temperature and moisture at multiple depths with high frequency to capture rapid changes; sparse data risk errors.

c) *Site controls* – Soil type, drainage, and groundwater level shape moisture gradients, stiffness, and bearing capacity, affecting durability and cracking risk.

This study conducted a two-winter program near Kostanay, recording hourly temperature and moisture at several depths to assess freeze–thaw behavior and variability. The results compare seasons, explain moisture jumps during thaw, and suggest mitigation—drainage, groundwater control, and geosynthetic reinforcement—to reduce spring failures (Chen and Zhang, 2024).

2 MATERIALS AND METHODS

2.1 Site and pavement.

The study site is a highway section near Kostanay (Karabutak–Kostanay–Yekaterinburg–Almaty corridor), northern Kazakhstan, characterized by a long, severe winter. The pavement consists of asphalt-concrete layers over a granular base and sandy subgrade.

2.2 Instrumentation and sensor layout.

A field station recorded hourly temperature and volumetric soil moisture at nine depths in the subgrade: 11, 22, 40, 60, 80, 120, 160, 200, and 240 cm. Sensors were installed vertically below the shoulder to minimize surface disturbance. Air temperature and precipitation were taken from the nearest meteorological post.

2.3 Monitoring periods and logging.

Two winter seasons were monitored: 01-Nov-2021→15-May-2022 and 01-Nov-2022→15-May-2023. Data were stored at 1-hour intervals with routine

synchronization and quality checks to remove spikes or short outages.

2.4 Definitions and derived metrics.

The freezing depth was defined as the deepest zero-isotherm crossing that remained below 0 °C continuously. Moisture dynamics were summarized by apparent drops during freezing (phase change) and abrupt rises during thaw. For each depth we computed daily minima/maxima and seasonal envelopes; season-maximum freezing depths were compared between winters.

2.5 Data processing and analysis.

Time series were aggregated to daily extrema and depth profiles at key dates (mid-winter, onset of thaw). Cross-plots of temperature versus moisture highlighted coupling at each depth. Results are reported comparatively for the two winters and related to the pavement structure to explain depth-dependent behavior.

Name of the measuring station	Geographical latitude	Geographical longitude
Karabutak-Kostanay-Yekaterinburg-Almaty, Kostanay", KM 1	53,139900	63,551765

Materials of structural layers	Thickness of layers
1. asphalt concrete, type B	h_1=5 cm
2. coarse-grained porous asphalt concrete	h_2=13 cm
3. coarse-grained, highly porous asphalt concrete	h_3=9 cm
4. mixture C-4	h_4=35 cm
5. ground sand	h_5=90 cm
6. soil chernozem	h_6=148 cm

Vertical coordinates of the sensor location	y_1	y_2	y_3	y_4	y_5	y_6	y_7	y_8	y_9
Numeric values, see	3	11	22	45	78	111	145	180	240

3 RESULTS AND DISCUSSIONS

The measuring station was equipped with 11 temperature and humidity sensors at depths *y1–y9* and one air sensor at 2 m height. Measurements were recorded hourly and transmitted online.

From Nov 1, 2021 to May 15, 2022 (196 days), each depth accumulated 4704 readings. To reduce excessive data compression, daily temperature minima were analyzed instead of all hourly values.

Comparison of temperature and humidity changes (depths 45–145 cm) revealed a strong correlation: rapid surface cooling at the start of winter caused an abrupt drop in moisture due to freezing, which is not detected by sensors in ice form. During thawing, moisture levels rose sharply from melting ice and frost. At greater depths, both parameters changed more gradually, showing stable thermal-moisture conditions throughout the season (Chang and Li, 2021).



Photo 1. Field station and borehole used for installing temperature and moisture sensors.

4 CONCLUSIONS

The figures show only part of the results, highlighting differences in water and heat regimes of road structures under slight annual weather variations in northern Kazakhstan. A deeper study will help determine optimal construction and operation modes.

Soil moisture variability with depth affects pavement stress–strain state. Humidity gradients cause deformations, reduced bearing capacity, and faster deterioration. Modern modeling and experiments allow prediction and stabilization, including drainage systems, geosynthetics, and optimized designs (Chen and Zhang, 2024).

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Reconstruction of the ventilation system of the automobile tunnel of Too-Ashuu named after K.Kolbaev

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ABSTRACT

The Bishkek-Osh highway, 672 km long, is the only communication route between the northern and southern regions of the country and is strategically important. This road is part of an international transport corridor. At the 129th kilometer, it passes through the tunnel named after. K. Kolbaeva.
In 2001, after reconstruction, 11 Korean-made fans were installed inside the tunnel, but due to power outages and tunnel congestion, the fans quickly failed and had to be replaced in 2013 with fans manufactured in China.
At the moment, all the fans are in working order, but due to the increase in the intensity of the flow of trucks powered by an internal combustion engine, gas pollution inside the tunnel is difficult to cope with, and the decision is not just to replace the fans with new ones like the previous time, but to make a comparative analysis of the options for calculating ventilation systems for this tunnel and according to the chosen optimal option, it is necessary to carry out reconstruction of tunnel ventilation systems in the near future.
The health of the people there and the safety of transport operations depend on the proper and uninterrupted operation of ventilation systems. This work aims, using the rich experience of tunnel ventilation in foreign countries, as well as the research conducted by the authors of the article, to expand the range of knowledge in this field and help specialists of design and construction organizations make informed decisions when choosing tunnel ventilation systems during their construction and operation, as well as when teaching students of relevant specialties. The methods of calculating the required air flow are described and a method for calculating it is proposed, taking into account the peculiarities of traffic in a high-altitude tunnel, taking into account the peculiarities of the highlands. To identify a number of factors contributing to a more stable mode of operation of ventilation systems of road tunnels, and to outline further ways of research and development in this direction.

Keywords: traffic flow, traffic intensity, traffic modes, combustion products, emissions, concentration.

1 INTRODUCTION

The Too-Ashuu automobile tunnel, located in the Kyrgyz Republic at an altitude of more than 3,200 meters above sea level, is one of the most complex transport infrastructure facilities in the region, crossing the Kyrgyz Ala-Too Mountain range. Its operation in conditions of low atmospheric pressure, sudden temperature fluctuations and intensive air circulation in mountain valleys places special demands on the ventilation system, which must provide not only standard conditions for the movement of vehicles, but also effective fire protection. At the time of the inspection, the tunnel had a length of 2,510 meters, a cross-sectional area of about 38 m2, two-lane traffic, an approximate slope of up to 3.5% and had no escape passages.

2 MAIN VENTILATION PROBLEMS

The main problems of the existing tunnel ventilation scheme include: the use of outdated jet fans that are not designed to operate in thin air conditions; the lack of automated monitoring of air pollution and its condition; instability of air flows during transitional periods (winter-spring); and the lack of backup smoke extraction systems. Field studies have shown that under maximum load conditions (especially from diesel trucks), the CO level exceeds 70 mg/m^3 compared to the permissible limit of < 50 mg/m^3, and the smoke extraction rate during fire simulation does not meet fire safety requirements.

Thus, the reconstruction of the ventilation system of the Too-Ashuu tunnel is based on the introduction of engineering solutions adapted to the conditions of the highlands, modern automation systems, reliable power supply and digital monitoring. This will increase the level of safety, reduce the risks of emergency situations and ensure a standard level of air humidity in difficult climatic and operational conditions.

3 CALCULATION OF THE LONGITUDINAL VENTILATION SCHEME OF A ROAD TUNNEL

3.1. Calculation of the required amount of air for tunnel ventilation

Calculation of the number of vehicles of each type located in the tunnel for rush hour conditions and complete cessation of traffic.

In the absence of data on the predicted traffic intensities of vehicles, the calculation is performed according to a given value of the given vehicles using the formulas (E.1) - (E.13) applications E [1].

For traffic mode "A", the traffic intensity is found by the formula (E.7) applications E [1] at the traffic intensity of the above vehicles, 1875 ave.auth./hr

$$C_{\text{rush hour}} = \frac{1875}{[(1-0,2)+2\cdot 0,2]} = 1563\text{tr.wed/hr.} \quad (1)$$

The traffic intensity of passenger cars with gasoline and diesel engines, light trucks, heavy trucks weighing up to 15 tons and buses, heavy trucks weighing up to 32 tons is calculated using the formulas (E.8) - (E.13) applications E [1].

$A_{pc.g} = 1563\cdot(1 - 0,01\cdot 20)(1 - 0,01\cdot 20)/(1+0,01\cdot 10) = 911$ car/h;
$A_{pc.d} = 1563\cdot(1 - 0,8)\cdot 0,01\cdot 20/(1 + 0,01+10) = 227$ uth./hr;
$B_{lt.g} = (B_{l.t.g} + B_{l.t.d}) = 58+55=113$ car/hr;
$C_{t1} = 0,001\cdot 20\cdot 75\cdot 1563=234$ car/hr;
$C_{t2} = 0,01\cdot 20(1-0,01\cdot 75)\cdot 1563 = 78$ car/hr;

The number of vehicles of various types located simultaneously in the tunnel is determined by the formulas (E.1) - (E.6) applications E [1].

$$n_{pc.g} = 911\cdot 2,51/40 = 57;$$
$$n_{pc.d} = 227\cdot 2,51/40 = 14;$$
$$n_{l.t.g} = 113\cdot 2,51/40 = 7;$$
$$n_{t.1} = 234\cdot 2,51/40 = 14;$$
$$n_{t.2} = 78\cdot 2,51/40 = 5$$

For calculations of slow motion mode "B" and traffic stop mode (traffic jam) "C" The traffic intensity of vehicles of various types and the number of vehicles in the tunnel are determined in the same way, respectively, with the traffic intensity of the vehicles in rush hour, which is 57% of the traffic intensity at the set speed, and the number of vehicles in the tunnel, calculated at 165 tr.cp./ km per lane. The calculation results are shown in Table 1.

Table 1. Traffic intensity and number of vehicles n of different types for driving modes "A", "B" and "C".

Driving mode	The value of the indicator for the type of vehicle									
	$A_{pc.g}$	$n_{pc.g}$	$A_{pc.d}$	$n_{pc.d}$	$B_{lt.g}$	$n_{lt.g}$	C_{t1}	$n_{l.t.g}$	C_{t2}	$n_{t.2}$
Mode «A»	911	57	227	14	113	7	234	14	78	5
Mode «B»	497	60	124	15	62	7	120	14	51	6
Mode «C»	-	132	-	35	-	18	-	37	-	14

3 CONCLUSIONS

The renovation of the tunnel ventilation system includes upgrading or replacing existing ventilation systems to ensure efficient and safe air circulation. This may include improved filtration, increased fan power, optimized duct placement, and increased control over air parameters such as temperature and humidity.

Based on the calculation, we provide a solution:

- to replace 11 existing axial fans, their total capacity at nominal pressure was no more than 181.5 m^3/sec, which was not enough to ensure the normative multiplicity of air exchange even with normal loading of passenger cars, not to mention diesel trucks and emergency situations. At a reduced atmospheric pressure (on average 69-70 kPa) and an air density of 0.91 kg/m^3, the performance of ventilation equipment decreased by 25-30% of the rated characteristics. In addition, the system did not have automatic control, pollution sensors and reverse equipment, which made it unsuitable for the rapid removal of combustion products in case of fire;

- the main fans are equipped with 4 fans per two gantries with two installations each, which can supply 238,3 m^3/sec. A fresh jet will be supplied through a ventilation pipe d = 1000 mm, which will be installed in the roof.

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Application of geosynthetic materials to increase the strength of road foundation soils

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ABSTRACT

The article discusses the use of geosynthetics as an effective solution to the problem of increasing the strength and stability of road foundation soils, especially in difficult engineering-geological and hydrogeological conditions. An overview of modern technologies is presented, including reinforcement, drainage, filtration and insulation using geogrids, geotextiles, geomembranes and geogrids. Examples of successful use of geosynthetics in road construction, experimental data, calculation methods and recommendations for choosing a material depending on geological conditions are given. It is shown that the use of geosynthetics significantly increases the bearing capacity of the foundation, reduces settlements and deformations, and increases the durability of road structures.

Keywords: geosynthetics; soil stability; reinforcement; road base; geotextile; geogrid; road construction; drainage; slope stabilization.

1 INTRODUCTION

One of the key tasks of modern road construction is to ensure the strength and stability of road foundations, especially in regions with weak, waterlogged or mobile soils. The development of transport infrastructure requires the use of new engineering solutions that ensure the reliability and durability of structures without excessive costs for replacing or excavating unstable soil.

The purpose of this article is to summarize scientific and practical data on the use of geosynthetic materials for strengthening the foundation soils of highways, analyze their effectiveness and develop engineering recommendations.

2 LITERATURE REVIEW

2.1 Geosynthetics as a technology of the 21st century

According to the International Geosynthetics Society (IGS), geosynthetics are synthetic products applied in contact with soil to provide reinforcement, separation, drainage, filtration, protection and insulation. They include a wide range of products: geotextiles, geogrids, geonets, geomembranes, geomaterials and composite systems (Koerner, 2012).

The works of Koerner (2012) and Giroud (2000) lay the foundation for the scientific understanding of the performance of geosynthetics in various engineering settings. Koerner's research is particularly important in the context of classifying geosynthetics by function and structure, while Giroud proposed the first theoretical model for reinforcing road surfaces with geotextiles.

2.2 Increasing the strength of road bases

Numerous studies (Bathurst & Jarrett, 1988; Ling et al., 1998) confirm that the use of reinforcing geogrids and geocells significantly increases the strength and stability of road bases. Due to the redistribution of loads and limitation of transverse deformations, a significant reduction in settlement and an increase in bearing capacity are achieved.

In the works of Zornberg (2002) and Abu-Farsakh et al. (2007) present field studies where reinforced foundations showed a reduction in deformations of up to 40–60% compared to unreinforced ones. Particularly effective geosynthetics on weak clay and peat soils.

2.3 Application in conditions of high groundwater

Water-saturated bases often lead to the destruction of road structures due to floating, undermining or over-wetting of the base. In such conditions, geotextiles and geomembranes, which simultaneously have drainage and filtering functions, have proven highly effective (Holtz et al., 2008; Palmeira, 2009).

In particular, geocomposites with an integrated drainage layer allow for targeted water drainage and simultaneously strengthen the structure. This is especially important in climate regions with seasonal waterlogging. increasing strength characteristics under conditions of seasonal fluctuations in humidity and temperature.

Experiments conducted in a number of countries (USA — Koerner, India — Shukla, Canada — Rowe) have proven the effectiveness of using geosynthetics to strengthen foundations in complex geotechnical conditions. In particular, reinforcement with geogrids increases the strength of the foundation by 30–60% compared to unreinforced systems, and geotextiles effectively cope with filtration and separation of layers in embankments.

3. MATERIALS AND METHODS

Method for determining the modulus of deformation of soil under static loads

To solve the main problems of foundation construction, forecasting the settlement of a structure, it is necessary to determine the deformation modulus.

It is recommended to determine the deformation modulus using data from laboratory and field soil testing methods. The method for laboratory determination of the deformation modulus is shown in GOST 12248-96. The technique for field determination of the deformation modulus is given in GOST 20276-85 "Method for field determination of deformability characteristics ".

When determining the settlement of the foundations of a structure, the deformation modulus is used, as shown in SP 50-101-2004 in expression for static loads from the weight of buildings or structures. The results of the deformation modulus can be subtracted from the empirical results of laboratory tests of soil samples obtained directly from the earth's interior and the results of field tests.

The deformation modulus during stamp tests of foundation soils is calculated in accordance with GOST 20276-2012 using the formula:

$$E=(1-\mu^2)K_p K_1 D \Delta P/\Delta S$$

where, μis the Poisson's ratio: for clays - 0.42; for loams - 0.35; for sandy loams and sands - 0.30; for semi-rocky rocks - 0.25; for rocky rocks - 0.15;

K_p– dependence coefficient 0.7;

K_1– dimensionless coefficient 0.88;

D– stamp width;

ΔP– increase in ground pressure;

ΔS– increase in stamp settlement.

The calculation of the soil deformation modulus is based on the results of laboratory tests with a stamp.

4. RESULTS OF RESEARCH.

Based on the experimental data, a graph of the dependence of the stamp settlement on pressure was constructed (see Fig.). The range of pressures for which the deformation modulus of the model soil E, MPa, will be calculated was determined. This section of the graph should be on the linear section of the graph. Then, the increment of the stamp settlement S is determined for a given pressure range P 1 – P 2.

Thus, the introduction of geosynthetic solutions into the foundation structure allows not only to ensure its strength, but also to significantly extend the service life of the entire structure, reducing operational risks and costs.

Several series of experiments were conducted. The soil was laid in layers and compacted with metal rammers with the same number of blows on one track.

Test No. 1 – without foundation reinforcement. The model was tested with a step-increasing load in the following sequence: 25 kPa, 50 kPa, 75 kPa, 100 kPa, 150 kPa, 200 kPa, 300 kPa. The ultimate settlement of 59.35 mm was achieved at 300 kPa..

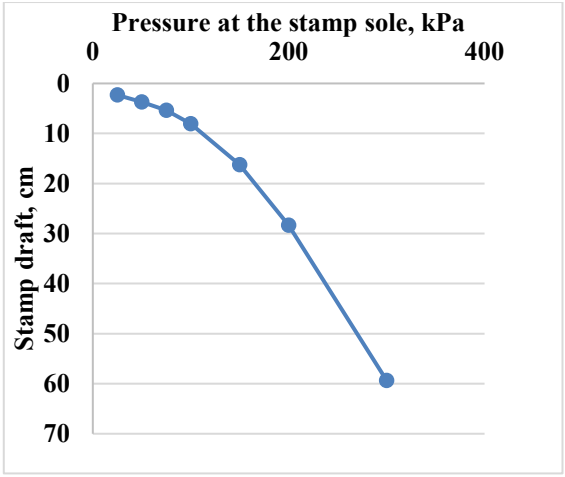


Figure 1 – Graph of the dependence of settlement on the load on test stamp No. 1.

Test No. 2 – reinforcement of the foundation soil with six piles. The model was tested with a step-increasing load in the following sequence: 25 kPa, 50 kPa, 75 kPa, 100 kPa, 150 kPa, 200 kPa, 300 kPa. The ultimate settlement of 51.365 mm was achieved at 300 kPa.

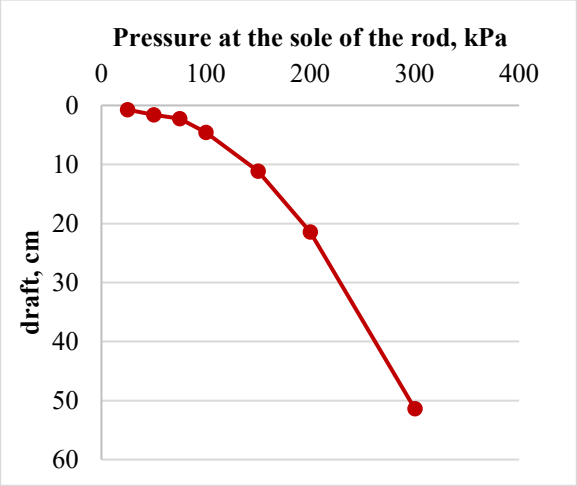


Figure 2 – Graph of the dependence of settlement on the load on test stamp No. 2

5. CONCLUSION

The use of geosynthetics in road construction is an effective, economically justified and technically sound solution for strengthening the foundations of motorways, especially in areas with weak and water-saturated soils. Geosynthetics not only improve the strength characteristics of road structures, but also significantly extend their service life.

The development of calculation methods, improvement of materials and accumulation of practical experience indicate the growing potential of this technology. Future research areas are related to the creation of intelligent (smart) geomaterials and the integration of monitoring systems into road structures.

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Retrofitting Urban Drainage Systems to Mitigate Flooding: A Case Study of Groundwater-Induced Basement Flooding at Semey Emergency Hospital, Kazakhstan

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ABSTRACT

Urban flooding presents a significant challenge in cities worldwide, particularly for buildings constructed without consideration for increased groundwater levels and surface runoff due to climate variability and urban densification. This study focuses on the Emergency Hospital in Semey, Kazakhstan, which suffers from persistent basement flooding caused by high groundwater levels and an underperforming drainage system. A comprehensive assessment of the site's hydrogeological conditions was conducted, revealing that the primary issue lies in the insufficient burial depth of the existing drainage system—up to 0.9 meters above the seasonal groundwater level. Retrofitting strategies are evaluated, emphasizing the integration of Sustainable Urban Drainage Systems (SUDS) such as bioswales and permeable pavements, alongside engineering-based solutions. Closed trench drainage systems are proposed as the optimal approach given the site's proximity to residential areas and critical infrastructure. These systems involve deeper installation (up to 6 meters), artificial groundwater drawdown through well-pumping during construction, and the use of geotextile-wrapped gravel backfill to prevent silting. Hydrogeological modeling (using Ansdimat software) indicates that a redesigned 250-meter drainage line can reduce water inflow from 246.5 m^3/day to 110.8 m^3/day. The proposed solution also includes a ring-trench layout surrounding the facility at a 3-meter distance from the foundation to maximize protection and ensure sustainable operation. This case study underscores the importance of site-specific hydrological modeling, modern drainage technologies, and adaptive retrofitting practices to enhance flood resilience in healthcare infrastructure, especially in hydrogeologically active zones. Recommendations are made for future monitoring, phased implementation, and policy support to ensure long-term functionality.

Keywords: urban flooding, drainage retrofit, groundwater management, sustainable drainage systems (SUDS), Semey hospital

1 INTRODUCTION

Urban flooding poses significant challenges, especially for existing buildings not originally designed to withstand increased surface runoff and rising groundwater levels. Recent research emphasizes the importance of retrofitting urban drainage systems to mitigate flood risks effectively.

The multidisciplinary city hospital "Semey Emergency Hospital", consisting of 9 buildings and designed for 325 beds, was built in 2011-2013 within the framework of the state program "One Hundred Schools, One Hundred Hospitals". The hospital has 14 clinical departments and 8 auxiliary units. Semey Emergency Hospital is located on the north-eastern outskirts of Semey city, administratively part of the East Kazakhstan region. For now, there is an acute issue of flooding of basements with groundwater. The existing drainage system, built during the construction of this hospital for the purpose of water drainage, does not cope with its task, does not function fully. The drainage system is laid along two lines: the first line with a total length of 450 meters is laid along the line of the northern part of the territory with a length of 200 meters and the western one - 250 meters (Fig.1). The second line with a total length of 450 meters is laid along the line of the eastern part of the territory with a length of 250 meters and the southern one with a length

of 200 meters. Both lines are connected into one discharge line located in the southwestern part of the site. Only the second line is partially operational, the first line is not functioning. Fine silty sand is found throughout the site, lying at a depth of 0.9-2.1 meters to 7.8 - 9.3 meters, covered from above by a thin layer of 0.5 to 1.0 meters with moderately decomposed peat. In geological and morphological terms, the site is located on the border of a ribbon pine forest and the second floodplain terrace of the Irtysh River valley. At the border, groundwater discharge occurs from the aquifer of the Glukhovsky groundwater deposit.

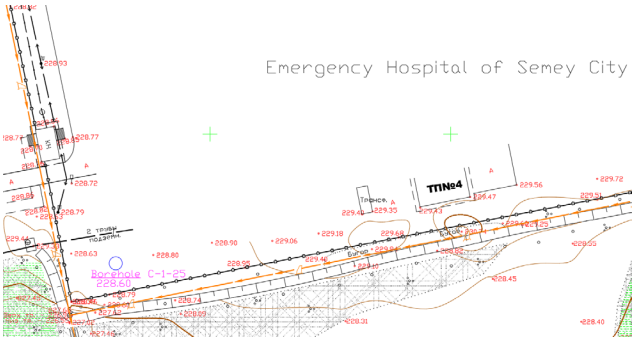


Fig. 1. Topographical Plan section of the Emergency Hospital.

The objective of this study is to reduce flood risks in existing urban areas using the example of Groundwater-Induced Basement Flooding at Emergency Hospital in Semey city in Kazakhstan.

2 METHOD AND RESULTS

Infinite drains and drains of a given length were considered in the modeling using the Ansdimat program (Fig. 3). Calculation of the inflow into a horizontal drain located in the roof of a gravity-fed aquifer. The possibility of considering the difference in the radius of influence to the left and right of the drain.

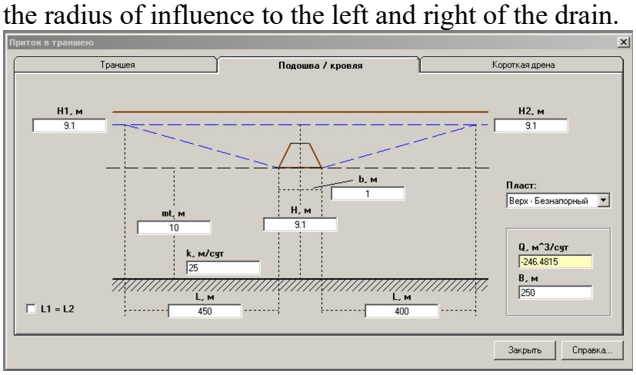


Fig. 3. Modeling of total water inflow under initial specified characteristics of soil and hydrogeological conditions.

Selection of groundwater regime and drain position during modeling: the drain is in the upper part of the unconfined aquifer section. The constant pressure values at the left "H1, m" and right "H2, m" boundaries of the unconfined aquifer are 9.1 m and are measured from the base of the aquifer. The information field "Q, m³/day" for determining the total inflow into the drain per drain length B is calculated using formula (1) with equal influence radii.

$$Q = kB\left[\frac{s^2}{R} + \frac{\pi s}{ln\frac{2m_t}{\pi b}+\frac{\pi R}{2m_t}}\right] \quad (1)$$

Q - discharge (flow rate), m³/day; k - hydraulic conductivity, m/day; B - width or thickness of flow section, m; s - drawdown or depth of flow, m; R - drainage radius, m; m_t - distance to impermeable boundary, m; b - radius of the drain, m.

According to formula (1), the total water inflow was calculated as Q=246.4815 m³/day with the initial specified characteristics of soil and hydrogeological conditions. Designing a drainage with a length of B=250 m and a drain width of b=1 m reduces the expected water inflow to Q=110.805 m³/day (Fig. 2).

3 CONCLUSIONS

A comprehensive assessment of the hydrogeological conditions of basements of the Emergency Hospital of Semey city showed that the main problem is the insufficient depth of the existing drainage system. To solve the issue of flooding it is necessary to deepen the existing drainage system to a depth of 3.0-3.5 meters from the ground surface. According to calculations, the load from water inflow is reduced by 2 times, more accurate data can be obtained after field studies, such as drilling, experimental filtration work, determining the properties of the soil. To solve the problem of driving canals for drainage pipes, it is necessary to organize an artificial lowering of the groundwater level during construction work by organizing group pumping from hydrogeological production wells drilled for this task. Production wells must be laid along the lines of the drainage system. The total number of production wells and their depth to be determined after studying the hydrogeological conditions of the site by means of drilling, experimental filtration work, testing, laboratory work and engineering and geological testing of the soils under study.

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Main risk factors in reinforcement of soils by "Geocomposite" method

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ABSTRACT

Design and construction in complex engineering and geological conditions, on weak soils or in dense urban development increasingly forces engineers and planners to apply new and modern methods to improve soils and increase the stability of facilities. With today's demands on the stability and safety of buildings, the need for additional measures to improve the physical and mechanical properties of soils is becoming increasingly evident.
Such methods include injection reinforcement, compaction and replacement of weak soil layer. In this paper the greatest attention is paid to one of the modern methods of soil reinforcement by hydraulic fracturing or as it is also called Geocomposite method [1].
This method is based on additional soil fracturing and filling the resulting cracks with pressurized mortar. This complex process is accompanied by a large number of risks, which in turn can lead to improper consolidation and failure to set the required physical and mechanical characteristics of soils.
The article identifies the main risk factors on the basis of examples of fixed objects

Keywords: soil fixation, injection, risk factors, risks, Geocomposite method

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INTRODUCTION

Increasingly, during design and construction, engineers are faced with the need for additional measures to improve the properties of soils. This is due to the fact that nowadays construction is carried out on weak soils and on sites with complex engineering and geological conditions [2,3]. Due to these factors, construction without additional measures is not possible.

Therefore, during construction, additional measures to improve the physical and mechanical properties of soils are often resorted to, such as injection reinforcement with the introduction of additional solutions into the foundation soils, soil compaction, replacement of weak soil layer, etc.

Often already in the process of construction it is found that the settlement of the structure under construction begins to exceed the standard or deformations of the surrounding buildings begin to appear. In these cases the method of soil reinforcement with the help of Geocomposite or as it is also called hydraulic fracturing method is applied now.

The essence of the method is the injection reinforcement of soils with the creation of additional cracks in the soil mass, into which the solution penetrates. The solution penetrates not only into the existing voids, but due to the pressure of up to 20 atmospheres creates additional fractures, which leads to

an increase in the strength and stability of the soil and additional compaction of the soil around the solution (Fig. 1).

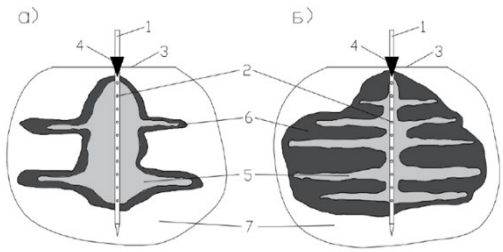


Fig. 1 Distribution of mortar when reinforcing soils by Geocomposite method: a) sandy or sandy loam soil; b) clayey or any heterogeneous soil. 1 - injector; 2 - nozzles; 3 - concrete base; 4 - parker; 5 - cement stone; 6 - compacted soil; 7 - reinforced massif.

Geocomposite method can be applied in various spheres of geotechnics: reinforcement of soils, reinforcement of bases of existing buildings and structures, stabilization of slopes and slopes, elimination of ground subsidence, etc. This article considers objects reinforced in the process of construction [4,5].

METHODOLOGY

The authors considered 71 objects on the territory of Moscow and Moscow region as the object of the study. As stated earlier soil reinforcement at these objects was carried out during construction.

Soil reinforcement was carried out at different depths and for different soils.

As soils subjected to reinforcement were considered:

- bulk soils
- sands
- loams
- sand loam
- peat
- peat.

The best results in the process of reinforcement were shown by embankments and peats, the reinforcement of soils on average reached 300% [6]. The deformation modulus E was taken as the main characteristic considered. The increase of deformation modulus for dusty-clay soils and sands increased on average by 2 times (Fig.2).

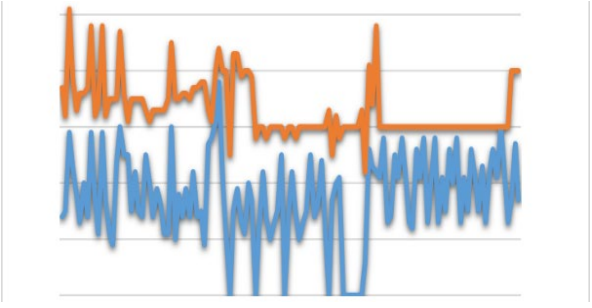


Fig. 2 Variation of soil deformation modulus E before and after reinforcement

RESULTS

In addition to the positive aspects of reinforcing soils by the method of hydraulic fracturing, there are also negative aspects [7].

Among the advantages of soil reinforcement by Geocomposite method are:

- Possibility to reinforce soils to the required depth;
- change of physical and mechanical characteristics of soils;
- Reduction of water permeability of soils;
- The possibility of reducing the roll of already existing buildings;
- minimal negative impact on the environment;
- possibility to control the injection process by means of pressure control, speed of solution supply, etc.;
- wide range of applied mortars;
- cost-effectiveness.

Geocomposite reinforcement of soils is a complex task that requires attention to a very large number of details [8]. Any assumption in soil reinforcement can lead to insufficient reinforcement, which in turn leads to the inability to correct the settlement of buildings and structures [9].

On the basis of the considered objects 23 main risk factors in soil reinforcement were identified and the consequences to which their manifestation can lead were determined (Table 1).

Table 1. Determination of possible risks and consequences of their manifestation at reinforcement of soils by Geocomposite method

№	Risks	Consequences
1	Incorrect ground preparation	Insufficient anchorage zone resulting in voids, material breaks, which will reduce reinforcement
2	Insufficient consolidation depth	Improperly calculated reinforcement depth. This factor will cause damage to the ground during mechanical impact or freezing of the ground
3	Incorrect calculation and design	Errors in calculation and design resulting in incorrect selection of the type of geocomposite, its geometric parameters (size, spacing, number of layers) and insufficient reinforcement.
4	Uneven compaction (expansion) of cracks	Insufficient or uneven distribution of cracks through which the mortar passes leads to uneven distribution of load and reduction of bearing capacity of the soil
5	Damage to the reinforced zone during installation	Damage to the area reinforced with geocomposite will reduce the strength and durability of the consolidation zone

6	Influence of groundwater	Failure to take groundwater into account during design and installation can lead to mortar sinking
7	Lack of quality control	Lack of or inadequate quality control at all stages of work execution, which can lead to non-compliance with technology, use of poor quality materials and reduced reinforcement efficiency
8	Incorrect installation of the injector	Causes solution to come to the surface, not anchoring the zone
9	Lack of test sites	Lack of test sites can lead to failure to identify hidden factors not taken into account in the design
10	Unaccounted for factors	Unaccounted for factors such as seismic activity, dynamic loads, or other impacts that could lead to premature failure of the geocomposite and reinforced soil
11	Improper use of geocomposite	Misuse of the geocomposite, such as for tasks for which it is not intended, which may result in ineffective reinforcement.
12	Inadequate geologic investigations	Inadequate scope or quality of geologic investigation resulting in inaccurate determination of soil characteristics, water table and geologic disturbances. This can lead to incorrect fracturing parameters and ineffective reinforcement
13	Incorrect choice of solution	Improper choice of injection mortar composition (cement, bentonite, resins) resulting in low penetration, rapid setting or incompatibility with the soil. This can lead to incomplete fracture filling and ineffective reinforcement.
14	Incorrect choice of pressure	Incorrect injection pressure resulting in either insufficient crack opening (low pressure) or failure of the soil structure and uncontrolled spreading of the mortar (high pressure).
15	Incorrect spacing between boreholes	Incorrect spacing of injection wells resulting in incomplete coverage of the soil mass and reduced reinforcement efficiency. Distances that are too large can leave unreinforced zones and distances that are too small can lead to overpressure and soil failure.
16	Incorrect injection sequence	Incorrect injection sequence resulting in voids or blocked cracks, preventing further spreading of the mortar.
17	Inadequate supervision of the injection process	Lack of or inadequate control of injection volume, pressure and rate, resulting in uncontrolled spread of mortar and ineffective reinforcement.
18	Exit of mortar to the surface:	Injection solution escaping to the surface or nearby utilities due to high pressure or the presence of cracks and voids in the ground. This can lead to environmental contamination and damage to utilities.
19	Void formation	The formation of voids in the ground due to the washout of fine particles during injection, especially in sandy soils. This may result in reduced bearing capacity and subsidence.
20	Blocking cracks	Blocking of cracks in the ground due to the use of too thick mortar or the presence of coarse particles in the mortar, which prevents further spreading of the mortar
21	Lack of control over reinforcement results:	Lack of or inadequate monitoring of reinforcement results (e.g., through geophysical methods), which prevents the effectiveness of the work performed from being evaluated and corrective action from being taken in a timely manner
22	Improper selection of equipment	Use of faulty or inappropriate injection equipment, which can lead to inaccurate solution dosing, pressure surges, and other problems.
23	Insufficient qualification of personnel	Inadequate qualification of personnel performing hydraulic fracturing operations, which can lead to technology failure and reduced reinforcement efficiency.

During the analysis of objects the identified risks are presented in the form of a graph (Fig. 3). On the horizontal the numbers of risks are written by numbers, on the vertical the number of main violations in case of their manifestation [10].

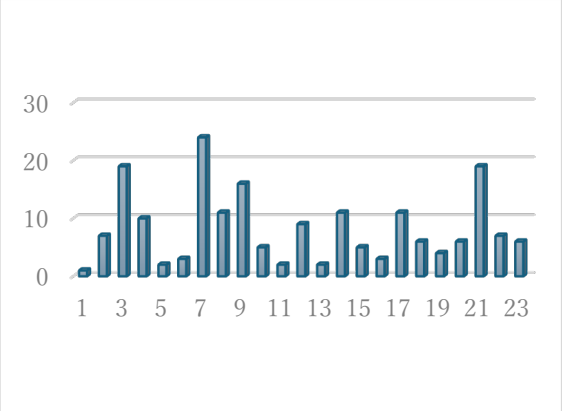


Fig. 3 Typical risks when reinforcing soils with Geocomposite

CONCLUSIONS

The conducted research on the basis of 71 objects in Moscow and the region showed that reinforcement of soils with the help of Geocomposite method is an effective way to improve physical and mechanical characteristics, which proved itself especially well in bulk soils and peats, where the change of deformation modulus increased by 3 times and more. For dusty clayey soils and sands, the strain modulus increased on average by a factor of 2.

However, despite the positive results it was found that the hydraulic fracturing method has advantages as well as disadvantages. The geocomposite method, although effective, requires strict adherence to

technology and consideration of numerous risk factors [11]. Based on the analysis of the reviewed sites, 23 major risk factors were identified, which emphasizes the need for careful planning and control when performing soil reinforcement works to ensure the reliability and durability of buildings and structures. The obtained results allow us to conclude the significance of the correct choice of the reinforcement method and strict compliance with the technology to achieve the desired results and prevent negative consequences.

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Geotechnical analysis of the causes of dam failure in Central Kazakhstan and recommendations for its rehabilitation

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ABSTRACT

The collapse of hydraulic structures poses a significant threat to the environment, the local population and infrastructure. This paper examines the emergency condition of the Togyzkudyk dam in Karaganda oblast, built in 1954. Engineering and geological investigations revealed critical structural defects, including slope collapse, crest deformation and spillway failure. These defects were caused by many years of non-repair, active water seepage processes through the dam body and seasonal changes in hydrogeological conditions.
Based on an analysis of the condition of the dam and the geotechnical characteristics of the soils, consideration is given to possible methods of rehabilitation and reinforcement of the structure, including replacement of failed spillway elements, reinforcement of the soil mass and installation of modern monitoring systems. In addition, special attention is given to measures to reduce flood risks and prevent repeated failures.
The results obtained can be used to develop strategies for the reconstruction of hydrotechnical facilities that are in a state of disrepair, as well as to improve the reliability of dams in regions with similar geological conditions.

Keywords: dam, slope, geotechnical engineering, failure

1 INTRODUCTION

This study presents the results of a geotechnical investigation into the failure of the Toguzkuduk dam, located in the Bukhar-Zhyrau district of the Karaganda region, Central Kazakhstan. The Toguzkuduk dam, originally constructed in 1954 as an earth-fill embankment, suffered complete structural failure during the spring floods, leading to uncontrolled water discharge and environmental risks. The dam was initially designed for seasonal regulation to provide livestock watering, but due to the lack of maintenance and severe deterioration, it has not been operational for several years.

2 TECHNICAL CHARACTERISTICS AND OBSERVATIONS

The Toguzkuduk dam is classified as a homogeneous earth-fill embankment, approximately 338 meters long, with a maximum height of 6.0 meters and a crest width of 6.5 meters, suitable for a service road of category III-c. The dam features reinforced slopes:

Upstream slope:

1:3 (protected with stone riprap to resist wave impact);

Downstream slope:

1:2 (reinforced with vegetation layer).

A field inspection conducted in 2022 revealed the following critical conditions:

- severe deformation and erosion of both upstream and downstream slopes;
- active filtration through the dam body along its entire length;
- a significant breach (erosion channel) of approximately 15–20 meters;
- full destruction of the concrete-ring spillway system located within the dam body.

The catastrophic failure during spring floods resulted from the combination of structural degradation, lack of spillway functionality, and inadequate embankment stability under hydraulic loads. The dam's condition is classified as emergency, posing risks to nearby rural settlements and ecological safety [1-4].

3 REHABILITATION AND DESIGN RECOMMENDATIONS

A comprehensive rehabilitation project was developed to restore the functionality and safety of the dam, considering modern engineering standards and

site-specific geotechnical conditions. The project includes:

- reconstruction of the dam body using compacted clayey soils with a target dry density of 1.69 t/m³;

-upstream slope reinforcement with riprap stones (fraction 100–300 mm, minimum stone grade M600) and filter layers to prevent erosion and reduce seepage;

-complete replacement of the destroyed spillway with a modern KVAT-4M tubular spillway with a discharge capacity of up to 43.0 m³/s, designed for extreme flood events;

-construction of a controlled outlet structure (pipe outlet) to regulate ecological releases and reservoir water levels;

-installation of safety features, including concrete signal posts along the crest road.

The design complies with national standards for Class III hydraulic structures and considers hydrological risks such as extreme wind speeds (25.0 m/s), snow load zones (3.0 kPa), and potential flood scenarios based on a 0.5% exceedance probability.

The rehabilitation aims to enhance dam safety, ensure controlled water management, support agricultural needs, and mitigate the risk of future failures. Construction is scheduled to commence in May 2025 with an estimated duration of 4.5 months.

4 CONCLUSIONS

The failure of the Toguzkuduk dam highlights the critical need for regular maintenance, geotechnical monitoring, and timely reconstruction of aging hydraulic structures in Central Kazakhstan. The proposed rehabilitation plan incorporates both traditional and modern engineering approaches to ensure long-term stability and operational reliability of the dam under regional climatic and hydrological conditions.

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Geotechnical support for mechanical safety of underground water supply structures taking into account complex soil conditions and extraordinary impacts

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ABSTRACT

The purpose of the study is the geotechnical substantiation of methods for the protection of underground aquifers operated for a long time in difficult engineering and geological conditions, taking into account the combined effects of external forces and internal environmental influences. Taking into account the specifics of the set goal and the complexity of engineering and geological tasks, their achievement is proposed to be solved by modeling the interaction of underground aquifers with the host soil mass under the combined influence of external forceful static and dynamic influences. A computational justification is given and methods for strengthening and protecting tunnel structures under various combinations of extraordinary force and dynamic effects of train loading are presented. In order to take into account the specifics of the operation of sewer tunnels in difficult ground conditions, identified as a result of extensive engineering surveys, including instrumental, hydraulic and engineering-geological surveys, modeling of the interaction of underground aquifers with the host soil was performed based on a set of software tools: geotechnical (Plaxis), calculation programs (Robot). During the simulation, the calculated situations were considered, taking into account the most unfavorable tunnel operating conditions. As a result of modeling design situations that take into account the most unfavorable conditions that may arise when exposed to force and dynamic loads on linear underground structures. Based on the modeling results, an integrated approach to the choice of methods for their protection is proposed and, if necessary, a calculated justification for their constructive reinforcement is given. The implementation of the proposed methods for strengthening and protecting underground aquifers has been successfully tested during the reconstruction of Russian Railways station facilities.

Keywords: underground structures, roadbed, train load, combination of impacts, protection methods

1 CHARACTERISTICS OF THE RESEARCH OBJECT

With a multi-track roadbed, the specifics of the stress-strain state (VAT) of the base, composed of unstable soils under conditions of vibro-dynamic impact from a moving load in conditions of oncoming train traffic, are manifested. At the same time, experimental studies have recorded a 1.5- to 2-fold increase in the horizontal component of the amplitude of vibrations of the soil of the earthbed in the inter-water zone at a depth of up to 2.5 m from the level of the main site and in the body of the water-carrying underground structure [1]. Since the soils of the roadbed are quite homogeneous and have a high degree of distributivity due to the natural geotechnical granularity, which allows for the redistribution of stresses in the geomass, the fact of the imposition of a vibro-dynamic effect during oncoming train traffic does not cause a violation of the strength and deformability of the roadbe [2]. The paper considers the picture of the formation of a stress-strain state in the zone of interaction between the base of the earth bed and the structural elements in contact with it, with values of elastic deformations differing by several orders of magnitude (from 25MPa to 150MPa for embankment soils and from 17000MPa to 32000MPa for concrete joints and inclusions). The interaction of water-carrying engineering communications, which are in long-term operation and have areas with a high degree of deterioration, refers to concrete structures of artificial structures, for example, bridge abutments with the base of unstable soils of a multi-track roadbed have not been considered before. The task is to provide a calculated justification for the methods of protecting water-carrying communications at the base of the roadbed during the reconstruction of the railway track, taking into account its long-term operation and changes in the magnitude and combination of loads. This goal is achieved by studying the specifics of the work of the research object based on modeling the interaction of water-carrying communications with the host soil mass at the base of the roadbed in the

conditions of railway track reconstruction, taking into account changes in the combination of force effects. Since the problem of protecting underground tunnel sewer structures under external influences was solved earlier by us using structural reinforcement methods [3], which are quite complex and unique in design, in this work an attempt will be made to combine the most traditional protection methods for railway construction conditions.

The implementation of reconstruction measures is associated with the exhaustion of the capacity and carrying capacity of the existing railway tracks. On the St. Petersburg Baltiyskiy - Bronevaya section, the construction of the III and IV main tracks is planned in accordance with the prospect of developing the railway hub. The project provides for an increase in passenger traffic, train traffic, as well as an increase in travel speeds up to 200 km/h and axial load up to 274 kN/axle. The projected flow is shown in table 1.

Table 1. Projected passenger traffic at St. Petersburg Balti statio

	2025 year	2030 year	2035
Total per year, thousand pass.	17 236,0	14 834,7	16 757,3

2 GEOTECHNICAL MODELING OF THE RESEARCH OBJECT

The construction of the III and IV main tracks, which are part of the complex of works for the reconstruction of the existing railway, as well as an increase in passenger traffic, train traffic, travel speeds up to 200 km/h and axial load up to 274 kN/axis (at Kd = 1.25), form a new picture and specifics of the stress-strain state (VAT) of the base, composed of unstable The soils of their interaction with the water-carrying structure located at the base of the geomass are TCC (Fig.1, Fig. 2).

a b

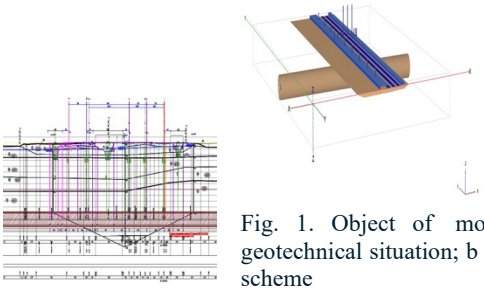


Fig. 1. Object of modeling: a - geotechnical situation; b - calculation scheme

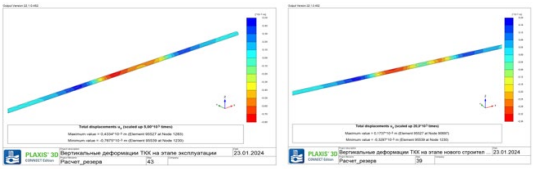


Fig. 2. Isofields of additional vertical deformations of the TCC at the stage of reconstruction and operation: a,b, respectively, without protective measures and with the use of a protective composite layer

3. CONCLUSIONS

An analysis of the calculation results shown in Fig.2a, b shows that the initial deformations and stresses without protective measures were 2.95 MPa, and the vertical deformations of the TSS were 0.8 mm. During the reconstruction work, the maximum stresses are 1.46 MPa, respectively, and the deformations of the TSS are 0.3 mm. The stresses are concentrated in the tray part of the collector section. It is quite obvious that this model, equipped with a generator, is made of an M400 batna (class B30) with an estimated compression speed of Rb = 17 MPa, which significantly dominates the voltage in the shell. According to the initial data, the collector cladding is made of M400 grade concrete (class B30). The estimated compressive velocity of concrete is Rb = 17 MPa, which means a large stress in the circumference. It is shown that the obtained results of computational modeling of the nature of the force impact and changes in VAT of a water-carrying structure at the base of the roadbed during the reconstruction of the railway make it possible to determine under which combinations of force effects and in which sections of underground utilities assume the risk of loss of bearing capacity of structures and require a calculated justification for protective measures.

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A study of ground vibration characteristics of Sakuradani tomb damaged during the 2024 Noto peninsula earthquake using microtremor measurements

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ABSTRACT

In the 2024 Noto Peninsula Earthquake, significant damage including building collapses, fires, landslides, and ground deformation due to liquefaction were caused, especially in Ishikawa and Toyama Prefectures. Collapse and some large displacement/deformation appear in not only the cultural stone walls in Kanazawa Castle, Nanao Castle as well as Maeda Toshinaga Masoleum, but also some large subsidence and collapse of slopes were observed in the Sakuradani Tomb, located in Takaoka City, Toyama Prefecture. The affected tombs were the 1st (62 m in length) and the 2nd Tombs (50 m in length) of Sakuradani Tombs are believed to be constructed as earthen mounds. The earthquake-induced damage to the tombs will be significantly related to the ground conditions and especially, the relation between the vibration characteristics and strong seismic motions will play significant roles. Although many microtremor measurements have been conducted and reported in previous studies, very few studies focus on the ground vibration characteristics of tombs constructed as earthen mounds.
Therefore, in this paper, Microtremor measurements on the ground at Sakuradani Tombs were conducted to investigate the ground vibration characteristics using the shape of the H/V spectra and peak frequencies. Also, the ground vibration characteristics of the ground with strong seismic motions were compared between the 1st and 2nd Tombes, in order to investigate the effect of the ground vibration characteristics on the earthquake-induced damage such as subsidence and collapse.

Keywords: Sakuradani Tombs, earthen mounds, earthquake-induced damage, microtremor measurement, H/V spectrum

1 INTRODUCTION

In the 2024 Noto Peninsula Earthquake, significant damage including building collapses, fires, landslides, and ground deformation due to liquefaction were caused, especially in Ishikawa and Toyama Prefectures. Collapse and some large displacement/deformation appear in not only the cultural stone walls in Kanazawa Castle, Nanao Castle as well as Maeda Toshinaga Masoleum, but also some large subsidence and collapse of slopes were observed in the Sakuradani Tomb, located in Takaoka City, Toyama Prefecture. The affected tombs were the 1st (62 m in length) and the 2nd Tombs (50 m in length) of Sakuradani Tombs are believed to be constructed as earthen mounds. The earthquake-induced damage to the tombs will be significantly related to the ground conditions and especially, the relation between the vibration characteristics and strong seismic motions will play significant roles. Although many microtremor measurements have been conducted and reported in previous studies, very few studies focus on the ground vibration characteristics of tombs constructed as earthen mounds.

Therefore, in this paper, Microtremor measurements on the ground at Sakuradani Tombs were conducted to investigate the ground vibration characteristics using the shape of the H/V spectra and peak frequencies. Also, the ground vibration characteristics of the ground with strong seismic motions were compared between the 1st and 2nd Tombes, in order to investigate the effect of the ground vibration characteristics on the earthquake-induced damage such as subsidence and collapse.

2 DAMAGE TO SAKURADANI TOMB FROM THE 2024 NOTO PENINSULA EARTHQUAKE

During the "2024 Noto Peninsula Earthquake," Takaoka City was struck by a tremor of seismic intensity 5 upper, and the Sakuradani Tomb, a historic site and natural monument, was affected. The Sakuradani Tomb is a group of 13 burial mounds from

the 4th century. Figure.1 shows the damaged areas of Tomb 1, and Figure.2 shows the damaged areas of Tomb 2.[1) 2)] The red circles indicate the sinkholes, and the areas enclosed by red lines indicate the cracks. As a result of the earthquake, significant damage occurred at Sakuradani Tomb. At Tomb 1, a crack extending 400 cm and a sinkhole 50 cm wide were observed. Additionally, at Tomb 2, a sinkhole 120 cm wide was found.

3 ANALYSES OF FACTORS USING MICROTREMOR MEASUREMENT FOR DAMAGE AT SAKURADANI TOMB

Subsidence damage at Tomb 1 occurred near points 2 and 3, but no correlation was found between the peak values of acceleration Fourier spectra from strong ground motion and the peak frequencies of the H/V spectra from microtremor measurements. Since the subsidence damage at Tomb 1 was minor, it is considered to have been influenced by factors other than the ground vibration characteristics.

On the other hand, crack damage at Tomb 1 occurred near point 4, where the peak frequencies of the acceleration Fourier spectra of strong ground motion in the vertical and north-south directions matched the 4 Hz peak frequency in the microtremor H/V spectra. Furthermore, this 4 Hz frequency corresponds to the peak frequency of the surface ground H/V spectra at Tomb 1. Therefore, it is considered that seismic motion caused a strong resonance effect by affecting both the surface ground and the embankment, resulting in the observed damage.

At Tomb 2, subsidence damage occurred from around point 7 to point 9. Near point 8, closest to the subsidence damage, the peak frequencies of the acceleration Fourier spectra of strong ground motion in the vertical and north-south directions matched the 4 Hz peak frequency of the microtremor H/V spectra. This peak frequency also matches those at points 1 and 2 at the lowest part of the mound. Similar to Tomb 1, it is considered that resonance phenomena occurred due to the match between the ground vibration characteristics of the mound and the surface ground with the acceleration Fourier spectra of the strong ground motion, which influenced the occurrence of large-scale subsidence damage.

4 CONCLUSIONS

This study targeted Tomb 1 and Tomb 2 of Sakuradani Tomb, analyzing factors of earthquake-induced damage such as subsidence and ground cracks by comparing microtremor measurement results with strong ground motion data. The results suggest that the relationship between the acceleration Fourier spectra of strong ground motion and the H/V spectra of the surface ground and mound areas may be related to the occurrence of earthquake damage such as subsidence and cracks. Future studies should conduct comprehensive analyses using other investigations such as radar surveys and soil testing to examine the factors behind the damage in greater detail.

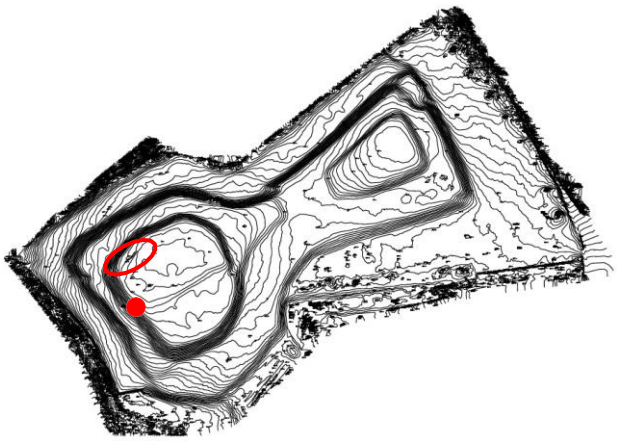


Fig. 1. The damaged areas of Tomb 1.

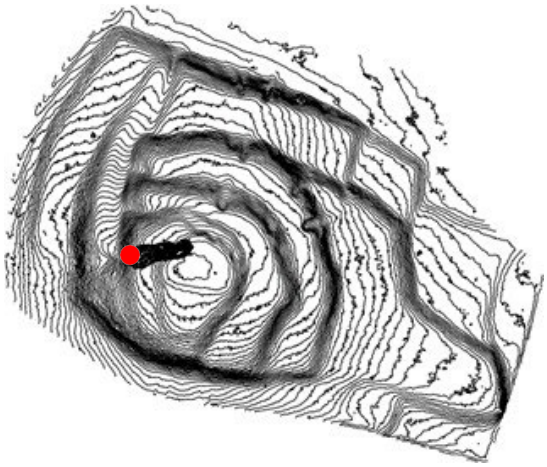


Fig. 2. The damaged areas of Tomb 2.

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Numerical study of bearing capacity of composite stone column

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ABSTRACT

In weak clay soils, the effectiveness of the improvement technique using stone columns may be limited due to insufficient lateral pressure on the soil. The stone columns need to be reinforced in such a way as to provide the minimum required lateral confining pressure. Plaxis 2D 20 software package was used to verify the numerical analysis. The numerical model was validated based on previous studies and the same material properties and boundary conditions were selected for the analysis as in the experimental study carried out by A. P. Ambili and S. R. Gandhi. After confirming the validity of the model, four variants of concrete and stone composite column were considered keeping the same material properties and boundary conditions. The objective of this study is to investigate and evaluate the effect of the concrete portion on the behavior of concrete-crushed stone composite column compared to conventional stone column. This is accomplished by comparing experimental and numerical results, including the following objectives:

1. Compare the results of the conventional stone column study with the previous experimental study to confirm their consistency and accuracy;
2. Modify the form of the numerical simulation while maintaining all established material properties and boundary conditions. This is necessary to be sure that changes in the model do not affect the key parameters of the study.

The paper presents the results of numerical studies of the bearing capacity of stone composite column with increasing concrete part between 0.5 and 2.0 column diameters. The results showed that the vertical stress increases with increasing the length of the concrete part. The concrete part increases the stiffness of the crushed stone column, which in turn increases its bearing capacity. When the length of the concrete part approached half the length of the crushed stone column, the stress doubled to 2000 kPa at 30 mm settlement. The maximum bulging of the crushed stone column occurs at a depth equal to approximately one column diameter, which proves that the horizontal deformation decreases with increasing length of the concrete part of the column.

The significance of the results obtained for the construction industry is that the concept of composite concrete-crushed stone columns successfully combines the advantages of various conventional crushed stone columns. These piles have a high bearing capacity superior to that of conventional crushed stone columns, and this is achieved at a competitive rate. In addition, composite concrete-crushed stone columns exhibit better bearing capacity compared to bored columns of similar diameter and length. In addition to these advantages, composite concrete-crushed stone columns provide significant resistance to bulging failure. The level of deformation associated with bulging decreases as the length of the concrete portion increases. Also, the stiffness of the concrete-crushed stone composite columns increases if the concrete portion is located close to the ground surface. In addition, the characteristics of the crushed stone column undergoes changes due to the presence of the concrete component.

Keywords: soft soil, settlement, bearing capacity, composite concrete-crushed stone column.

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Load-bearing capacity of span structures of transport facilities under dynamic effects

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ABSTRACT

Many existing bridge structures in Kazakhstan are outdated and require a thorough study of their bearing capacity. One of the reasons for the loss of bearing capacity is the ever-increasing traffic intensity and high load capacity. Until now, the dynamic impact of vehicles on bridges in regulatory documents has been taken into account based on the generalization of experimental data. Methods for studying the behavior of complex structures (bridges, overpasses, flyovers), which include a large number of different types of elements, under the influence of a moving load are not yet sufficiently developed. Superstructures of modern transport structures are a complex system, and their operation raises a number of problems caused by increasing requirements for their safety and reliability.

The article presents an algorithm for the dynamic calculation of load-bearing beams and rigid pavement slabs using analytical and numerical methods. Two models of a road overpass are considered:
Model 1: calculation scheme of the overpass - split beams loaded with a constant load from the weight of the superstructure slab and a temporary load from moving vehicles. Model 2: calculation scheme of road pavement - rigid ribbed slabs of pavements, loaded with a load from moving transport. The dynamics of beam spans at various conjugations with supporting structures and ribbed slabs of pavements is considered. The results of analytical and numerical solutions are presented on the example of a four-span road overpass. In the calculation, the iteration method was used to solve transcendental equations. In order to avoid resonance, the frequencies of free transverse vibrations with and without taking into account the resistance forces are determined, comparisons with experimental measurements are given. Under the action of external harmonic load caused by traffic, the maximum internal forces, stresses and deformations are determined. An assessment of the bearing capacity of the overpass and corresponding recommendations and forecasts are given.
Keywords: dynamics, load-bearing beams, rigid slabs, overpass, mathematical models, calculation schemes, vibrations, frequencies, dynamic coefficient.

1 INTRODUCTION

The design of transport facilities in terms of strength, reliability and stability must meet current modern technical requirements and ensure the passage of modern loads at established speeds. Assessing the condition of beam bridges requires a detailed examination of the patterns of the oscillatory process of span structures under the influence of moving loads and identifying parameters that objectively reflect the degree of the bridge structure's ability to withstand real dynamic impact, and also allow identifying defects and damage that reduce the load-bearing capacity of the structure. The study of the dynamic impact of vehicles is especially important for road bridges with increased deformability. Many existing road overpasses in Kazakhstan are outdated and require a thorough study of their bearing capacity. The study of the dynamic impact of currently

existing types of moving loads on bridge structures is becoming relevant and requires improvement of the theory and methods of calculation. There is an urgent need to create modern methods of dynamic analysis of bridge structures based on modern computer technologies, allowing to correctly take into account the most important features of the interaction of existing and prospective moving loads with various types of bridge structures. However, performing the analysis of the dynamic behavior of the oscillatory system "highway bridge - moving load" even with the use of modern software packages is extremely difficult due to the need for constant adjustment of the calculation model.

2 SETTING RESEARCH OBJECTIVES

The aim of the study is to improve the dynamic calculation transport structures, identifying the main factors for enhancing their bearing capacity and operational safety.

The results of dynamic tests were carried out using a set of instrumental measurements to examine the actual condition of the overpass. Comparisons of the obtained theoretical data with experimental data were made.

3 CALCULATION SCHEME OF THE OVERPASS

The paper presents a dynamic calculation of beam structures and road surfaces of a motorway overpass using analytical and numerical methods [4,5]. A mathematical model has been developed for the adopted calculation scheme and the results of numerical and analytical calculations are presented. Two overpass models are considered.

Model 1: calculation scheme of the overpass - split beams loaded with a constant load from the weight of the span structure slab and a temporary load from moving vehicles.

Model 2: calculation scheme of road surface - rigid ribbed pavement slabs loaded with traffic.

In progress the frequencies of natural transverse vibrations, dynamic and static forces and displacements for various types of boundary conditions were determined, and an assessment of the stress-strain state of the overpass was made.

4 MATHEMATICAL MODELS OF OVERPASS

Model 1 considers the span structure as a system with an infinite number of degrees of freedom. The differential equation for transverse vibrations of a beam with distributed mass without taking into account the resistance forces and its solution have the form:

$$EI\frac{\partial^4 y}{\partial x^4}+m(x)\frac{\partial^2 y}{\partial t^2}=0 \qquad (1)$$

In (1) y is the deflection, EI is the rigidity of the section of the supporting beams in bending, m(x) is the linear mass.

Model 2 considers the roadway of the overpass as a rigid ribbed slab.

Longitudinal and transverse beams are considered as stiffeners. The technical theory of rigid plates is used for calculation. Let's considerfree vibrations of a rectangular plate. The figure shows the principal stresses. According to the technical theory of plates, the differential equation of equilibrium of the curved middle surface of the plate is as follows:

$$\frac{\partial^4}{\partial x^4}w+2\frac{\partial^4}{\partial x^2\partial y^2}w+\frac{\partial^4}{\partial y^4}w=\frac{q}{D} \qquad (2)$$

In (2) the following designations are used: q – intensity of external normal load; W – normal displacements or deflections of points of the midplane; D – cylindrical rigidity during plate bending.

5 RESULTS OF DYNAMIC TESTS AND ANALYTICAL CALCULATIONS

The compressive strength of concrete is 16.5 MPa, bending strength is 2.05 MPa, and for reinforcement, tensile strength is 150 MPa. The results for internal forces and deformations for span beams do not comply with the operating standards for bearing capacity; for reinforcement, it is necessary to replace the span beams with those with the required load capacity to pass current and future loads. Due to high tensile forces, cracks appear in the roadway slabs, affecting the load capacity, durability of the structure, and do not fully ensure the safety of traffic.

Taking longitudinal vibrations into account results in a slight increase in internal forces and deformations. Their consideration is necessary when considering seismic impacts.

6 CONCLUSIONS

The reserve of the bearing capacity of transport structures directly depends on their dynamic characteristics; therefore, it is necessary to clarify the calculation models, calculation methods and experimental measurements. For the overpass under consideration, the calculation results showed that the internal forces and deformations for the beams of the span structures do not correspond to the operating standards for bearing capacity, since they do not have the necessary load capacity to pass modern and future loads. In the roadway slabs, due to large tensile forces, cracks may appear in the longitudinal and transverse directions, affecting the load capacity of the entire structure and the stability of their equilibrium state. Based on the results of the theoretical studies, it was found that the dynamic response of the span structure to the passage of trucks

depends significantly on the speed of the vehicle and the smoothness of the roadway on the bridge. With an increase in the speed of the vehicle, the dynamic coefficient increases, this is due to an increase in inertial forces.

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Evaluating the Effect of Groundwater Table Variations on the Seismic Resilience of Residential Buildings in Almaty City

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ABSTRACT

The seismic resistance of residential buildings is based not only on materials and design but also on the soil conditions. In Almaty City, Kazakhstan, where earthquakes are a real concern, the seasonal rise and fall of groundwater levels can significantly impact how buildings respond to seismic activity. When groundwater levels are high, soil loses strength, increasing the risk of liquefaction and foundation instability during an earthquake. In contrast, low groundwater levels can cause differential settlement, which can also weaken structures over time. The study explores how seasonal groundwater fluctuations affect the earthquake resilience of residential buildings. Using PLAXIS 2D, soil and foundation behavior under different water table levels and seismic loads were simulated. By analyzing soil deformation, pore pressure buildup, and building stability, insights into potential risks and ways to improve seismic safety are proposed. The research involves a detailed investigation into how varying groundwater levels influence soil strength, pore water pressure, and foundation response during seismic events. Data from field reports and historical earthquake records are gathered and analyzed. PLAXIS 2D's dynamic analysis capabilities enable to model realistic seismic scenarios, considering both horizontal and vertical ground accelerations. This study underscores the importance of incorporating seasonal groundwater variations into seismic design practices. The findings provide practical recommendations for engineers, urban planners, and policymakers to improve the earthquake resilience of Almaty City's residential infrastructure. By implementing these strategies, engineers can enhance the safety and durability of buildings in the face of seismic events, ensuring better protection for the city's residents and infrastructure.

Keywords: seismic resistance, groundwater table variations, Plaxis dynamic simulation, soil-structure interaction, liquefaction potential.

1 INTRODUCTION

Almaty City, the largest urban center in Kazakhstan, is situated at the foothills of the Trans-Ili Alatau Mountains and lies within a moderate-to-high seismic hazard zone. Seasonal and anthropogenic factors produce fluctuations in the local groundwater table of up to 5 m annually. These variations alter effective stress in near-surface soils, influence soil stiffness and damping properties, and may exacerbate liquefaction potential during strong ground shaking. Despite extensive seismic design guidelines, the influence of groundwater table dynamics on structural resilience remains insufficiently quantified for Almaty's typical mid-rise reinforced concrete residential buildings. Objectives

1. Quantify how seasonal groundwater level changes affect dynamic soil properties (stiffness, damping, and liquefaction resistance) at representative Almaty sites.
2. Assess the subsequent impact on seismic demand (base shear, inter-story drift, and foundation settlement) of standard residential building typologies.
3. Develop design recommendations incorporating groundwater variation into seismic resilience frameworks for residential structures in Almaty.

2 METHODS

Numerical Modeling: A two-dimensional soil‑structure interaction model was constructed in PLAXIS 2D. Soil layers were assigned dynamic properties calibrated to laboratory results for both high and low groundwater scenarios. A six‑degree‑of‑freedom model of a typical 5-story reinforced concrete frame (8 m × 12 m plan) was embedded in the mesh. Real earthquake records (Mw 6.5 events scaled to PGA 0.3 g) were applied at the model boundary. Information about peak ground acceleration is retrieved from historical data according to Silacheva (2018).

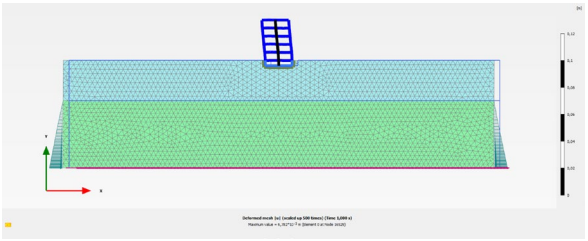


Fig. 1. Plaxis 2D preliminary model.

Figure 1 represents the soil, geometry, and mesh model developed in Plaxis 2D software.

3 RESULTS AND DISCUSSION

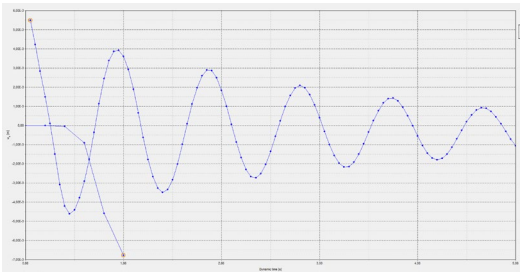


Fig. 2. Free vibration of a house.

By looking at Figure 2 showing free vibration of a house during 5 seconds, the natural frequency of a typical 5-storey building can be estimated. The time range between two peaks is approximately 1 second, which means that the natural frequency of oscillations is 1 Hz.

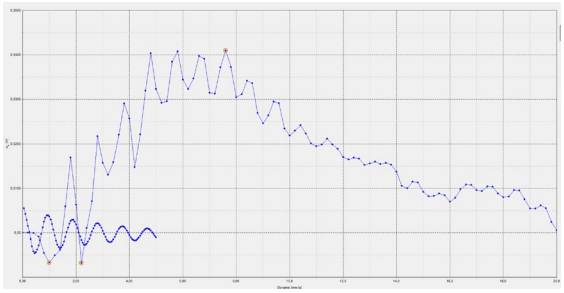


Fig. 3. Earthquake simulation.

Figure 3 shows an earthquake simulation graph, where on y axis total displacement and on x axis dynamic time. The deformation is elastic, because in 20 seconds the building's top point returned to its initial position.

Also, it is important to find information about the groundwater level at different seasons and make a dynamic simulation for all cases for further analysis.

There are different methods of mitigating seismic loads on the foundation. Calvi (2018) observes use of seismic isolation systems, specifically friction-based isolators.

4 CONCLUSIONS

Groundwater table fluctuations significantly influence the seismic resilience of residential buildings in Almaty, especially on fine-grained alluvial and lacustrine deposits. It is therefore recommended that seismic design procedures for such sites:

1. Incorporate seasonally adjusted dynamic soil parameters (modulus and damping) into site response analyses.
2. Evaluate liquefaction potential considering worst-case groundwater levels, and design foundations (e.g., stone columns, deep piles) accordingly.
3. Adopt performance-based drift limits that account for increased deformation demands under high water table conditions.
4. Implement long-term groundwater monitoring near critical buildings to inform maintenance and retrofitting decisions.

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Study of the Stiffness of Floor Diaphragms Made of Hollow-Core Reinforced Concrete Slabs

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ABSTRACT

As part of the housing construction program in Kazakhstan, experimental studies were conducted to assess the rigidity of hollow-core reinforced concrete floor slabs with a thickness of 180 mm. The aim of the study was to determine the horizontal stiffness and performance of the floor system under seismic loads. For the tests, a full-scale floor fragment was fabricated using three hollow-core slabs. The longitudinal joints between the slabs were reinforced with flat vertical reinforcement cages, while the perimeter of the fragment was strengthened with a monolithic reinforced concrete tie beam. The tests were carried out by applying a horizontal load in the plane of the floor along its diagonals to evaluate deformations under racking conditions. During the experiments, values of horizontal force, concrete compression, and tension deformations in diagonal directions were recorded. The loading process was divided into two stages: in the first stage, the fragment was subjected to two cycles of repeated loading up to 90 tf, while in the second stage, the structure was loaded until concrete failure occurred under the support pads at 101.1 tf. The results showed that at a load of up to 90 tf, the floor system functioned as a rigid diaphragm, with residual deformations within acceptable limits. Failure occurred at a load of 101.1 tf due to concrete crushing under the supports. Calculations performed in the "Lira SAPR 2022" software confirmed the reliability of the experimental data.

Thus, the research results confirmed the feasibility of using 180 mm thick hollow-core reinforced concrete slabs in seismic regions, provided that necessary measures are taken, such as reinforced joints, monolithic tie beams, and anchorage connections. These measures ensure sufficient diaphragm stiffness and stability under seismic effects.

Keywords: zoning, engineering-geological conditions, geoinformation database, soil properties, geotechnical mapping,

1 INTRODUCTION

Application of Hollow-Core Prestressed Concrete Slabs in Seismic Zones. Hollow-core prestressed concrete slabs produced using long-line slip-forming are widely used worldwide due to their high stiffness, strength, surface quality, and cost-efficiency. Manufactured on beds over 100 meters long, these slabs can span up to 15 meters and are reinforced solely with longitudinal prestressed strands, without additional mesh, cages, or embedded parts.

However, their use in seismic regions is limited because of challenges in ensuring diaphragm action and insufficient data on their lateral stiffness under seismic loads. According to Kazakhstan's seismic construction code (SP RK 2.03-30-2017), precast slabs must have ribbed or keyed sides and reinforcement extensions to connect with seismic belts and structural elements.

Several retrofit techniques have been applied to adapt these slabs for seismic conditions, such as cutting cores to insert reinforcement cages, drilling lateral holes for anchorage, and installing concrete plugs with rebar. Horizontal bars are also used to anchor slabs to tie beams.

Numerous studies have examined the behavior of such slabs under flexure, shear, and torsion. Researchers like Zhou, Michelini, Nguyen, Park, Tawadrous, and Tienpont have provided experimental data on shear strength, diaphragm action, and membrane behavior. More recent work by Ghawais and El-Sayed focused on the effect of shear span-to-depth ratios.

In Kazakhstan, based on European practices, modified slab designs now include shelves and vertical reinforcement cages to enhance shear capacity and enable monolithic integration. Experimental testing at KazNIISA confirmed sufficient diaphragm stiffness of 220 mm thick slabs for seismic use. The present study assesses the diaphragm stiffness of standard 180 mm thick hollow-core slabs for seismic applications.

2 METHODOLOGY OF RESEARCH

The aim of the study was to evaluate the in-plane stiffness of a floor diaphragm constructed from 180 mm thick hollow-core prestressed concrete slabs manufactured using the long-line slipforming method. For this purpose, a test specimen of the floor diaphragm was fabricated, measuring 6400×4000 mm in plan and consisting of three slabs, each 5980×1195 mm in size (see Fig. 1).

The slabs were reinforced with Ø5 mm high-strength

wire (4Ø5 Vr1400 in the top zone and 2Ø5 Vr1400 in the bottom zone), cast with concrete of class C20/25, and featured longitudinal recesses along the sides. A monolithic reinforced concrete edge beam with a cross-section of 200×180 mm was cast around the perimeter, reinforced with 4Ø12 A400 longitudinal bars and Ø6 A240 stirrups spaced at 150 mm. Vertical flat reinforcement cages (2Ø10 A400 longitudinal bars and Ø6 A240 transverse bars at 200 mm spacing) were placed in the longitudinal joints between the slabs.

After the concrete in the edge beam reached its design strength (at 40 days), the slab fragment was tested under in-plane horizontal loading applied along its diagonals using a hydraulic jack (up to 101.1 tf). Deformations were measured using digital displacement transducers with a resolution of 0.01 mm. Concrete strength was assessed using an IPS-MG.03 rebound device, a Kashkarov hammer, and standard cube compression tests.

The test program included two stages of repeated loading cycles along both diagonals. Strain diagrams were obtained along the compressed and tensioned diagonals for the central slab and for averaged zones that included parts of adjacent slabs and their joints.

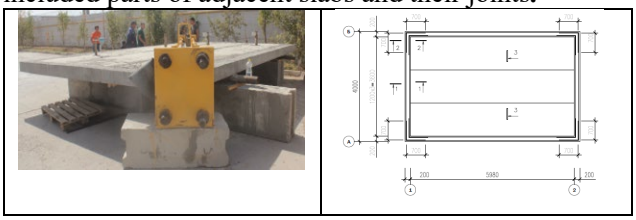


Fig. 1. Floor plate testing scheme

The loading of the test floor fragment was carried out in two stages. In the first stage, two cycles of horizontal loading up to 90 tf were applied along one diagonal. In the second stage, two additional cycles were applied along the opposite diagonal. In the final cycle, the load was increased until failure of the concrete occurred in the support zones, at a horizontal force of 101.1 tf. Figure 1 shows the strain diagrams along the compressed and tensioned diagonals: for the central slab (P2 and P5), as well as for the averaged zones involving the edge slabs and the joints between them (P1, P3, and P4, P6, respectively).

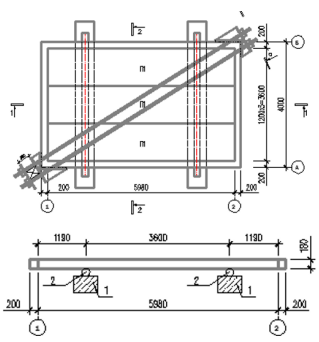


Fig. 2. Floor plate testing scheme

4 CONCLUSIONS

Experimental studies on the behavior of a floor fragment made of 180 mm thick precast hollow-core prestressed concrete slabs (manufactured using the long-line slipforming method) provided data on the stiffness, strength, and deformation characteristics of the structure under repeated horizontal racking loads.

At a horizontal force of N = 90 tf, the slab system acted as a rigid diaphragm: the compressed diagonal shortened by approximately 2 mm, while the tensioned diagonal elongated by 1.5–2 mm. Failure occurred at N = 101 tf due to concrete crushing beneath the support pad.

Numerical analysis performed using LIRA-SAPR 2024 showed that at N = 90 tf, compressive stresses in the central zone of the slab reached 4.5 MPa, and concrete strain was approximately 0.15 ‰.

Overall, the test results confirm the feasibility of using such slabs (series UDS-2PBs1.01.2018) in seismic zones rated 7, 8, and 9 on the MSK scale, provided the following conditions are met: presence of interlocking keys along slab edges, vertical reinforcement cages in the joints, anchorage to underlying structural elements, careful grouting of joints, and the construction of monolithic edge beams and seismic belts.

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BEHAVIOR OF SANDY SOILS OF DIFFERENT PARTICLE SIZES UNDER DINAMIC LOADS

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ABSTRACT

Nowadays various dynamic loads including pile sinking, vibropiling and intensive traffic circulation have a considerable effect on subsoils in case of modern urban development. Dynamic effects might cause additional settlements of foundations of already built structures surrounding a building site. It has been established that dynamic loading leads to liquefaction of sandy soils in certain cases. For this purpose, the experimental research of soil behavior under dynamic effect is a relevant objective. An analytical review of scientifical literature on the topic of the research was performed. The question of sand behavior under dynamic impact was studied by many Soviet and Russian scientists. Foreign authors' scientific works were also analyzed. First of all, these are professors D.D. Barkan, N.N. Maslov, E.A. Voznesensky, K. Ishihara, I.T. Mirsayapov, Z.G. Ter-Martirosyan, A.Z., Ter-Martirosyan V.M. Polunin, Jianfeng Li e.t.c..

The article deals with integrated laboratory study of the water-saturated sandy soil samples of different particle size (coarse, medium and fine sands) and fixed dry density under the conditions of triaxial dynamic compression. A sample was compacted in a latex film in the mold. Then the sample was loaded into the device chamber and saturated with distilled water. After that consolidation process was conducted under drained conditions. Upon completion of consolidation, the drainage valve was closed and vibration began. The experiments were performed with a modern dynamic triaxial compression unit, which allows to simulate various modes of static and dynamic loadings of soil samples. As controlled variables in the research vertical and horizontal static stresses ratio, frequency, time (number of cycles) and particle size. The frequency of the dynamic impact was 1.2 and 3 Hz with an amplitude of 20 kPa. The test was considered complete when a number of cycles reached 10,000 or liquefaction of the soil specimens took place.

The degree of soil strength and deformation characteristics dependence of particle size distribution of dynamic loading is specially noted. Changes in the soil sample's structure and strain accumulation caused with dynamic dresses were taken into account. The results of the research are presented in a graphical form. It is characteristic that for all the fractions considered, the slope of the graph of the dependence of axial deformations on the number of cycles decreases with an increase in the frequency of dynamic impact.

As a result of long-term dynamic loading in sandy soils, the phenomenon of vibrocreep is observed - repacking of particles occurs, as a result, vertical deformations accumulate. When determining the deformation of vibrocreep based on the test results, the dependence of axial deformations on the loading time is plotted. This graph allows tracking the fact of sand liquefaction, which occurs upon reaching 5-% axial deformations. The dependence of pore pressure and axial strains on time and

number of cycles was obtained. It was noted that modulus of deformation of sandy soils decreases as a result of dynamic loading. The dependence of deformation modulus on frequency of vibration was determined. The value of the reduced deformation modulus can be calculated taking into account the magnitude of vibration creep deformations.

Due to the results of the research practical recommendations on building sustainability assessment in case of dynamic loading are given. The veracity of the results is verified with the massive volume of data, that was gotten during the experimental research. Precision of experimental and calculation results has been observed. The results of the research might be applied by civil engineers in foundation design in case of dynamic loading and development of activities for engineering protection of vibration effects.

Keywords: dynamic soil properties, sandy soils, triaxial dynamic compression unit, dynamic loading, liquefaction, amplitude, frequency, stress-strain behavior

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Study of seismic hazard in Zhambyl region of Kazakhstan

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ABSTRACT

The article is devoted to the analysis of the seismic situation in the territory of Zhambyl region (Kazakhstan). To ensure the safety of the population, it is important to specify the level of seismic hazard, which is associated with the rapid growth of urbanized areas. Regional and local seismicity of the Zhambyl region were studied. The earthquake representativeness is estimated by accumulating the number of earthquakes over different observation periods. Macroseismic manifestations of strong earthquakes and patterns of attenuation of the intensity of concussions in the study area are studied. Maps of earthquake epicenters and epicenter densities were constructed, which led to the conclusion that the main seismic activity is observed in the southern and southeastern parts of the region, confined to the Kyrgyz and Talas Alatau ranges, with the western end of the Ile Alatau and with the transition to the Shu-Ili Mountains. In the eastern and western parts of the Zhambyl region, seismic activity is moderate, while the northern part of the territory is characterized by a relatively low level of seismicity. Maps of probabilistic seismic hazard assessment (PSHA) for the territory of Zhambyl region in MSK-64(K) scores were developed using the Open Quake Engine software. Maps are presented in two versions: - with a probability of exceeding the estimated intensity of earthquakes of 10% within 50 years (the average period of occurrence of earthquakes of 475 years) and - with a probability of exceeding the estimated intensity of earthquakes of 2% within 50 years (the average period of occurrence of 2475 years). Thus, the seismic potential of the Zhambyl region is quite high – the possible magnitudes of strong earthquakes with foci in these zones are up to 6.5, while the intensity of concussions in the south of the region can reach up to 8-9 points.

Keywords: earthquake, seismicity, seismic regime, isoseists, seismic generating zones, seismic hazard.

1 INTRODUCTION

The results of studies of seismicity and seismic potential of the territory of the Zhambyl region (Kazakhstan) are presented. According to the official map of the seismic zoning of Kazakhstan, 80% of the region's territory is affected by earthquakes with the intensity of shaking the earth's surface from 6 to 8 points. At the same time, the level of seismic hazard varies in different parts of the territory of the Zhambyl region. The information base of the research was catalogues of strong and weak earthquakes in the Zhambyl region and adjacent territories, which were compiled from macroseismic and instrumental data. In this paper, the seismic hazard assessment is carried out using modern methodologies and analysis tools, as well as the most complete and up-to-date information available for the territory under consideration.

1.1 Methods of research

When studying the seismic situation, the reliability of conclusions significantly depends on the representativeness of earthquakes for different periods of observation, separately for strong and weak earthquakes. At the same time, great importance is attached to the assessment of the K-min level (where K is the energy class of the earthquake), which is associated with the practical majority of seismic events in the Zhambyl region in the earthquake catalog since the early 50s, since this level changes over time due to changes in seismic observation systems. Data on strong earthquakes are more reliable due to their presence over a longer period of time.

2 RESULTS

The region is susceptible to strong earthquakes with foci located both on the territory of the region itself and beyond. Local foci are confined mainly to mountainous areas in the south of the region, which belong to the so-called young tectonic structures. Tectonic movements here are very intense, which creates an increased level of seismic activity. According to information (Nurmagambetov A., et al., 2006), the first reliable mention of a strong earthquake in the Zhambyl region dates back to March 22, 1865. Thus, during the period from 1865 to the present, one earthquake occurred in the territory of Zhambyl region with the intensity of concussions in the epicenter I_0=8 points, one-with the intensity in the epicenter I_0=7-8 points, two-with I_0=7 points, and five earthquakes with I_0= 6 points. In addition, numerous transit earthquakes were felt with the intensity of shaking the earth's surface of 6-5 or less points. A strong earthquake is a rare event and it is very difficult to assess the level of the current seismic situation in a particular area. Weak tremors come to the rescue. Based on them, you can quickly accumulate observational material that is sufficient for statistical conclusions. This material can be compared with data for strong earthquakes and find common patterns for both. In addition, a detailed study of the entire population of earthquakes, both strong and weak, can help in assessing the degree of seismic hazard for individual territories and in developing methods for predicting strong earthquakes.

As we can see from Figure 1, in those places where strong earthquakes occurred, the density of epicenters of weak tremors is relatively high. This is one of the fundamental patterns of the spatiotemporal distribution of weak earthquakes. Indicators of the level of current seismic activity in the region as a whole and individual seismically active zones are parameters of the seismic regime, which are estimated using the earthquake repeatability graph (Sydykov A. 2004; Sadykova A. B., 2012).

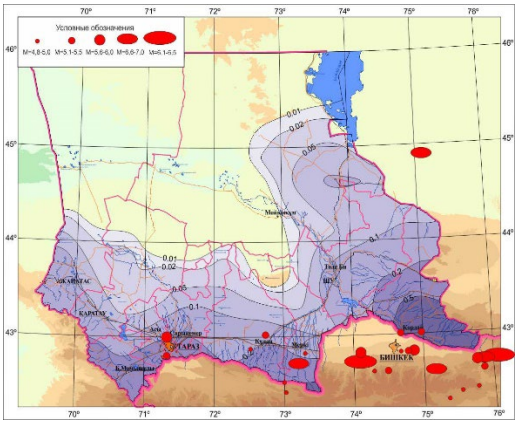


Fig. 1. Map of the density of earthquake epicenters with epicenters strong M≥4.8 from earliest times to 2022.

3 DISCUSSION

Assessment of the level of seismic hazard is aimed at identifying the seismic generating zones, determining the parameters of their seismic regime, and finally calculating the seismic effect on the earth's surface created by them. These studies are based on a detailed and comprehensive study of the deep structure of the Earth's crust, seismotectonics, regional seismicity, and engineering seismology. In the practice of seismic hazard assessment, two approaches are used: deterministic and probabilistic. The deterministic campaign was the standard and was used in Kazakhstan in earlier times, until the end of the 20th century. In the 21st century, the second approach is mainly used – probabilistic, which brings the domestic methodology closer to the international one, which has long been generally accepted in many countries when assessing seismic hazards. The methodology of seismic hazard assessment based on the probabilistic approach applied to seismically active regions of Kazakhstan is described in some detail in a number of publications (Silacheva, N. V., et al, 2023; Allen, T. I., et al, 2012; Bindi D., et al, 2011). In general terms, it is reduced to determining the maximum seismic impacts expected with a certain probability in a given territory in a given time interval and due to the frequency of earthquakes.

Following this methodology, using the Open Quake Engine software (Open Quake Engine software), taking into account linear and areal sources, probabilistic maps of the seismic hazard assessment of the Zhambyl region were obtained for two probability levels of 10% and 2% over 50 years in points of the MSK-64(*K*) scale, which are presented on a scale of 1:1,000,000 in Figures (Fig.2).

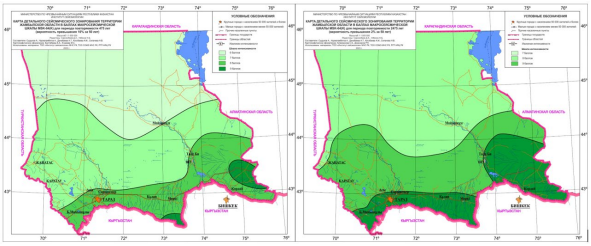


Fig. 2. - Maps of seismic hazard assessment for the territory of the Zhambyl region in points of the MSK-64(K) macroseismic scale with a probability of exceeding the calculated intensity: 10% for 50 years (average repeatability period of 475 years) and 2% for 50 years (average repeatability period of 2475 years).

As can be seen from the map for the 475-year repeatability period of seismic shocks, the maximum intensity of concussions (9 points) occurs in a small area south of. Merke village (western dip of the Kyrgyz ridge) and in the north-east of Kordai village (Shu-Ili Mountains). The less intense (8-point zone) is extended in a north-northeasterly direction. The map for the period of occurrence of seismic shocks of 2475 years differs markedly from the previous one, where the zones of shocks occupy large areas. So, the area of the 7-point zone practically extends up to the northern borders of the region.

4 CONCLUSION

To study the nature of seismicity and its manifestation in the territory of Zhambyl region, an information seismological database was formed based on macroseismic materials and regional catalogs of earthquakes of various levels. Spatial and temporal patterns of earthquake occurrence are investigated not only within the Zhambyl region, but also on a larger scale - within a single seismically active belt, which is more appropriate when studying the nature of the seismic process, and which is difficult or even impossible to do with only local or regional studies. The seismic hazard assessment of the territory of Zhambyl region was carried out in a probabilistic setting and for quantitative parameters using the Open Quake Engine - an open software for calculating seismic hazard and risk, which is a new direction for Kazakhstan. In conclusion, the potential seismic hazard of the territory of the Zhambyl region is determined by the seismogenic zones in the south and southeast of the territory, geographically connected with the Kyrgyz and Talas Alatau ranges, and the western end of the Ile Alatau range, which passes into the Shu-Ili Mountains. The seismic potential of these zones is quite large – possible earthquake magnitudes, with foci in these zones, reach up to M=6.5, while the intensity of concussions in the south of the region can reach up to 8-9 points.

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Monitoring of permafrost conditions in built up areas with thermal imaging

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ABSTRACT

The degradation of permafrost in certain areas poses a significant threat to the stability of buildings and structures. To ensure the operability of bases and foundations, it is essential to implement geotechnical monitoring systems to promptly detect and address any issues. The organization of geotechnical monitoring for all buildings and structures and creation of an automated monitoring system requires high costs. It is important to use monitoring methods that will allow obtaining data for the entire built-up area at once, which will help assess the overall picture of changes in the temperature regime of soils and prompt identification of areas with increased risk for the operation of buildings and structures. A review of Russian and international scientific research and practical experience in the use of thermal imaging methods for monitoring soils and foundations in permafrost is conducted. The advantage of the thermal imaging method is its high scalability and the ability to survey large areas in a short time. The use of thermal imaging has been proposed for use as part of geotechnical monitoring and for mapping the condition of permafrost soils in built-up areas under climate warming conditions. The possibility of using the thermal imaging method to monitor the operability of heat pipes and identify risk zones for urban development in the permafrost zone is investigated. The importance of correct processing of the results of thermal imaging is emphasized in order to determine the actual temperature of the observed object and select the appropriate thermal imaging equipment for the tasks to be solved. The potential and limits of the application of the thermal imaging method for monitoring permafrost soils in built-up areas are assessed.

Keywords: permafrost, climate warming, thermal imaging, geotechnical monitoring, permafrost soils, heat pipes

1 INTRODUCTION

The permafrost region is an ecological system that is extremely sensitive to external influences, such as human activity and global climate change. Permafrost may extend for several kilometers in depth, but the top layer, which serves as the base for buildings and structures, is significantly impacted by technogenic thermal effects and climate change.

The degradation of permafrost in certain areas poses a significant threat to the stability of buildings and structures. To ensure the operability of bases and foundations, it is essential to implement geotechnical monitoring systems to promptly detect and address any issues. For many objects located in the permafrost zone, measurements are not carried out or the geotechnical monitoring system is not installed. The organization of geotechnical monitoring for all buildings and structures requires high costs, in addition, to determine the condition of permafrost soils and facilities in a certain area, it is necessary to create an automated monitoring system.

It is important to use monitoring methods that will allow obtaining data for the entire built-up area at once, which will help assess the overall picture of changes in the temperature regime of soils and prompt identification of areas with increased risk for the operation of buildings

and structures. Thermal imaging is one of these methods.

2 THERMAL IMAGING METHOD FOR MONITORING SOIL BASES AND FOUNDATIONS ON PERMAFROST

2.1 Usage and study

Despite its effectiveness and informative value, few works by Russian and international researchers have been devoted to the thermal imaging method for monitoring soil bases and foundations on permafrost [1,4]. Thermal imaging is used for geocryological studies [3], in the design and monitoring of roads and railways, and allows determining the operability of heat pipes and artificial structures. The thermographic camera refers to remote sensing instruments, therefore, the advantage of thermal imaging is the ability to survey large areas in a short period of time. Thermal imaging aerial photography is used to monitor the condition of pipelines and other linearly extended objects [4].

The use of thermal imaging has been proposed for use as part of geotechnical monitoring and for mapping the condition of permafrost soils in built-up areas under climate warming conditions.

2.2 Determination of the operability of cooling devices and monitoring of the condition of permafrost soils in built-up areas

The thermal imaging method shows its effectiveness for monitoring the operability of cooling devices, but its application requires a thorough technological study.

The thermal imaging method allows to identify risk zones for urban development in the permafrost zone. It enables to control the surface temperature and identify the centers of its increase in a timely manner. With its help, it is possible to obtain information about the distribution of thermal energy on the surface and identify anomalies caused, for example, by heat pipes leaks [2].

All of the above makes it potentially possible to use the thermal imaging method to identify risk areas for urban and settlement development in the permafrost zone.

3 RECOMMENDATIONS FOR THE USE OF THE THERMAL IMAGING METHOD

3.1 Choice of thermal imaging equipment

It is important to correctly process the results of thermal imaging to determine the actual temperature of the observed object and select the appropriate thermal imaging equipment.

At the same time, it is necessary to methodically coordinate the technical characteristics of the thermal imager, the distance along the monitored surface and the size of the monitoring area. Otherwise, distant temperature measurements will not be reliable, but can only be used as visualization of temperature anomalies.

3.2 Processing of the thermal imaging results

The processing of thermal imaging results includes standard technologies for detecting temperature anomalies based on defect-quality characteristics, as well as comparing calculated and experimental data obtained using models of heat transfer of thawing of frozen soils and temperature fields recorded by a thermal imager, taking into account the coordinate identification system.

3.3 The limits of the method's application

The boundaries of the thermal imaging method for monitoring permafrost in built-up areas are determined by the use of appropriate thermal imaging technology and diagnostic procedures.

4 CONCLUSIONS

1. A review of Russian and international scientific research and practical experience in the use of thermal imaging techniques for monitoring soil bases and foundations in permafrost shows that this task requires further methodological study.

2. The authors proposed the use of thermal imaging for use as part of geotechnical monitoring and for mapping the condition of permafrost soils in built-up areas in a warming climate.

3. The possibility of using the thermal imaging method to identify risk zones for urban development in the permafrost zone is investigated.

4. Recommendations are given on processing the results of thermal imaging and the selection of thermal imaging equipment for monitoring the condition of permafrost soils in built-up areas.

5. The potential and limits of the application of the thermal imaging method for monitoring permafrost soils in built-up areas are assessed.

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Study of the possibility and efficiency of using geosynthetic reinforcement of weak soils in seismic-prone regions of Kazakhstan

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ABSTRACT

Strong earthquakes pose a real threat to the population and infrastructure of earthquake-prone regions of the Republic of Kazakhstan. Reducing the effects of seismic impacts on buildings and structures is one of the fundamental tasks of earthquake-resistant construction. Research is currently being conducted in this area, taking into account modern technologies. There are many methods for ensuring seismic resistance of buildings and structures, one of which can be considered various methods of strengthening soil foundations by improving their physical and mechanical properties and increasing their bearing capacity. Particularly relevant in the context of construction in seismically active regions is the study of the most effective methods of strengthening weak soils using innovative building materials. Currently, methods of soil strengthening using geosynthetic materials, which are used to reinforce soil massifs, have become widely used in geotechnical practice. Geosynthetic reinforcement of the foundation makes it possible to use local soils at the construction site and thus reduce the costs of the zero cycle construction. Reinforcement with geosynthetic materials allows for increasing strength and deformation properties by creating a more rigid area under the foundation sole, reducing settlement and increasing its bearing capacity. In international construction, the use of geosynthetic materials for strengthening soil foundations in road construction has been known for over thirty years. The basis for the development of this area is availability, reliability and the possibility of application in mass construction. However, the use of geosynthetic materials in Kazakhstan is limited due to little practical experience in their use and insufficient research in this area. At present, it will be relevant to study the seismic resistance of buildings on soils reinforced with geosynthetic material and to determine the effectiveness of using this technical solution in seismically hazardous regions.

Keywords: methods of strengthening weak soils, geosynthetic materials, geosynthetic reinforcement of soil foundations, ensuring seismic resistance of structures

MATERIALS AND METHODS OF THE STUDY

As part of this study, an experiment was conducted on the seismic resistance of a small-scale model of a building on a foundation made of undrained soil reinforced with geosynthetic material. The purpose of the experiment was to study the effect of geosynthetic reinforcement on the seismic resistance of a building, as well as the possibility and effectiveness of using this method of strengthening soils with weak seismic properties in construction conditions in seismically hazardous regions.

The experiment was conducted in two stages: a study of the seismic resistance of a building on a soil foundation made of undrained soils without reinforcement and after reinforcement using geosynthetic material.

A prefabricated small-scale model of a single-story block building was used as the object of the experiment. The tests were conducted in the laboratory of earthquake-resistant construction of the International Educational Corporation using a seismic platform simulating dynamic effects of the seismic vibration type. This seismic platform contains a monolithic reinforced concrete foundation with embedded parts for fastening laid during its construction, and a platform for placing a model of a building or structure, which serves as the base of the model being tested. The means for generating dynamic loads is an electric motor fixed to the monolithic reinforced concrete foundation. The entire process of the experiment (switching on, switching off, regulating the speed of rotation of the electric motor) is set and controlled by the researcher via the control panel.

The process of recording seismic vibrations is carried out by seismic sensors, which are connected to a computer that processes the received data using a special program and displays the vibrations of the site

for placing the model of the building or structure, as well as the vibrations of the models of the buildings or structures themselves in tabular and/or graphical form. Research on this seismic platform is carried out as follows: compact three-component seismic sensors (accelerometers, seismometers) are installed and connected depending on the task at hand. Due to the interaction of all structural units and parts of the seismic platform, a volumetric controlled imitation of the impacts and vibrations of the site is ensured for placing a model of a building or structure that is as close as possible to seismic ones, for studying the seismic resistance of physical models of buildings or structures.

Geogrid was chosen as a reinforcing geosynthetic material for the experiment, as one of the most common representatives of geosynthetic materials. They are used in most cases for reinforcing those construction objects where reinforcing materials can be subject to strong stretching.

The soil base made of loam was formed in a special box made of organic glass and installed on the seismic platform site. For the first stage, the soil base was examined without any additional strengthening, at the second stage, the soil base was strengthened with a hesynthetic material (geogrid). During the experiment, the change in the height of the ground level in the box "before" and "after" the seismic impact was recorded at both stages of the experiment in order to determine the subsidence of the building model as a result of the seismic impact.

The model prepared for testing is installed in a box with a soil base. Sensors are fixed to the model and the soil base for subsequent recording of accelerograms in strict accordance with the axes. Two sensors were fixed to the model itself and two to the soil base. The seismic platform test at the first stage of the experiment lasted for 100 seconds.

For the second stage of the experiment, the soil base was formed from the same non-draining local soil (loam) and reinforced with two layers of geosynthetic material (geogrid). This was done by excavating a portion of the soil, laying down the reinforcing material, and compacting it using partial wetting and manual tamping.

EXPERIMENTAL RESULTS

During the experiment, the following data were obtained during comparative tests of reinforced and unreinforced soil foundations: as a result of strengthening the soil foundation with a geogrid, the settlement decreased by almost 2 times. During the first stage of the experiment, measurements showed that the soil settlement was 9 mm (based on the average results of measurements from four sides of the model). After strengthening, the soil settlement averaged 5 mm. Thus, it can be concluded that the deformation properties of the soil have been improved due to geosynthetic reinforcement, since its settlement has decreased, and the bearing capacity has increased due to the additional inclusion of the bearing capacity of the reinforcing material.

NUMERICAL MODELING USING THE PLAXIS PROGRAM

To verify the results and compare the obtained experimental data, numerical modeling of the conducted reinforcement of the soil foundation was carried out using the PLAXIS program. Analyzing the obtained results, we can conclude that reinforcement with geosynthetic material has the greatest effect on the deformation of the soil base in the vertical direction, thus improving the deformation properties of the soil and reducing settlement in the event of seismic impact.

CONCLUSIONS

The analysis of numerical modeling data using the PLAXIS program allows us to draw the following conclusions:

1. Reinforcement of the soil base with horizontal layers allows us to increase the bearing capacity of the loam and reduce its settlement as a result of seismic impact.
2. The efficiency of geosynthetic reinforcement depends on the primary characteristics of the soil and the geosynthetic material used.
3. The highest reinforcement coefficient can be obtained by using geosynthetics with high strength and deformation characteristics.
4. In case of increased intensity of seismic load, it is worth considering the issue of using horizontal-vertical reinforcement or replacing part of the soil (for example, replacing part of the loam with crushed stone).
5. The feasibility and effectiveness of using this method of strengthening weak undrained soil to increase the seismic resistance of buildings in seismically hazardous regions requires further research in this area.

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Investigation of Pile-Soil Interaction in Bridge Foundations for Complex Ground Conditions

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Abstract

Pile-soil interaction is a critical component in the design and construction of bridge foundations, particularly in regions with complex geological formations. This study presents the results of a static plate load test performed in borehole 01.03 at Pier No. 1 of a bridge crossing the Chusovaya River, which is part of the Perm–Berezniki highway project in Russia. The objective of the investigation was to determine the actual bearing capacity of the gravelly soil layer beneath the pile tip and to verify the engineering assumptions employed in the structural design. The testing involved a rigid circular plate with a diameter of 325 mm subjected to incremental static loading, reaching a maximum pressure of 11.7 MPa. The recorded maximum settlement was 195.54 mm. The test confirmed that the designed pile tip resistance of 5.8 MPa is consistent with in-situ test data. These findings contribute to the reliability of foundation performance assessments and the advancement of design methodologies for pile-supported bridge structures in variable ground conditions.

Key words: pile, bridge foundations, FEA, soil, settlement control, geotechnical engineering.

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1. INTRODUCTION

Understanding pile-soil interaction is fundamental to ensuring the performance and safety of bridge foundations, especially when structures are founded on deep piles embedded in complex ground conditions. In many bridge construction projects, uncertainties in subsurface conditions—such as the presence of loose fill, heterogeneous layers, and water-saturated soils—necessitate rigorous geotechnical evaluations.

The relevance of this study lies in its application to a real-world engineering project involving the construction of a new segment of a bridge along the Perm–Berezniki highway corridor in Russia. In such large-scale infrastructure, the use of deep bored piles requires careful verification through field tests to assess the bearing capacity and deformation characteristics of the soil under load. Plate load testing, a classical and reliable method for such evaluations, is utilized here to analyze the load-displacement response of the soil beneath the foundation level.

3. GENERAL SPECIFICATIONS

The present study is conducted within the framework of a large-scale infrastructure development project involving the construction and modernization of the Perm–Berezniki highway and the Eastern bypass of the city of Perm in the Russian Federation. The specific focus of the investigation is on Pier No. 1 of a newly constructed bridge crossing the Chusovaya River—an essential structural element that forms part of a critical transportation corridor.

4. FIGURES

The static plate load test was performed according to Russian standards, particularly GOST 20276-2012 [4, 7] and SP 24.13330.2011 [5,7], which regulate the field methods and design principles for pile foundations (Fig.1) .

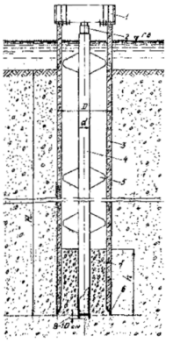


Figure 1. Schematic diagram of installation for stamp tests of soils at the level of pile bottom.

Table 1. Loading Steps and Holding Times.

Step No.	Load on Top of the Loading Column (kN)	Pressure Under the Plate (kPa)	Holding Time / Condition
1	0	368*	—
2	10	489	5 minutes
3	20	609	Stabilization to ≤ 0.1 mm
4	250	3382	Until ≤ 0.1 mm over the last 30 minutes
5	480	6154	—
6	710	8927	—
7	940	11699	—
8	480	6154	15 minutes

* Load due to self-weight of the plate, column, and jack (approx. 3.07 t)

6. Conclusions

The results of this study highlight the significance of conducting static plate load tests at the pile base in assessing the geotechnical performance of deep foundations under complex soil conditions.

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Investigation of the bearing capacity of drilling piles by field testing under static loads

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ABSTRACT

The article presents the results of field control tests of working drilling piles conducted on a construction site in order to determine their actual bearing capacity under the influence of static pulling loads. The object of the study is three drilling piles No. 70, No. 143 and No. 169 with a diameter of 620 mm, made according to the project. The tests were carried out as part of the technical quality control of pile work and assessment of compliance with design requirements. The main objective of the experiment was to establish the actual bearing capacity of the piles under pulling load, compare the results with the calculated values, and analyze the influence of soil conditions and pile construction technology on their behavior under load. For the tests, the static loading method was used, which includes a smooth increase in forces with the movement of the pile head fixed at each stage. The loads were transferred using a special installation with a hydraulic jack and an anchor system that eliminates the displacement of surrounding elements. The test results showed that the bearing capacity of all the piles under study exceeds the minimum design values, which indicates sufficient quality of work and reliability of the technology used. The tests also revealed features of the deformative behavior of piles, characteristic of specific engineering and geological conditions of the site. A comparative analysis of the data from the three piles allowed us to draw conclusions about the stability of the results and the repeatability of drilling and concreting technology. The results obtained can be used in the further design of similar foundations, as well as to improve the methods of quality control of pile work. The conducted field tests have confirmed the effectiveness of the method of pulling loads as one of the reliable ways to assess the bearing capacity of drilling piles.

Keywords: drilling pile, static tests, engineering and geological conditions, bearing capacity.

1 INTRODUCTION

Drilling piles are widely used as deep foundation elements for buildings and structures erected in areas with weak and compressible soils. Their reliability and load-bearing capacity are key to ensuring the overall stability and safety of structures (Havryliuk, O. et al.,2024).

This study examines the results of field tests of drilling piles performed on a construction site in order to determine their actual bearing capacity under the influence of static pulling loads. The bearing capacity of drilling piles is influenced by the properties of the soil, the geometry of the piles and the construction technology.

The tests made it possible to assess the compliance of the actual characteristics of the piles with the design requirements and to identify the impact of the engineering and geological conditions of the site on their operation (Cong-Oanh Nguyen et al., 2024). Special attention is paid to the analysis of the interaction of piles with the soil base and the influence of pile construction technology on their behavior under load. In addition, the practical application of the test results makes it possible to refine the design models of piles and increase the efficiency of design solutions. The data obtained is used to develop engineering recommendations for optimizing the design and technology of drilling piles (Kurniadhi, R. et al., 2023).

2 METHODOLOGY OF RESEARCH

Static tests of drilling piles were carried out at a construction site in Pavlodar, Republic of Kazakhstan.

Field tests were performed on three drilling piles No. 70, No. 143 and No. 169, with a diameter of 620 mm, at the construction site of the facility. The tests were carried

out using static vertical loads aimed at pulling piles out of the base, in accordance with regulatory requirements.

The absolute marks of the ground surface on which the piles were made are:

- for pile No. 70 — 119.766 m;
- for piles No. 143 and No. 169 — 119.762 m.

The depth of immersion of all piles was:

- for pile No. 70 — 26.866 m, the total length of the pile is 27.066 m;
- for piles No. 143 and No. 169 — 26.862 m, the total length of each pile is 27.062 m

The absolute mark of the sole of all piles is 92.900 m. The tested piles are related to the working elements of the structure and were selected for testing as part of the quality control of pile work. Working drilling piles were tested with static vertical-pulling, stepwise-increasing loads up to 1047kN, steps of 185kN and 62kN (the first three steps of 185kN and then 62kN). Field tests were carried out 21-37 days after the production of the boring piles, after the concrete pile strength was at least 80%.

The measurement of pile movements (exit from the ground) was carried out by two deflection meters of type 6 PAO with a division price of 0.01 mm. All the data obtained were entered into the field test log, accordingly, Fig. 1 shows a reference system with 6PAO deflectors, hydraulic jacks Energpred DG100P200, dynamometer Load Cell 100t.



Fig. 1. Reference system with 6PAO deflectors, hydraulic jacks Energpred DG100P200, dynamometer Load Cell 100t

3 RESEARCH RESULTS

Working drilling piles were tested with static vertical-pulling, stepwise-increasing loads up to 1047kN, steps of 185kN and 62kN (the first three steps of 185kN and then 62kN).

With the maximum applied load on piles No. 70, No. 143 and No. 169 equal to 1047 kN, their movements (exit from the ground) were 3.47mm, 3.61 mm and 3.35mm, respectively, shown in Table 1.

Table 1. Test Results of Piles Under Static Uplift Loads

№ of piles	The brand and cross section of the piles	The depth of the pile made in the ground, m	Maximum applied load, kN	Displacement at the highest applied load, mm	Bearing capacity of the pile, kN
№ 70	Bored pile Ø620мм, BP-1	26.866	1047	3.47	678
№ 143	Bored pile, Ø620мм, BP-1	26.862	1047	3.61	678
№ 169	Bored pile Ø620мм, BP-1	26.862	1047	3.35	678

4 CONCLUSIONS

Static tests of three working drilling piles SB-1, Ø620 mm No. 70, No. 143 and No. 169 with the application of vertical pulling loads were carried out at the construction site of the studied object.As a result of the tests, it was found that the actual bearing capacity of the piles for pulling is 678 kN.According to clause 4.4.1.11 of the Joint Venture of the Republic of Kazakhstan 5.01-103-2013 "Pile foundations", taking into account the reliability coefficient of responsibility $\gamma_к$=1.2, The estimated allowable load on the drilling piles is 565 kN. The results obtained confirm the compliance of the bearing capacity of the piles with the design requirements and allow their use as reliable foundation elements in these engineering and geological conditions.

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Assessment of the bearing capacity of driven piles in subsiding soils based on static tests

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ABSTRACT

The article is devoted to the study of the behaviour of driven reinforced concrete piles C120.30-13 in type I subsiding soils in order to determine their bearing capacity and suitability for use in complex engineering-geological conditions. The relevance of the work is due to the need to ensure the reliability of foundations of buildings and structures in areas with soils prone to subsidence when wet. The study uses field tests of piles conducted in accordance with the requirements of GOST 5686–2012. Before the tests began, the site foundation was soaked to simulate settlement conditions. Drainage wells and trenches filled with crushed stone were used for this purpose, ensuring uniform saturation of the sandy loam with moisture to a depth of 3.8 m. The physical and mechanical characteristics of the soil confirmed that the required moisture level had been achieved, creating realistic conditions for testing. The field programme included static pull-out tests of three piles with a cross-section of 300×300 mm and a length of 12 m. Loads were applied to the piles in stages using a hydraulic jack, and the displacements were recorded with high precision using deflection gauges. The maximum load applied reached 664 kN. The results showed minimal pile settlement (from 4.87 to 8.19 mm), which indicates the high bearing capacity and reliability of the piles even in conditions of subsiding soils. The findings of the study confirm the possibility of widespread use of driven piles in areas with Type I subsiding soils. The technology ensures reliable interaction between the piles and the saturated foundation, minimal deformation and compliance with regulatory requirements. The data obtained can be used in the design of foundations and the development of technical solutions for construction in complex geotechnical conditions.

Keywords: Keywords: settlement soil, driven pile, static tests, bearing capacity.

1 INTRODUCTION

Studying the behaviour of driven piles in subsiding soils is important for foundation design. Piles interacting with saturated soil demonstrate stability with minimal deformation (Mohamad H. et al.2012). The study is based on field tests that allow assessing the suitability of piles for complex geological conditions (Meyerhof G. G. et al. 1978).

2 METHODOLOGY OF RESEARCH

Based on engineering and geological surveys, the presence of subsiding sandy loam was established at the construction site. Before conducting the tests, the foundation was soaked using the trench and drainage well method, followed by filling with crushed stone, which ensured uniform water saturation.

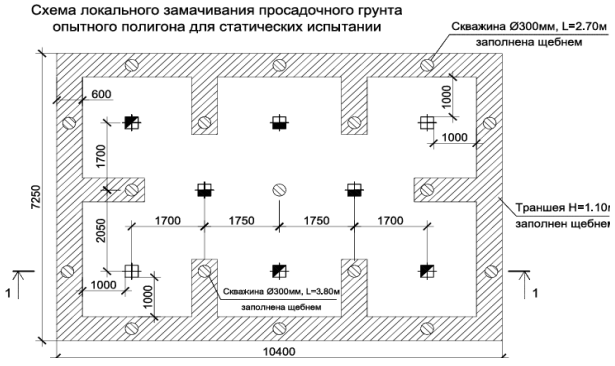


Fig. 1. Soaking scheme for subsiding soil

As part of field tests to assess the bearing capacity of

the foundation, three driven piles were tested using static pull-out loads.

The tests were conducted 40–43 days after driving, on three test piles C120.30-13 (TR-1, TR-8, and TR-7) with a length of 12.0 m and a cross-section of 300×300 mm. The loads were applied in stages using a hydraulic jack and force control via a pressure gauge. Settlement measurements were taken with deflection gauges with an accuracy of 0.01 mm.

The load on the piles was increased in stages: the first three stages were 91 kN each, the subsequent stages were 61 kN each, and then 30 kN, up to a maximum value of 664 kN. During testing at each load application level, the readings of all control and measuring instruments are recorded, and average values are calculated, which are used to control the load value and evaluate pile displacement. Each load stage is maintained until a conditional stabilization criterion is reached, characterized by pile displacement of no more than 0.1 mm during the last hour of measurements. After reaching the maximum force, unloading is performed in stages of double the magnitude, with observation for at least 15 minutes at each stage. The end of unloading was followed by 60 minutes of observation of elastic recovery movements, with data recorded every 15 minutes. The tests are shown in Fig. 2.



Fig. 2. Conducting static tests of driven piles for bearing capacity

3 RESEARCH RESULTS

As a result of soaking the base to a depth of 3.80 m, uniform water saturation of the subsiding soil within the polygon was ensured. Physical and mechanical tests of sandy loam samples showed a moisture content above 0.8 at all depths, which complies with the requirements of clause 4.6 of GOST 5686–2012 and confirms the correctness of the foundation preparation for testing.

Based on the results of field tests conducted in accordance with the requirements of GOST 5686-2012, the following values were obtained for pile settlement at a maximum load of 664 kN: their displacement (settlement) was 4.87 mm, 8.19 mm, and 6.51 mm, respectively, as shown in Table 1.

Table 1. Results of pile testing with static pull-out loads

Pile number	Pile brand and cross-section, mm	Depth of pile driving, m	Absolute elevation of pile heads	Max. applied load, kN	Displacement at maximum applied load, mm
TR-1	C120.30-13, 300x300	11.4	120.772	664	4.87
TR-8	C120.30-13, 300x300	11.4	120.816	664	8.19
TR-7	C120.30-13, 300x300	11.4	120.884	664	6.51

4 CONCLUSIONS

The results of the field static tests of driven reinforced concrete piles C120.30-13 under simulated conditions of type I subsiding soils demonstrated high bearing capacity and minimal settlement (4.87 to 8.19 mm) at a maximum applied load of 664 kN. This confirms the suitability of such piles for use in challenging geotechnical conditions with saturated sandy loam. The preparation method—pre-soaking the soil using trenches and drainage wells filled with crushed stone—provided effective water saturation, creating realistic testing conditions that comply with GOST 5686–2012. The piles showed stable performance and reliable load-bearing interaction with the subsiding foundation soil. These findings support the feasibility of using driven piles for foundation construction in areas prone to subsidence and can serve as a basis for engineering design solutions aimed at ensuring structural safety and durability.

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Accepted extended abstracts:

(Presentations will not be given.)

Determination of the soil resistance value for the foundation design of reconstructed buildings

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ABSTRACT

The calculation of foundations for reconstructed buildings on sandy and clayey soils requires consideration of the increased design resistance of compacted soil (R_{comp}). Analysis of established methods shows that soil properties typically improve during long-term operation due to foundation-induced compaction. This study refined the determination of R_{comp} by modifying SP 22.13330.2016's formula (5.7) with coefficients accounting for changes in soil characteristics. The method demonstrates that clayey low-plastic soils show the greatest R_{comp} increase (45-60%), while water-saturated soils exhibit minimal gains (20-25%). Importantly, when additional loading (p) is below 0.6-0.7R, resistance may decrease by 10-15%, whereas optimal compaction occurs at $p \geq 0.8R$. These findings emphasize the need for careful evaluation of compaction effects when assessing foundation capacity for reconstructed structures, particularly considering operational duration and loading history. The developed approach provides more accurate predictions of bearing capacity by accounting for both beneficial compaction effects and potential weakening from saturation, offering practical improvements over conventional design methods.

Keywords: design resistance of the foundation soil, shallow foundations, compaction of the base by pressure from the foundations, improvement of soil properties

1 INTRODUCTION

The calculating the bases and foundations of reconstructed buildings on sandy and clayey soils in Russia uses the parameter of design soil resistance R according to SP 22.13330.2016. Long-term building operation leads to soil compaction under foundation pressure, typically improving soil properties and increasing resistance to R_{comp}. The authors analyzed established methods for determining R_{comp} developed by Russian researchers (Dalmatov, Konovalov, Polishchuk et al.) over the past 50 years, revealing the need for methodological refinement. This study focuses on developing an improved approach for evaluating R_{comp} to enhance foundation design accuracy for building reconstruction projects, accounting for soil property changes during long-term loading while maintaining compliance with current regulatory framework. The research particularly addresses the need to consider both soil strengthening from compaction and potential weakening effects from factors like water saturation during a structure's service life.

2 MATERIALS AND METHODS

Current design standards provide limited guidance for determining the design resistance of compacted foundation soils (R_{comp}) in building reconstruction projects, often leading to the use of natural soil characteristics from new construction investigations. Several established methods exist for evaluating R_{comp}, including experimental approaches involving soil sampling beneath existing foundations, and analytical methods incorporating enhancement factors [1 – 7]. The experimental method, while reliable, requires extensive field investigations and laboratory testing. Analytical approaches include: 1) the Zurnadzhi-Filatova method using load-dependent coefficients (1.0-1.5); 2) the Konovalov-Roitman formula accounting for soil compression effects; 3) the Polyakov-Sokolov simplified approach; and 4) the Polishchuk method (TSASU) which modifies standard calculations with coefficients for compaction and saturation effects. The Polishchuk method represents the most comprehensive approach, applicable to clayey soils of varying consistency and incorporating factors for density (K_γ), friction angle (K_φ), cohesion (K_c), and settlement utilization (K_s) [8, 9]. These coefficients vary based on the p/R ratio (0.5-0.8: 3-5% density increase, 4-8% friction angle increase, 25-60% cohesion increase; p/R>0.8: 7-17% density increase, 8-10% friction angle increase, 30-60%

cohesion increase). The methods demonstrate that maximum soil improvement occurs when operational pressures exceed 80% of design resistance ($p/R \geq 0.8$), while lower pressures ($p/R \leq 0.7$) may actually reduce capacity by 10-15%. These approaches enable more accurate assessment of foundation capacity in reconstruction projects by accounting for long-term soil behavior under sustained loading.

3 RESULTS

It has been determined that for the design of foundations and shallow foundations (ribbon, isolated, deep) for reconstructed buildings, it is essential to determine the parameter of calculated resistance of compacted soil at the base of the structure. Engineering methods for calculating the R_p are based on recommendations from regulatory documents for calculating R for natural (uncompacted) foundations (in accordance with SP 22.13330.2016) that have been supplemented to account for changes in soil properties beneath foundations due to compaction during building operation. A review of experimental and theoretical studies suggests that the methods currently used for determining R_{comp} are applicable to certain soil types (sands, clays, loams, and sandy loams) and have limited applicability. Of the domestic methods considered for determining R_{comp}, the most effective is the engineering approach proposed at TSASU, which considers not only the improvement of soil properties under foundations due to compaction, but also the weakening (deterioration) caused by soil flooding during building operation. This method has broader applicability, covering clays, loams, and sandy loams from hard to soft-plastic (plastic) consistency states.

4 DISCUSSION

The analysis of existing methods for determining R_{comp} reveals distinct applicability limitations based on soil types. The Dvorkin-Sorochan method exclusively addresses sandy soils, while Konovalov-Roitman and Polyakov-Sokolov approaches extend to both sandy and low-plasticity clayey soils ($J_L \leq 0.5$) with ≥15 years service life. The Zurnadzhi-Filatova method provides only preliminary estimates. The most comprehensive TGASU (1997-2000) method covers clayey soils across consistency ranges (hard to soft-plastic) and uniquely accounts for both compaction-induced strengthening (45-60% R increase for $J_L \leq 0.5$ clays) and water-induced weakening (20-25% increase when flooded). The method demonstrates that when operational pressures are low ($p/R \leq 0.6$-0.7), resistance may decrease by 10-15%, while maximum compaction benefits occur at $p \geq 0.8R$ [10]. These findings emphasize the necessity of considering both soil-specific characteristics and loading history when evaluating foundation capacity for reconstruction projects, particularly for structures with long service periods where soil properties may have significantly evolved. The approach of the authors provides the most complete framework for such assessments, incorporating both beneficial and detrimental long-term soil modifications.

5 CONCLUSION

The design of foundations for reconstructed buildings requires accurate determination of the compacted soil resistance (R_{comp}), building upon standard calculations (SP 22.13330.2016) while accounting for soil property changes during operation. Current methods, though effective for specific soil types like sands and clays, show limitations in scope. The TSASU-developed approach stands out by comprehensively addressing both soil strengthening from compaction and potential weakening from flooding, applicable across clayey soils of varying consistency from hard to plastic states.

The developed approach provides more accurate predictions of bearing capacity by accounting for both beneficial compaction effects and potential weakening from saturation, offering practical improvements over conventional design methods.

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Method of construction of bored-injection piles with increased bearing capacity for foundations in seismic areas

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ABSTRACT

The advancement of bored-injection pile technology has significantly expanded their application in seismic regions and complex geotechnical conditions. These piles are formed by pressure-injecting fine-grained concrete through hollow rods equipped with perforated drill bits. However, conventional designs exhibit limitations in sandy clay soils, particularly regarding reduced horizontal load-bearing capacity and crack resistance due to the absence of comprehensive reinforcement, relying instead on a central small-diameter (≤103 mm) hollow rod. To address these limitations, an enhanced design incorporates a hollow metal pipe reinforcement element (0.3-0.4 m diameter) in the upper section, connected to the lower rod via a screw-type transition coupling. Following installation, the pipe is filled with concrete and includes reinforcement bars to establish rigid connections with the superstructure's foundation elements. This modification significantly improves resistance to bending deformations while ensuring adequate load-bearing capacity under seismic conditions, effectively overcoming the critical limitations of conventional bored-injection piles in challenging geotechnical environments.

Keywords: Bored-injection pile, seismic areas, pile foundation, increased bearing capacity, complex engineering-geological conditions.

1 INTRODUCTION

Recent advancements in foundation engineering have focused on improving bored-injection pile technology for challenging geotechnical conditions, particularly in seismic regions [1, 2]. The development of Titan micropiles (TMP) by Ischebeck in the 1980s marked significant progress, introducing small-diameter piles (≤300 mm) capable of bearing substantial loads with minimal soil disturbance. These piles, formed by pressure-injecting fine-grained concrete through hollow rods with perforated drill bits, demonstrate effective vertical load transfer but exhibit limitations in horizontal load-bearing capacity, especially in weak, seismically active soils. This study aimed to enhance pile performance in silty-clayey soils through innovative design modifications, including a reinforced upper section with hollow metal pipes (0.3-0.4 m diameter) connected to the central rod via transition couplings [3]. The research methodology encompassed experimental investigations of full-scale piles under compressive and horizontal loads, with particular attention to soil-structure interaction mechanisms. The developed solutions were subsequently implemented in construction and reconstruction projects across various seismic-prone clay soil conditions, demonstrating improved seismic resistance while maintaining the technology's inherent advantages of minimal vibration during installation and efficient load transfer.

2 MATERIALS AND METHODS

Extensive field testing has confirmed the high compressive bearing capacity of Titan micropiles (TMP), while revealing limitations in horizontal and seismic load resistance, prompting design improvements [4, 5]. One enhanced method involves constructing an enlarged upper pile section (4-6 m long, >1.0 m diameter) with inclined hollow tubes (2-4° angle) for inserting extended reinforcement rods, demonstrating improved load-bearing capacity through numerical analysis, though requiring complex installation procedures. The more advanced

solution incorporates specially designed reinforced heads, offering multiple advantages including increased bearing capacity (4-4.5 times greater horizontal and 20-25% greater vertical resistance), improved crack resistance, and simplified construction technology. Numerical modeling within a 10×10×15 m domain compared performance of 9 m long piles with varying head lengths (1-3 m) and standard headless piles under vertical and horizontal loading. Full-scale testing in silty-clay soils characteristic of seismically active southern Russia validated these findings, following a comprehensive installation protocol: initial 350 mm pilot drilling, pile placement with different head configurations, and load testing after 28-day concrete curing. The geological profile consisted of layered silty clays and water-saturated sands, typical of the region's challenging construction conditions. This systematic approach confirmed the effectiveness of the reinforced head design for seismic applications while maintaining construction efficiency.

3 RESULTS

Experimental studies were conducted on full-scale piles with specially designed heads under vertical and horizontal loading conditions in accordance with GOST 5686-2020 [6]. Four pile configurations were tested (without head, 1 m head, 2 m head 3 m head) with deformation stabilization criteria set at ≤0.1 mm settlement over 120 minutes per loading stage. The results demonstrated strong correlation between numerical modeling predictions and empirical load-settlement behavior for all configurations, validating the model's accuracy in predicting pile performance under both vertical and horizontal loading conditions. The high degree of convergence between experimental and numerical data confirms the reliability of the developed design solutions for practical engineering applications in seismic regions.

4 DISCUSSION

The study established patterns of interaction between reinforced bored piles and soil, though final validation requires implementation in seismic areas with full-scale testing. During construction in Sochi, analysis revealed significant horizontal forces in corner piles, necessitating reinforced pile heads. Preliminary calculations showed insufficient bearing capacity (design pressure σ_{z1}=101.2 kPa exceeding ultimate $\sigma_{z,ult1}$=70,3 kPa), prompting replacement of 12 corner piles with specially designed heads (1500 mm length, 273 mm diameter). Post-installation testing after 28-day curing demonstrated the reinforced heads increased compressive capacity by 30% and horizontal resistance fivefold, validating the design's effectiveness. The technical solution employed a comprehensive analytical approach incorporating soil-structure interaction parameters (deformation coefficients, bending moments, lateral forces) while considering geotechnical characteristics (internal friction angle, soil cohesion, water saturation effects), ultimately achieving stable performance in challenging seismic conditions.

5 CONCLUSION

The research developed innovative methods to enhance the bearing capacity of bored-injection piles for building foundations in seismic regions through strategic use of reinforced metal elements. The proposed design incorporates a hollow metal pipe (0.3-0.4 m diameter) in the pile's upper section, positioned 1-2 meters below weak soil layers to minimize bending deformations while optimizing load transfer. Comprehensive 3D numerical modeling established precise correlations between head dimensions and load-bearing performance in silty-clay soils. Experimental verification demonstrated remarkable improvements - horizontal load resistance increased 4-4.5 times and vertical capacity by 20-25% for head lengths of 1-3 meters. These advancements enable significant reductions in required pile quantities while maintaining structural safety, offering both economic and construction efficiency benefits. Field implementation in various seismic-prone regions with silty-clay soils confirmed the technical solutions' reliability, with test results showing consistent agreement between predicted and actual performance under both vertical and horizontal loading conditions. The developed methodology represents a substantial advancement in foundation engineering for challenging geotechnical environments, effectively addressing the critical need for improved seismic resistance in weak soil conditions.

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Case Study on Conservation and Renewal of Historical heritage buildings in Albania

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ABSTRACT

Historical buildings with certain preservation values can reflect a city's history and cultural characteristics and often civilizations. Traditionally historical preservation has dealt with architectural embellishment, but this paper emphasizes the importance of including complex disciplines and sustainable development concepts. The paper analyzes the protection and renewal process of a historical wall in Durres City after a seismic event in 2019. The aim was to preserve the original appearance and demonstrate the authenticity and integrity of this ancient wall. Therefore, appropriate conservation techniques were employed to minimize intervention and prolong the lifespan of the ancient city wall, while embodying sustainable development principles. Before interventions through the construction process the following investigations were conducted such as: desk studies with all historical and geological background studies, design model constitution, fieldwork, laboratory testing. Field works included geo-seismic study utilizing geophysical methods, boring, sampling, and monitoring during all construction stages and after the finalization of the project. Meanwhile, geophysical methods included GPR measurements where a major geotechnical risk was detected and prevented. Laboratory testing determined all the needed physical and mechanical characteristics of the soils, and construction materials needed to compute the geotechnical models for design purposes, and the monitoring process included the use of crack matters and other noninvasive tests to measure and prevent further collapses of the wall. The paper describes the geo-engineering aspect of the investigation and its findings.
Keywords: conservation, cultural heritage, geotechnical engineering, intervention, stability,

1 INTRODUCTION

The paper describes the intervention works done for the renewal of a historical wall in Durres city, built from the 3rd century to the 5th century AD to protect the city from invasions. The ancient walls nowadays attract many tourists each year.

The Venetian Tower (used as a weaponry magazine for the army and a base for guarding the port and the bay of Durres), is one of the main towers that compose the walls of the castle. The interior is built with bricks.

Tower C is located 200 meters from the Venetian Tower, near the main road that brings to the hill of King Zog's villa, and few meters from Durres Amphitheatre. (see Fig.1)

After the seismic event of the 26th of November 2019, tower C has been under a long restauration process. In 2019 CHwB-Albania, through the financial support of Cultural Preservation Fund (AFCP), Swedish International Development Cooperation Agency (SIDA) and the scientific support of World Monuments Fund and Ministry of Culture, began the restoration process that follows three phases of activities:

- 1st phase – Immediate intervention to ensure short term stability
- 2nd phase – Proposals of Design project for conservation measures
- 3 – Implementation of the agreed conservation measures



Fig. 1. The citadel of Durres (today is known as the neighborhood "Kala".



Fig. 2. Damage scale of Tower C, picture taken on Nov.27th, 2019.



Fig. 3. Immediate strengthening intervention at Tower C, after the earthquake of Nov. 2019. Installation of stainless-steel ties to improve the global behavior of the structure.



Fig. 4. During geological work at Tower C.

2 GEOLOGICAL STUDY BACKGROUND

To design the conservation and renewal project of Tower C, a series of studies were conducted as follows: geological study; geophysical measurements; testing of construction materials; archaeological survey, and seismic analysis through kinematics and numerical calculations applied to define the structure's capacity.

The purpose of the geological study was to determine the physical-mechanical features of construction material, the soils, and rocks layers encountered at "Tower C", to obtain a clear picture of what caused the collapse and compile a geotechnical model, for the tower to withstand further challenges. **3.1 Summary of geological engineering study**

- Geological mapping.
- Desk study.
- Drillings (6.00-20.00 m) depth, (vertical and inclined).
- SPT field tests were carried out according to the ASTM D 1586-11 methodology.
- Georadar measurements to access the foundation area and possible archaeological artifacts.
- Granulometric analyses according to the ASTM D6913-04(2009) and ASTM D 7928-16e1 methodologies.
- Atterberg Limits analyses according to the ASTM D 4318-10 methodology.
- Shear Test tests were carried out according to the ASTM D 3080/3080-11 methodology.
- Oedometric tests were carried out according to the ASTM D 2435/2435 M-11 methodology.
- Geophysical measurements interpretation report.
- The final geotechnical report, including all the findings and results from field, laboratory testing and engineering judgment.

3 CONCLUSIONS

In the site under study, Quaternary and Neogene deposits are found.

The area where “Tower C” is located, has complex geological-engineering conditions, due to the presence of deposits with poor to moderate physical-mechanical characteristics.

The FEM model of relative movement was conducted due to differential settlements, to compare the existing cracks in the tower with the ones generated by the numerical model.

The result gave a clear picture of the collapse causes; the cavity in the foundation led to the development of vertical cracks, rotation of detached block, large deformation, while the earthquake caused the final overturning of the wall.

The conservation and strengthening design proposals were based on the interpretation of the results from the analysis.

These proposals consist in the consolidation and repair of the foundation wall through traditional techniques, rebuilding of the collapsed wall with the fallen blocks, and cleaning and repairing of the wall to restore Tower “C”.

Monitoring: necessary during and after the restoration works to evaluate the effectiveness of the engineering measures.

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Investigation of Microstructural Changes in Soil During Modification

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ABSTRACT

Soil modification techniques are widely employed to enhance the engineering performance of soils in geotechnical applications. Among these, biopolymer-based treatments have gained significant attention due to their environmentally friendly nature and effectiveness in improving key properties such as strength, cohesion, and erosion resistance. Biopolymers like chitosan, xanthan gum, and guar gum interact with soil particles to form a reinforced matrix that increases mechanical stability while maintaining permeability. This study focuses on the microstructural changes induced by biopolymer treatments, with particular emphasis on scanning electron microscopy (SEM) analysis. SEM provides high-resolution visualization of particle arrangement, pore structure, and mineral composition, offering insights into the bonding mechanisms at the microscale. The experimental results reveal that biopolymers promote strong interparticle bonding, reduce pore sizes, and contribute to the formation of a more compact and stable soil matrix. The study also highlights the critical role of standardized sample preparation in ensuring the accuracy and reproducibility of SEM-based microstructural analysis. These findings support the potential of biopolymer-based modification as a sustainable and effective soil stabilization approach, offering practical benefits for geotechnical engineering design and construction.

Keywords: soil modification, biopolymers, microstructural analysis, scanning electron microscopy pore structure, particle bonding.

1 INTRODUCTION

A comprehensive set of methods is used to study the properties of modified soils, aimed at determining their strength, deformation, and filtration characteristics. Laboratory tests, such as compression and shear tests, provide quantitative assessments of changes in physical and mechanical properties [1]. Geophysical methods allow the identification of heterogeneities and monitoring of modification effectiveness under field conditions. Chemical analysis is applied to evaluate interactions between modifying additives and soil particles.

Microscopic examination, particularly scanning electron microscopy (SEM), plays an important role, enabling high-resolution visualization of the soil structure at the micro level and revealing changes in particle bonding and pore structure. In addition, numerical modeling is used to predict the behavior of modified soils in engineering applications.

The microstructure of soil plays a crucial role in shaping its properties after modification. The nature of microstructural bonds determines the strength, deformation resistance, and durability of the material. Changes in porosity, particle orientation, and distribution affect filtration properties and the bearing capacity of the soil. During modification, especially with biopolymer-based treatments such as xanthan gum, chitosan, and guar gum, new bonds are formed between particles, creating a denser and more cohesive matrix.

Chitosan, derived from crustacean shells, offers excellent adhesive and coagulating properties, while guar gum contributes to structural reinforcement through gel formation. SEM analysis provides detailed insight into these changes, showing the formation of binding matrices and shifts in mineral composition.Standardization of sample preparation for SEM is emphasized to ensure reproducibility and reliability in comparing various treatment methods.

2. METHODS AND MATERIALS

Sand with fine friction (Astana, Kazakhstan) was employed for the experiment. Sieve, hydrometer, liquid, and plastic limit analyses were conducted to classify the soil. The liquid limit and plastic limit analyses were performed to assess the soil's consistency and plasticity characteristics. These tests allowed for a comprehensive understanding of the soil's texture and behavior, which is essential for evaluating its suitability for reinforcement. Physical characteristics of soil are presented in Table 1.

Soil composition contains 60.794% sand, 19.193% silt and 15.609% clay (Table 1).

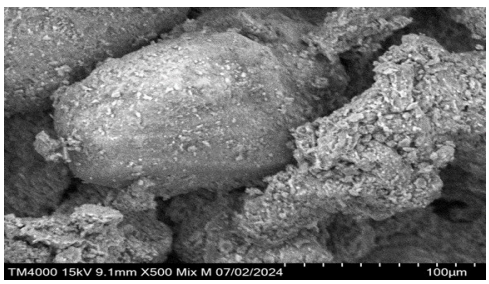
Table 1. Physical characteristics of the soil [2].

Soil characteristic	Value
Specific gravity, g/cm3	2.538
Maximum dry density, g/cm3	2.031
Optimum water content, %	10.194
Sand-sized fraction (75 μm–2 mm), %	60.794
Silt-sized fraction (5–75 μm), %	19.193
Clay sized fraction (<5 μm), %	15.607
Liquid limit, LL, %	23.251
Plastic limit, PL, %	1.190

The modified soil was prepared with sand, biopolymer and water (wet mixing) which was carefully inspected and weighed on a scale before being mixed. The proportion was 231.48:5:8 grams respectively. First, the biopolymer was mixed with water and mixed thoroughly until a homogeneous mass was obtained. The process involves carefully measuring the appropriate amount of biopolymer and water and then combining them in a mixing container. The mixture was stirred continuously to ensure that the biopolymer was fully dissolved or dispersed in the water, achieving a uniform consistency. After achieving a homogeneous mixture, the solution stabilized for 1 hour. This stabilization period is essential for ensuring the even distribution of biopolymer throughout the mixture. During this time, the biopolymer particles continue to interact with the water, allowing them to fully integrate and achieve the desired consistency. Stabilization helps in reducing any potential clumping or settling of the biopolymer particles, which could otherwise lead to inconsistent application and ineffective soil treatment. Once the stabilization period was complete, the mixture was applied to the soil. The wet method allowed easier control of the viscosity and flowability of the mixture, which simplified the mixing process.

3 RESULTS AND DISCUSSION

SEM image analysis shows (Fig. 1) that the structure of the polymer soil is modified, and the gum interacts with the soil particles to form a kind of film. The soil without polymer has a more homogeneous texture. The biopolymer films filled the pores and coated the soil grains, which facilitated the joining of neighboring particles through several mechanisms.



a)



b)

Fig. 1. SEM image: a – soil before treatment; b – soil after treatment [2].

These mechanisms include:

- adhesion: the biopolymer films adhered to the soil particles due to the attraction forces between the polymer and the soil surface;
- cohesion: within the biopolymer film itself, cohesive forces (forces of attraction between similar molecules) hold the polymer chains together and contribute to the integrity of the film and its ability to bridge gaps between soil particles;
- mechanical binding: as the biopolymer film dried and set, a rigid network was formed that mechanically bonded to soil particles, increasing the soil's resistance to breakdown.

4 CONCLUSIONS

The key conclusions drawn from this research can be summarized as follows:

- When a homogeneous mixture of gum and soil was prepared, uniform distribution of the polymer agent was ensured. If the mixture is not uniform, the reinforcing agent may concentrate in certain areas, resulting in uneven reinforcement of the soil. This can lead to localized weak spots that can reduce the overall strength and stability of the structure.
- In the quality control of the modified soil, SEM analysis was conducted to assess the extent of interaction between the gum and soil particles. This analysis provided insights into the effectiveness of the gum in consolidating the soil and the changes in soil structure at the microscopic level. The SEM images demonstrated the homogeneity of the consolidated soil, indicating that no adjustments to the mixture composition were necessary.

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Influence of pH on Biopolymer Activity and Risk Assessment in Soil Modification

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ABSTRACT

Weak soil requires stabilization to enhance load-bearing capacity and moisture retention. Various methods are used, including mechanical compaction, mineral binder treatments, and chemical modification. Mechanical methods, such as dynamic compaction and vibroflotation, increase soil density by reducing voids. Chemical stabilization with cement, lime, or fly ash reacts with soil minerals to form cement, but it can harm the environment, leading to interest in biopolymers as an alternative. Biopolymers improve soil structure and hydrophysical properties in an environmentally friendly manner. Their effectiveness depends on pH, which affects interactions between polymers and soil particles, altering moisture retention and strength. This study investigates the treatment of soil with chitosan and xanthan gum, focusing on how pH affects their solubility and activity, and how these biopolymers alter soil properties. The first stage examines the preparation and mixing of biopolymer-treated soil, analyzing texture to identify optimal solubility conditions. The second stage evaluates the modified soil's behavior under wetting conditions, as moisture directly impacts physical and mechanical properties. The analysis revealed that both moisture level and acidity significantly influence the soil's structure. Modified soils showed improved performance compared to the control sample, with chitosan-reinforced soil showing increased density. Additionally, potential risks during the preparation of the modified soil will be assessed, including improper mixing of biopolymers, pH instability, and contamination of samples. Measures will be taken to minimize these risks to ensure the reliability of results.

Keywords: pH influence, soil moisture retention, mechanical compaction, chemical stabilization

1 INTRODUCTION

Soil stabilization is crucial in geotechnical engineering, especially for weak or collapsible soils. Enhancing soil performance improves structural reliability and reduces deformation risks, particularly in areas with fluctuating groundwater or heavy rainfall. Stabilized soils offer better load-bearing capacity, lower permeability, and increased durability [1]. Conventional methods, such as mechanical compaction and chemical treatments, are effective but may have environmental impacts and long-term risks to groundwater and ecosystems. In response to these limitations, recent research has focused on the use of natural biopolymers as sustainable alternatives for soil stabilization [2].

Biopolymers can enhance cohesion, reduce erodibility, and improve the overall structure of treated soils. Their biodegradability and low environmental impact make them suitable for applications in ecologically constrained environments. The performance of biopolymers in soils depends on factors such as concentration, application method, and environmental conditions, including moisture and pH. Chitosan and guar gum are biopolymers studied for soil improvement. Chitosan, derived from chitin, enhances soil aggregation through film formation and ion exchange. Guar gum is known for its high viscosity and water retention. Both polymers interact with soil through physicochemical mechanisms influenced by pH, making them suitable for stabilization where conventional methods are less effective [3].

2 METHODS AND MATERIALS

The investigations were conducted in the "ENU-Lab" laboratory of L.N. Gumilyov Eurasian National University, Republic of Kazakhstan. The experiment's technical process consisted of the following main procedures: test setup and equipment calibration; determination of physical and mechanical properties of soil; preparing solutions with different pH; preparing modified soil; measurement of the pH level of soil and the biopolymer soil; evaluation of the behavior of modified soil samples under humid conditions; scanning electron microscopy analyses of soil and the biopolymer soil; analysis exploring the interrelationship between the results received; identification of experimental risks: analysis of potential sources of error risk mitigation measures: development of control and error prevention strategies. Sand with fine friction (Astana, Kazakhstan) was employed for the experiment. Sieve, hydrometer, liquid, and plastic limit analyses were conducted to classify the soil (Table 1). Process of soil mixing with

biopolymers presented in Fig.1.

Table 1. Physical characteristics of the soil [4].

Soil characteristic	Value
Specific gravity, g/cm3	2.538
Maximum dry density, g/cm3	2.031
Optimum water content, %	10.194
Sand-sized fraction (75 µm–2 mm), %	60.794
Silt-sized fraction (5–75 µm), %	19.193
Clay sized fraction (<5 µm), %	15.607
Liquid limit, LL, %	23.251
Plastic limit, PL, %	1.190



Fig. 1. Process of soil mixing with biopolymers.

3 RESULTS AND DISCUSSION

The data indicates that the control sample's weight decreased from 105 grams to 103 grams upon moistening, suggesting structural loosening. In contrast, the chitosan-modified soil sample's weight increased from 105 grams to 145 grams, indicating enhanced moisture retention due to improved soil structure and increased porosity. Similarly, the xanthan gum-modified soil sample's weight rose from 105 grams to 130.6 grams, reflecting improved moisture retention and structural characteristics. Figure 2 presents the values of the modified specimens' water absorption coefficient.

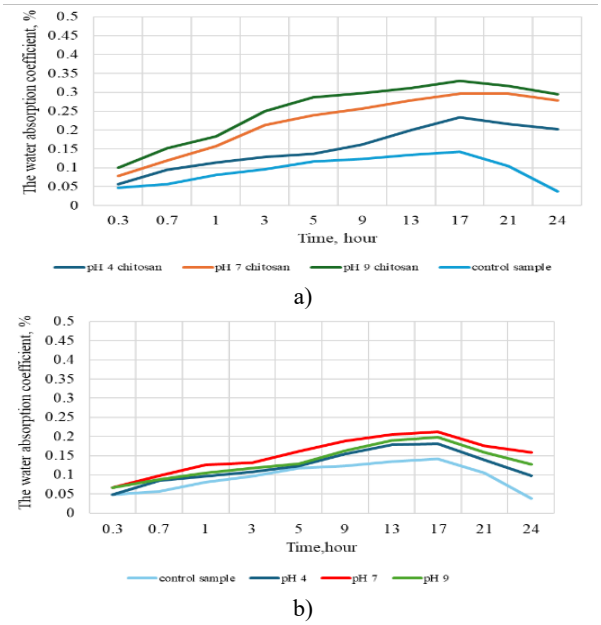


Fig. 1. Graphs of water absorption coefficient [4].

Table 2 presents risks that can be conducted during the experiment for the modified soil.

Table 2. Risks.

Method	Risks
Preparation of Solutions with Different pH	Errors in pH solution preparation
	Health risks when working with acids or alkalis
	Instability of biopolymers with changing pH
Preparation of Modified Soil	Improper mixing of polymer and soil
	Formation of clumps or gels
	Uneven polymer distribution
Measurement of pH in Soil and Biopolymer Soil	pH changes due to interaction with soil minerals
	Degradation of biopolymers and pH change over time
Evaluation of Modified Soil's Behavior Under Wet Conditions	Excessive or insufficient moisture
	Moisture impact on long-term stability

4 CONCLUSIONS

The study revealed the following findings:

1. Biopolymers demonstrated good compatibility with soil, and homogeneous structure.

2. The treated soil exhibited the best water absorption results in the following sequence using biopolymers: chitosan with alkaline medium base solution, chitosan with neutral medium, chitosan with acidic medium, gum with neutral medium, gum with alkaline medium, and gum with acidic medium.

3. The soil density increased compared to the control sample as follows: chitosan with alkaline medium 16%, chitosan with neutral medium 12%, chitosan with acidic medium 7%, gum with neutral medium 6%, gum with alkaline medium 4%, and gum with acidic medium 3%.

4. Soil sample reinforced with xanthan gum in a solution with pH 4 was most susceptible to shape change and destruction.

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Cyber-Physical Systems and Digital Twins to Eliminate Tilts of Multi-Story Buildings

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ABSTRACT

The aim of the report is to explain a new information technology for diagnosing the current state of multi-story buildings to eliminate its excessive tilting while simultaneously ensuring reliable operation throughout the alignment process without interrupting its engineering networks, technological equipment (elevators, etc.), and residents' relocation. This will enhance the quality of forecasts for changes in the technical condition of multi-story buildings during controlled elimination of excessive tilting and enable the responsible decision-making in conditions of uncertainty and the risk of construction accidents. For the first time, the new information technology for diagnosing the current state of multi-story buildings has been developed for tilting elimination under conditions of constant change in the building's vertical position, physical-mechanical properties of foundation soils, and materials of individual building structures (crack formation) throughout the alignment process and for some time after its completion without interrupting the functioning of the building's engineering networks, technological equipment, and residents' relocation.

Keywords: tilt, inclinometric sensors, risk of construction accidents, non-destructive methods, processing and transmission of experimental data using modern cloud technologies in real-time.

1 INTRODUCTION

In the paper the practical application of the Digital Twin technology (DTT) at the building's reconstruction stage (eliminating an excessive tilting of more than 10 aging multi-storey buildings (MB) in the city of Zaporizhzhia in Ukraine) is examined. The buildings and structures uneven settlements depend on a wide range of objective and subjective factors. The objective factors may include the base soil compositions, e. g., in Zaporizhzhia the loessial soils are prevailing. They are sensitive to moistening, which results in the uneven degradation of their bearing capacities. The subjective factors include the ruptures and malfunctions in the cold and hot water supply systems, as well as in the sewage disposal systems in buildings. Therefore, the systemic cyclic actions arise, i. e., the uneven settlements lead to the utilities network ruptures, and leakage from a network leads to the new uneven settlements and construction accidents and as a result – loss of their resilience. The MB (structure) tilt is one of the criteria determining the building safe operation possibility. However, in many cases at various stages of the construction object lifecycle the tilt exceeds the permissible normative value and sometimes even the limit value. The DTT for the excessive tilts elimination includes three stages as follows: (1) a MB digital model taking into account the building geometry changes (a damage or an absence of structural elements); (2) a digital shadow, which includes the refined MB digital model with taking into account the actual physical and mechanical properties (PMP) of the base soil and superstructure construction materials obtained by means of sensors, which are the non-destructive testing devices; (3) a MB digital twin (DT) including the inclinometer sensors distributed over the MB and the system for collecting and storing information, which is transmitted due to the Internet of Things (IoT), is subject to the operational processing and directly performs the straightening process operational control (feedback). At the DT final third stage, the information exchange in two directions between the MB physical entity and its digital model is implemented. Moreover, based on the change of the physical nature data the digital model is adjusted and the physical entity is adjusted vice versa. The MB digital model calculation data are used for making the managerial decisions that change the physical nature of the MB base and foundation soils during the multi-day process of the MB straightening. As a result, the DTT is implemented for the MB tilt elimination in conditions of building vertical position permanent changes and PMP changes in the base soils and individual building

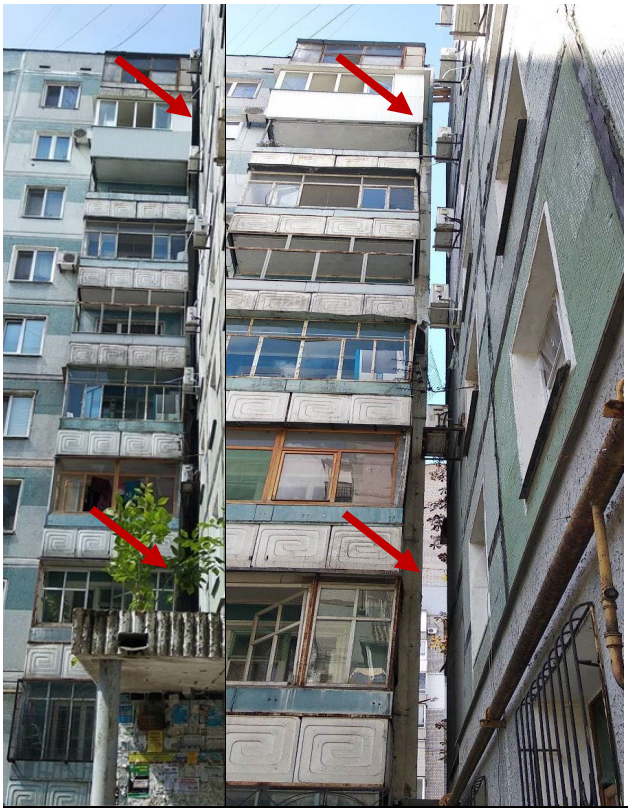
structural materials (cracking) without the MB and technological equipment operation stopping and the inhabitants resettlement throughout the straightening period and some time after its completion.

2 THE TECHNOLOGY FOR THE TILT ELIMINATION BY THE BUILDING BASE SOIL DRILLING UNDER A LESS SETTLED FOUNDATION.

The most researched and implemented method of affecting the foundation-superstructure subsystem usually involves the superstructure lifting with a piston or flat jacks installed in the special niches in the building foundation-basement part and combined into modular systems or lowering it with an active structural systems located in the base load-bearing elements. But this method has a significant drawback because the significant concentrated efforts affect the building structures, which causes a number of difficulties in the method application. For instance, a need can arise to significantly strengthen the building foundation-basement part or to divide it into the foundation and above-foundation parts by cutting them, etc. In the Zaporizhzhia branch of The State Research Institute of Building Constructions State Enterprise the method is developed for straightening the buildings and structures by means of the foundation rigidity adjustable changes by partially removing the excess soil from under the foundation part, which has settled less during operation. It is proposed to remove soil from under the less settled foundation part by controlled drilling the horizontal parallel wells with variable design parameters. The straightening planning and execution require meeting of two conditions. Firstly, the excavated soil volume must be equal to the volume of the building foundations settlements spatial diagram; secondly, the diagram of uneven settlement specified for the building must be linear for preventing the building structures deformations increase. The technologies are developed for straightening the buildings tilted in longitudinal, transverse and complex directions. To drill the horizontal wells under the foundations, the number of wells rows in height is calculated, that is, the thickness of the base layer, in which the perforation should be performed, is determined. The necessity of a retaining wall creation for the foundations to ensure their stability and strengthen the future slope of the foundation pit is determined by calculations; and before the pit manufacture, if necessary, the vertical reinforcement using the soil mixing technology is performed. The pit is dug from the side of the less settled part of the building; at the bottom of the pit the small-sized horizontal drilling machines developed by the authors are installed, and the base layer is weakened by drilling horizontal wells of variable cross-sections. Under the building pressure, the soil pillars between the wells and in their vaults are destroyed, and the perforated base layer is unevenly compressed.

~~63~~ THE RESULTS OF THE DTT USE FOR THE MB EXCESSIVE TILT ELIMINATION

The appearance of the expansion joint between the block-sections No. 3 of the residential building No. 23 and No. 1 of the residential building No. 25 before and after the excessive tilts elimination is shown in Fig. 1. The plan and elevation position of the block-section No. 1, after its straightening, meets the design requirements and the current building regulations of Ukraine. The block-section corners deviation is within 9...96 mm and the tilt is 0.0004...0.004.



a) b)

Fig. 1. General view of the expansion joint between block sections No. 3 and No. 1 before (a) and after (b) straightening at 23 and 25 Engineer Preobrazhenskyi Avenue in the city of Zaporizhzhia.

The DTT development was carried out for the MB tilt elimination in conditions of the permanent changes of the building vertical position, base soils PMP and certain civil structures materials (cracking) properties throughout the time of straightening without the MB operation stopping and inhabitants resettlement, as well as some time after the straightening completion for restoration it resilience. The improvement of the DTT mathematical and algorithmic apparatus, which included the buildings and base soils mathematical models used in the MB straightening process and the mathematical model of soil-binder mixtures for the buildings foundations stabilization after the straightening process completion, was carried out.

Engineering Challenges of Tropical Soil Hazards: Disintegration, Weathering, and Shear Failure

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ABSTRACT

Tropical soils in Southeast Asia pose significant geotechnical challenges due to their unique mineralogy, intense weathering, and climatic conditions. This paper synthesises the impacts of weathering, disintegration, and shear failure through field investigations and laboratory testing. Key findings indicate that heavy rainfall, moisture fluctuations, and rapid urbanisation significantly elevate slope failure risks. The study emphasises the need for improved site characterisation and integrated use of Cone Penetration Testing with pore pressure measurement (CPTu) and Multichannel Analysis of Surface Waves (MASW). Specialised strategies are essential to address the vulnerabilities of tropical soils, supporting sustainable engineering practices.

Keywords: Geohazards, Site Characterisation, Slope Failure, Weathering, Tropical Soils

1 INTRODUCTION

Tropical regions, particularly in Southeast Asia, feature deeply weathered soils with complex mineral compositions and highly variable geotechnical properties, which increase the risks of slope failure and foundation instability (Alatas et al., 2016; Gobena & Suppiah, 2018). Intense rainfall, rapid urbanisation, and inadequate site characterisation further exacerbate these hazards. Conventional geotechnical frameworks, originally developed for temperate zones, often fail to address the distinctive behaviours of tropical soils (Tjia, 1996; Toll, 1990). Although Cone Penetration Testing with pore pressure (CPTu) and Multichannel Analysis of Surface Waves (MASW) are increasingly utilised, inconsistencies in classification standards remain a challenge (Zainorabidin et al., 2023; Zhao et al., 2022). This study evaluates geohazard mechanisms and field characterisation methods to enhance design reliability.

2 QUANTITATIVE ASSESSMENT OF TROPICAL SOILS

Field characterisation remains critical for understanding the geotechnical behaviour of soft ground and weathered clay shales. Tropical soils often exhibit complex behaviour, such as disintegration, swelling, and strength degradation, under fluctuating moisture conditions (Alatas et al., 2016; Gobena & Suppiah, 2018).

Advanced in-situ testing methods, notably CPTu and MASW, provide improved characterisation but face challenges in fibrous soils and deeper formations (Zhao et al., 2022; Zainorabidin et al., 2023). The inadequacy of traditional laboratory testing highlights the need for integrated, site-specific assessment frameworks tailored to tropical environments.

3 EVALUATION OF TROPICAL SOIL BEHAVIOUR

Studies reveal significant gaps in quantitative benchmarking of tropical soil properties, with limited reports on shear strength and disintegration behaviours. Disintegration ratios (DR) and undrained shear strength (S_u) emerge as critical indicators of soil vulnerability.

3.1 Shear Strength and Weathering Effects

Alatas et al. (2016) found that Semarang-Bawen clay shales completely disintegrated upon saturation, showing substantial shear strength reduction. Typical undrained shear strengths (S_u) for tropical clays range between 10 to 40 kPa (Toll, 1990; Vanapalli & Fredlund, 2000).

3.2 Disintegration Behaviour

Disintegration ratio (DR) studies indicate that soils with a DR greater than 80% are highly weathered and prone to instability (Pinto, 2002). Disintegration testing should be a standard component of geotechnical assessments for tropical soils.

4 IN-SITU CHARACTERISATION METHODS

This section introduces CPTu and MASW methods, highlighting their roles in characterising tropical soils, improving subsurface profiling accuracy, and reducing uncertainties in geotechnical hazard identification and foundation design.

4.1 Cone Penetration Testing with Pore Pressure (CPTu)

CPTu measures cone tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u_2). These parameters help determine stratigraphy, relative density, undrained shear strength, and stress history (Zhao et al., 2022).

4.2 CPTu Insights

Low cone resistance (q_c < 2 MPa) signifies highly soft soils, such as Malaysian hemic peat (Zainorabidin et al., 2023). This reinforces the importance of accurate q_c profiling in foundation design.

4.3 Multichannel Analysis of Surface Waves (MASW)

MASW estimates shear wave velocity (V_s) by analysing surface wave dispersion, offering valuable data on subsurface stiffness (Zhao et al., 2022).

4.4 MASW Insights

MASW complements CPTu findings by identifying stiffness transitions. V_s values <150 m/s typically indicate incredibly soft soils (Kramer, 1996).

4.5 Integrated Methodology

Combining CPTu and MASW enhances hazard identification, reducing uncertainties.

4.6 Comparative Methodology Review

A comparative analysis is summarised in Table 1.

Table 1. Comparative Overview of CPTu and MASW Methods for Tropical Soil Characterisation.

Method	Primary Output	Best Use	Limitation
CPTu	Stratigraphy, shear strength, pore pressure profiles	Clayey soils, identifying soft zones	Poor in fibrous soils; equipment sensitivity
MASW	Shear stiffness profile (V_s)	Mapping stiffness contrasts, shallow profiling	Limited penetration depth; noise sensitivity

5 KEY INSIGHTS AND INTERPRETATION CHALLENGES

Weathering intensity and moisture sensitivity dominate hazardous behaviours. Integrated CPTu-MASW approaches enhance hazard mapping but require better standardisation. Existing classification systems often misrepresent the characteristics of tropical soils (Toll, 1990; Tjia, 1996).

6 RESULTS DISCUSSIONS

Tropical soils exhibit rapid strength loss and disintegration under wet conditions (Alatas et al., 2016; Vanapalli & Fredlund, 2000). CPTu and MASW effectively detect problematic layers, improving geotechnical modelling and hazard prediction (Zhao et al., 2022; Zainorabidin et al., 2023). Nevertheless, a significant need persists for tropical-specific classification frameworks that address lateralisation, anisotropy, and weathering-induced transformations.

7 CONCLUSIONS

Tropical soils are highly vulnerable to strength degradation and disintegration due to intense weathering and climatic influences. Integrating CPTu and MASW improves subsurface profiling and hazard identification. Engineering practices must adopt tropical-specific standards for reliable design and construction in these challenging environments.

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Hybrid Concept of Digital Twins and Cyber-Physical Systems in Dense Urban Environments

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ABSTRACT

A hybrid theoretical-methodological concept of digital twins and cyber-physical systems in dense urban environments (concept) has been proposed. As an example of the implementation of the concept, a comprehensive study has been carried out to scientifically substantiate and support the transformation of the Kyiv metro station "Heroes of Dnipro" into a complex building structure consisting of two parts: the modified metro station itself and the shopping and entertainment center "Oasis" built directly above it. In particular, Digital Twins of the existing and transformed metro station, along with the multi-story shopping and entertainment complex, were developed. For carrying out such a transformation, a visual inspection of the station's building structures was performed, documenting cracks and damages, and determining necessary reinforcements; vibration studies of the soil near the shopping and entertainment complex under the influence of metro trains and road traffic were conducted.

Keywords: Hybrid concept, dense urban development, cyber-physical systems, digital twins, modification of structures

1 INTRODUCTION

The purpose of the work is to describe the research for the scientific justification and support of the transformation of the Kyiv Metro station "Heroes of Dnipro" into a complex construction consisting of two parts: the modified metro station itself and the shopping and entertainment center built directly above it (Fig. 1 and Fig. 2). In particular, Digital Twins of the existing and transformed metro station, along with the multi-story SEC, were developed.



Fig. 1. View of the "Heroes of Dnipro" metro station in 1980.

For the implementation of such a transformation, the following scope of work was carried out directly at the operational "Heroes of Dnipro" metro station: visual inspection of the station's building structures with documentation of cracks and damages; selective determination of the station's concrete structure strength using non-destructive methods; necessary reinforcement of the structures; study of ground vibrations near the SEC caused by metro trains and vehicles; vibrodynamic inspections of the platform structures at the "Heroes of Dnipro" station. Subsequently, dynamic verification calculations were performed for the static loads on the system "metro tunnel – ground – foundation – upper structure of the shopping and entertainment complex " (Fig. 3) as a spatial system. Vibrodynamic monitoring was carried out during the installation of long bored piles along the station for the construction of the multi-story shopping and entertainment complex on them; scientific and technical support for the construction works; vibrodynamic inspections of the shopping and entertainment complex structures; verification of the compliance of the constructed structures with the approved project, and assessment of the technical



Fig. 2. Modern view of the "Heroes of Dnipro" metro station with the "Oasis" " shopping and entertainment center built above it.

condition and operational suitability of the load-bearing structures.

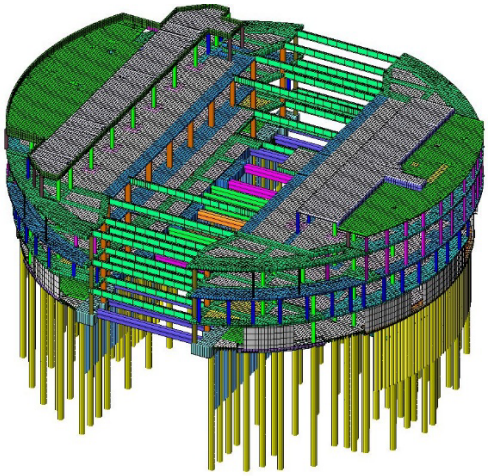


Fig. 1. Calculated numerical model of the "Oasis" shopping and entertainment center.

2 THE APPLIED METHODS

The applied methods included visual and instrumental inspection of the station using IoT, its engineering-geological studies, development of Digital Twins, and experimental verification of the mathematical and numerical models of the Digital Twins.

The dynamic surveys of the soil and the metro station platform surfaces under the subway trains and vehicles influences were performed in a vertical and two horizontal directions using the vibration sensors integrated into the IoT system. The vibration signals were simultaneously recorded by three channels using an 8-channel seismic monitoring system. Based on the performed measurements and the results of signal processing by applied software, the vibration accelerations time signals and their spectra were recorded. The analysis of the amplitude spectra graphs of the metro station and SEC structures vertical and horizontal vibration accelerations showed that the frequencies of their predominant vibrations under the subway trains influence were 12–90 Hz. According to the results of analyzing the actual experimental data of the vibration parameters of the platform foundation slab, column foundations, soil near the metro station and SEC bearing structures (the ceiling slabs and crossbars) obtained during the subway trains movement, as well as the operation of the ventilation systems of the dust removal system and elevator, the following conclusions could be drawn: the maximum vertical vibration acceleration of 0.45 m/s2 was recorded for the metro station platform foundation slab during the movement of subway trains; the maximum vertical acceleration of the soil surface above the station was from 0.27 m/s2 to 0.35 m/s2 under the influence of subway trains. The predominant frequencies of the vertical and horizontal vibrations of the station platform foundations under the subway trains influence were recorded in the range of 20–90 Hz. The predominant frequencies of the vertical and horizontal vibrations of the soil surface near the shopping and entertainment center were recorded in the range of 12–90 Hz. For assessing the shopping and entertainment center structures bearing capacity and their seismic resistance, the 6-points design seismicity should be taken (taking into account the high soil moisture). The LIRA-CAD software package was used for numerical calculations.

3 THE RESULTS

1. One of the key factors to consider during the station's modification was the dynamic impact of metro trains on the shopping and entertainment center building. During train passages, the maximum acceleration values in the soil base reached: 10 cm/s² within 5 meters from the metro station, and 3 cm/s² at a distance of 10 meters. The presence of the shopping and entertainment center piles reduces the calculated soil acceleration by 30–50%.
2. It was established that during metro train passages, the parameters of the stress-strain state change within the following ranges: vertical stresses in the shopping and entertainment center base directly beneath the tunnel increase by up to 16.0%; at a distance of 5 meters – by 3.5–8.0%, and at a distance of 10 meters – by up to 1.5%. Horizontal stresses in the shopping and entertainment center base directly beneath the tunnel increase by up to 27.0%; at a distance of 5 meters – by 8.0–13.2%, at a distance of 10 meters – by up to 3.0%, and at a distance of 15 meters – by less than 1%. Shear stresses in the base directly beneath the tunnel increase by up to 25.0%; at a distance of 5 meters – by up to 45.0%, at a distance of 10 meters – by up to 25.0%, and at a distance of 15 meters – by up to 4.5%.
3. The calculated model of the "metro tunnel – ground – foundation – upper structure of the shopping and entertainment center" as a spatial system corresponds to the actual object. The calculated acceleration values match the results of experimental vibrodynamic studies at the sensor installation points.

4 CONCLUSIONS

1. As part of the development of Digital Twins, their verification was conducted using the results of experimental vibrodynamic studies. For this, the dynamic characteristics of the soils and the load parameters on the metro tracks were adjusted in the numerical model. The discrepancy between the calculated results and field tests was no more than 15%.
2. It is important to note that part of the vibrations from the metro tracks is absorbed by the sheet pile retaining wall, while part is reflected back toward the metro station. This effect could, in the long term, lead to changes in the stress-strain state of the station itself and requires further research.

Development of brick architecture in the city of Pavlodar (pre-revolutionary period)

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ABSTRACT

This research examines the architectural development of Pavlodar, Kazakhstan, focusing on brick architecture during the pre-revolutionary period and the early 20th–21st centuries. It investigates residential, public, and religious buildings, emphasizing stylistic features, construction techniques, and their historical significance. The central research question explores how Pavlodar's architectural heritage reflects the cultural and economic influence of the region's merchants during this period. This study is particularly relevant to the ongoing «Rukhani Zhangyru» program, as many architectural objects were documented and surveyed for the first time. Previous studies on Pavlodar's architecture have been limited, particularly in fieldwork-based analyses of architectural styles, construction methods, and ornamentation. This research addresses these gaps by conducting detailed field surveys, measurements, and analyses of significant architectural objects. It also draws on archival materials and historical and contemporary literature to comprehensively understand the region's architectural evolution. The study reveals that Pavlodar's economic growth in the 1870s–1880s, driven by vibrant fair trade, enabled wealthy merchants to construct notable brick and wooden buildings. These structures, blending functionality with aesthetic appeal, feature intricate decorative brickwork and wood carvings. Prominent examples include the trading house of merchant Derov, the brick and wooden house of merchant Zaitsev, the estate of Tatar merchant F. Ramazanov, and the Surikov estate. Public structures such as the People's House and the Vladimir School, alongside agricultural facilities like granaries and stables, further illustrate the architectural diversity of this era. This research underscores Pavlodar's architectural heritage's cultural and historical significance, highlighting its role in shaping the city's identity. By applying modern fieldwork methods to historical architecture, the study contributes to architectural history. It emphasizes preserving this heritage, providing a foundation for future research and restoration efforts.

Keywords: brick houses, gates, styles, methodology, ornamentation.

1 INTRODUCTION

Pavlodar's historical development reflects its socio-economic and cultural evolution. The city's territory originally comprised the historical center and part of a former Cossack settlement, divided socially into Cossack, bourgeois, working, and Kazakh districts. The Zaton area functioned independently, while the outskirts were inhabited by «zhataki» (Kazakh poor) - impoverished Kazakhs employed at docks, salt mines, mills, and transportation [1,2]. Economic growth in the Semipalatinsk province during the late 19th century, driven by agriculture and the salt industry, catalysed Pavlodar's urban expansion [3].

The city's architectural identity emerged through Siberian modernism, characterized by eclecticism adapted to regional conditions. Residential and public buildings from the late 19th to early 20th centuries exhibit influences of the Art Nouveau style. Designs by Russian architects such as P. Batov and V. Petrov incorporated decorative features, including rustic masonry, forged metal structures, elaborate pediments, pylons, cornices, and plasterwork. Wooden architecture, prevalent in log houses clad with wooden planks, featured intricate wood carvings for ornamentation. Construction predominantly utilized local materials such as fired brick, wood, and metal, underscoring the integration of regional resources in Pavlodar's architectural development [4].

The study addresses the following research objectives:

- analyze the influence of historical, economic, and natural-climatic factors on Pavlodar's planning and architectural development up to the 21st century;
- identify regional characteristics of Pavlodar's urban planning organization during this period;
- provide a chronological overview of the city's architectural evolution within the specified region and timeframe;
- examine ornamentation types, compositional techniques, and form-making elements characteristic of the city's architecture;
- review and analyse architectural literature to define the core principles shaping regional architectural development in the modern era;

- investigate historical and architectural materials from the early 20th-century Pavlodar.

A key focus is preserving historical architectural objects and their integration into the contemporary urban fabric, addressing challenges through regional and traditional influences. A proposed solution involves developing a new architectural style termed «abstract symbolism», characterized by simple geometric forms representing Kazakhstan's cultural heritage. This approach facilitates the reflection of traditional values while enabling harmonious coexistence with modern architectural trends.

2 METHODOLOGY

The study employs a comprehensive methodological approach, including archival research in Pavlodar, Astana, Omsk, and Tomsk, oral histories from city elders, field surveys, photographic documentation, and graphic work. Detailed measurements and visual studies were conducted to document the architectural features of numerous stone and wooden buildings.

3 RESULTS AND DISCUSSION

3.1 Merchant A. Derov's trading house

Located on Vladimirskaya Street (now Astana Street 149), this two-story Siberian modernist building, built near the Irtysh River, enabled Derov to oversee cargo shipments. The house, which features decorative masonry, turrets, and wrought iron railings, was restored in 2012.

3.2 Merchant Filatov's Residential House

Built in the late 19th century, this L-shaped, two-story brick structure includes high plinths, tall windows, pointed arches, and wrought-iron balconies. Since 1988, it has housed the regional historical museum named after G. Potanin.

3.3 Surikov Estate

Constructed in 1896 at 168 Lenin Street, this U-shaped Siberian modernist complex was a major commercial hub. Featuring rusticated masonry and decorative pilasters, it housed cinemas in the Soviet era and now functions as the A. P. Chekhov Drama Theater.

3.4 Balandin Brothers' House

A single-story early 20th-century brick trading house with pilasters, tall rectangular windows, and an ornamental turret. It later served as a workers' club and was restored after a fire in 1982 [5-11].

CONCLUSIONS

The study has examined the architectural heritage of Pavlodar, emphasizing its historical, artistic, and cultural significance within the broader context of Kazakh heritage. The findings highlight that the city's architectural identity was shaped by regional influences and the impact of Russian settlers, who introduced wooden and stone construction elements adapted to local climatic conditions. The dominant styles of late eclecticism and Siberian modernism, characterized by brick masonry, rustication, and classical decorative techniques, played a crucial role in defining the city's architectural landscape. A key challenge in preserving Pavlodar's historical monuments is restoring their original appearance while accommodating contemporary urban development. Many structures underwent significant alterations during the Soviet period, necessitating a balanced approach to restoration that integrates modern advancements without compromising historical authenticity. International conservation practices from European and Middle Eastern countries provide valuable insights for implementing sustainable preservation strategies.

One of the most effective methods for revitalizing historical buildings is adapting the «loft» style, which enables the harmonious coexistence of historical authenticity and modern functionality. By adopting such approaches, Pavlodar can maintain its architectural legacy while fostering sustainable urban development, ensuring that its merchant heritage continues to serve as a cultural and historical landmark for future generations.

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Features of the formation of the architecture of secondary vocational education institutions in modern Kazakhstan

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ABSTRACT

The aim of the study is to identify the features of the formation of the architecture of secondary vocational education institutions in Kazakhstan. The article examines the architectural principles of designing secondary vocational education institutions in Kazakhstan, taking into account historical evolution, modern problems and promising approaches. At the same time, the article highlights aspects of adapting architectural solutions to the new socio-economic conditions of the country. The study examines the historical evolution of vocational education institutions, from Soviet standard buildings to modern examples of professional colleges. Particular attention is paid to problems associated with outdated material and technical base, insufficient adaptation of buildings to new educational standards and requirements. At the same time, aspects of adapting secondary vocational education buildings built in the Soviet period to new realities and requirements for vocational school buildings are covered in detail. The article highlights typological features of the formation of the architecture of secondary vocational institutions in the context of their historical development, and also examines their characteristic features and differences. The article considers current trends, including the introduction of the concept of sustainable design, the use of energy-saving technologies, the use of flexible spaces and modular structures, and analyzes modern problems and proposes innovative approaches to improving the educational infrastructure. The study emphasizes the need to develop regulatory standards and create adaptive architectural environments that meet modern educational needs. In the context of global challenges and rapid technological progress, Kazakhstan is faced with the need to revise and reform the architecture of secondary vocational education. Particular attention is paid to the synthesis of educational standards with modern infrastructure practices that take into account the climatic features of the regions and the needs of the labor market. The study highlights systemic problems in the formation of secondary vocational education institutions in Kazakhstan and offers solutions: integrating business into the educational process, updating curricula taking into account the "Atlas of New Professions", as well as architectural transformations to create flexible educational and industrial spaces. It should be noted that the article also highlights regional features in the formation of educational clusters of secondary vocational education institutions using the example of different regions of Kazakhstan. The topic of the article is relevant in Kazakhstan in the new economic and social conditions, taking into account the changing requirements for the educational process. The research materials have theoretical and practical significance for modern Kazakhstan science and production activities.

Key words: secondary vocational education, college architecture, architectural modernization, sustainable design,

1. INTRODUCTION

In the context of global challenges and rapid technological progress, Kazakhstan is faced with the need to review and reform the architecture of the secondary vocational education system. This initiative is aimed at implementing a number of reforms designed to significantly improve both the quality of vocational education and its contribution to the national economy. Consequently, the architecture of secondary vocational education institutions requires modernization and renewal. It is extremely important to create educational spaces that meet international standards and best practices in terms of architecture, typology and urban planning. The topic is especially relevant in Kazakhstan in the new economic and social conditions, taking into account the changing requirements of the educational process.

2. RESEARCH METHODOLOGY

This study uses a systems approach to the issues under study, including forecasting methods and a comprehensive analysis of the conditions and factors influencing the achievement of the goals and objectives of the study.

Research Objective:

The purpose of this study is to identify the features of the formation of the architecture of secondary vocational education institutions in Kazakhstan. The article examines the architectural principles underlying the design of such institutions, taking into account the historical context, modern problems and promising approaches.

Research object:

Buildings and structures of secondary vocational education institutions in Kazakhstan.

Research objectives:

To consider the main requirements influencing the formation of secondary vocational education institutions, based on the characteristics of the educational and upbringing process.

Against the backdrop of rapid socio-economic transformations and global challenges, the secondary vocational education (SVE) system in Kazakhstan is undergoing significant reforms. Modernization of the architectural environment of secondary vocational education institutions is becoming increasingly relevant, due to modern requirements for education, technological progress and urban development priorities focused on human capital, digitalization and sustainable development. This study examines the historical trajectory of vocational college architecture in Kazakhstan, from the standardized typologies of the Soviet era focused on industrial training, through the post-independence transition marked by decline and stagnation, to the contemporary phase characterized by a growing focus on individualized, high-tech, and environmentally oriented learning environments. The analysis highlights the obsolescence of much of the existing infrastructure, which often does not meet modern pedagogical, sanitary, and aesthetic standards.



Fig. 1. Photo by the author. Analysis of colleges in Kazakhstan

Particular attention is paid to the typological evolution of vocational education institutions. The article examines both traditional models (industrial, polytechnic, agricultural, and medical colleges) and new forms that integrate educational, research, and production functions into unified architectural and spatial systems. The growing number of innovative research centers in vocational colleges, the development of educational-industrial clusters and the introduction of modular approaches to learning require new design strategies such as mobile learning spaces, multifunctional laboratories, technical workshops and collaborative areas.

The urban dimension of secondary and vocational education institutions is also an important research area.

3 CONCLUSIONS

These institutions should not only be spatially integrated into urban or rural contexts, but also function as dynamic centers for youth interaction, innovation and cultural exchange. In the planning of modern educational campuses, more and more attention is paid to design principles such as human-centeredness, sustainable mobility, accessibility and architectural expressiveness.

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Using GIS to preserve natural landscapes in the process of developing bio-architecture and tourist infrastructure

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ABSTRACT

Contemporary development in tourism and architecture faces significant challenges related to the preservation of natural landscapes. Geographic Information Systems (GIS) provide a powerful tool for the analysis and management of natural resources, making them indispensable in the design of bioarchitecture and tourism infrastructure. The aim of this study is to develop an innovative approach to integrating GIS for the preservation of natural landscapes and the sustainable development of tourist facilities. The paper proposes a model for using GIS in spatial landscape analysis, identification of ecologically sensitive areas, and optimization of tourism infrastructure placement. Special attention is given to the integration of environmental protection mechanisms into tourism infrastructure, including the use of renewable energy sources, environmentally friendly building materials, and green technologies. The results of the study demonstrate that the application of GIS enables the minimization of environmental impact, reduces the risk of landscape degradation under growing tourist flows, and supports the creation of architectural structures that harmoniously blend with the natural context. In addition, the use of GIS enhances the efficiency of tourism zone management, including territory monitoring, evacuation route planning, and risk assessment. The conclusions of the study highlight the importance of synergy between technological and ecological approaches in design. The proposed methodology can serve as a foundation for creating sustainable tourism infrastructure focused on the conservation of natural wealth and cultural heritage. This research offers practical recommendations for architects, environmentalists, and planners interested in preserving natural landscapes and implementing innovative projects.

Keywords: GIS, bio-architecture, natural landscapes, tourist infrastructure

1 INTRODUCTION

The rapid growth of tourism and the construction of tourism infrastructure present new challenges for the preservation of natural landscapes and biodiversity. The concept of bioarchitecture offers principles of environmentally oriented design that take into account the natural environment and available resources. Geographic Information Systems (GIS) serve as a powerful tool that facilitates the integration of spatial data on topography, ecosystems, and existing infrastructure to support informed planning. The application of GIS technologies enables spatial analysis of ecological sensitivity and the identification of areas requiring special protection. This study aims to develop an integrated methodology for using GIS to preserve natural landscapes in the context of bioarchitectural and tourism infrastructure development.

2 METHODOLOGY

The research methodology is based on comprehensive geospatial analysis using GIS tools. The primary stage involves the collection and integration of various geodata sources (topographic maps, satellite imagery, vegetation data, hydrography, protected natural areas, and existing infrastructure). The main stages of the methodology include:

- collection of geospatial data;
- spatial analysis;
- identification of ecologically sensitive areas;

criteria-based assessment of territories with consideration of ecological, transport, and recreational factors, aimed at selecting optimal sites for bioarchitectural structures and tourism facilities with minimal environmental impact.

3 RESULTS AND DISCUSSION

The implementation of the proposed methodology enables the creation of detailed thematic maps that show areas of varying ecological value and suitability for infrastructure development. Spatial analysis identified areas with high concentrations of valuable natural resources (species diversity, aquatic ecosystems, ecological corridors). In addition, areas previously exposed to anthropogenic pressure (developed or degraded lands) were mapped. The resulting maps demonstrate that optimal zones for bioarchitectural and tourism projects are predominantly located in areas with minimal environmental conflicts, confirming the effectiveness of the GIS-based approach to planning.

The combined use of GIS and bioarchitectural principles provides design flexibility. For example, building orientation can be determined using solar radiation maps, while routing of tourist trails and site selection is planned to preserve natural topography and vegetation cover, minimizing erosion. Based on multi-criteria analysis, recommendations were developed for establishing buffer zones around protected and vulnerable areas, further reducing ecological pressure.

Thus, the proposed approach demonstrates the ability to minimize environmental impact through targeted site selection and the preservation of ecological connectivity (migration corridors, water networks, and forest-park systems). Integrated GIS analysis supports balanced development of tourism and bioarchitectural projects by adapting design solutions to actual site conditions and sustainable development goals. Overall, the implementation of GIS technologies provides a rationale for planning decisions based on spatial data and enhances coordination among engineers, ecologists, and architects.

4 CONCLUSION

The findings underscore the importance of synergy between modern geoinformation technologies and ecological principles in sustainable planning. The proposed GIS approach demonstrated high efficiency as a decision-support tool during the design phase, contributing to the reduction of negative impacts on natural landscapes. The practical value of the developed model lies in its ability to integrate ecological, geographical, and infrastructural data for comprehensive justification of spatial decisions. The proposed approach is universal and can be adapted to various geographical conditions and scales. It serves as a foundation for scientifically grounded recommendations on the placement of bioarchitectural structures and tourism infrastructure while considering ecosystem conservation requirements. Thus, the methodology confirms the importance of integrating technology and ecology in modern planning processes and can be practically applied to the development of sustainable projects.

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Architecture of brick buildings in Semipalatinsk in the 19th-20th centuries

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ABSTRACT

Brick architecture in Semipalatinsk is a key element of the city's historical and cultural heritage. Its development began in the 19th century, coinciding with the rise of stone construction in the region. The widespread use of brick was made possible by local raw materials and advancements in brick-firing technology, leading to the proliferation of brick structures that shaped the urban landscape of Semipalatinsk. The brick architecture in the city showcases a blend of stylistic influences, including Russian architecture, eclecticism, Art Nouveau, and constructivism. These styles reflect the complex historical and cultural processes that have influenced the region. Notable examples of brick construction include administrative and residential buildings, many of which have been preserved and are now considered significant architectural monuments. These structures are renowned for their intricate decorative elements, including elaborate brick masonry, arched openings, ornamental cornices, and carved details, each representative of a distinct historical period. In recent years, research and restoration efforts have focused on preserving and adapting these historic buildings for contemporary use. These initiatives play a vital role in maintaining the architectural heritage of Semipalatinsk and ensuring its integration into Kazakhstan's broader cultural landscape. This study aims to analyze the brick architecture of Semipalatinsk as a fundamental component of the city's historical and cultural identity. It examines the architectural styles, structural solutions, and decorative elements of brick buildings while tracing the historical stages of their development. Additionally, the study explores their impact on the city's urban appearance and local identity. A key aspect is assessing the current condition of preserved buildings and identifying practical strategies for their restoration and adaptation. The research methodology includes the analysis of archival documents, field surveys, sketches, and photographic documentation to offer a comprehensive understanding of Semipalatinsk's brick architecture.

Keywords: brick architecture, ornamental details, historical building, heritage, reconstruction.

1 INTRODUCTION

Brick architecture in Semipalatinsk holds a significant place in the city's historical and cultural development, shaping its unique urban landscape. Since the 19th century, brick has become the primary building material, contributing to the strength and durability of buildings and facilitating urban growth. Semipalatinsk, one of Kazakhstan's oldest cities, played a vital role in the region's economic, educational, and cultural development, which is reflected in its architectural traditions. The use of locally sourced brick enabled the construction of distinctive buildings that blend various architectural styles, including Russian classicism, eclecticism, modernism, and constructivism. Today, many of these brick buildings are considered valuable historical and architectural monuments. Preserving and restoring these structures is a crucial task, ensuring the city's rich architectural heritage remains integral to its identity. This study aims to explore the key features of Semipalatinsk's brick architecture, assess its historical significance, and identify strategies for its preservation and integration into the modern urban environment.

The development of brick architecture in Semipalatinsk (now Semey) began in the 19th century, driven by the city's position as a major trade and cultural center. The establishment of local brick factories spurred the construction of durable buildings, initially focused on administrative and religious structures. Early architectural styles included Russian Classicism, characterized by symmetry and simplicity, while later styles such as Eclecticism, Art Nouveau, and Constructivism introduced diverse design elements.

Notable examples include the Governor's House (1895), which transitioned from a military residence to a Soviet-era political hub, and has since been adapted into a local history museum. The architecture of Semipalatinsk was influenced by the region's climate, with thick walls, arched ceilings, and reinforced structures. Red brick, locally sourced, became the material of choice, and decorative masonry was used to enhance facades.

Semipalatinsk's brick buildings, shaped by historical

events, have become important urban landmarks, giving the city a unique character. To preserve this architectural heritage, it is crucial to implement restoration programs, promote cultural and educational initiatives, and adapt historic buildings for modern use. Modern technologies like 3D scanning can aid in restoration efforts, ensuring these structures remain integral to the city's identity for future generations.

2 METHODOLOGY

The study of Semey's historical architecture was conducted using a comprehensive approach, combining archival research, cartographic analysis, expert assessments, and comparative methods. This approach provided a thorough understanding of the city's architectural heritage and the challenges it faces in preserving it.

Key sources included archival documents, historical maps, and plans that offered insights into the city's architectural development. Official acts and regulations related to cultural heritage preservation, as well as historical documents about Semey's early military fortifications, were analyzed. The study also drew on scientific publications, monographs, and local historical research to examine the city's urban growth and architecture.

The study employed several research methods:

Historical and analytical method: to examine the evolution of the city's architecture and the influence of social, economic, and cultural factors.

Comparative analysis: to compare architectural features from different periods and assess changes in building functions, as well as the impact of modern development on the city.

Visual analysis: document the current condition of buildings, identify preservation issues, and study construction materials and techniques.

Content analysis: to examine scientific works on architectural heritage protection and restoration technologies and to identify global preservation strategies.

This multi-method approach enabled a detailed assessment of Semey's architectural heritage.

3 RESULTS AND DISCUSSION

The 1910 pumping station in Semey (formerly Semipalaa) is a notable example of industrial architecture and a significant part of the city's cultural heritage. Built with red brick, the station features intricate details, including arched openings, pilasters, cornices, and hexagonal towers, showcasing an individual design approach rarely seen in modern industrial buildings. Despite its age, the pumping station remains operational, providing the city with drinking water. However, its closed status limits public access, raising concerns about preserving its historical value while balancing its utilitarian role. Restoration efforts, including the preservation of architectural elements like plaques, geodetic signs, and metal artworks, are crucial for maintaining the station's aesthetic and historical significance. This building exemplifies the intersection of industrial functionality and architectural beauty, highlighting the need for comprehensive restoration programs to protect the city's architectural heritage.

4 CONCLUSIONS

The brick architecture of Semipalatinsk is a vital component of the city's historical and cultural heritage, showcasing unique craftsmanship and design from the late 19th to early 20th centuries. Many of these buildings, including industrial and public structures, have become integral to the city's architectural identity. However, they are at risk due to neglect and reconstruction. Preserving these historical monuments requires a systematic approach, including restoration efforts, adaptation to modern needs, and increased public awareness. Developing restoration programs, applying modern conservation technologies, and promoting the cultural value of these buildings are crucial for safeguarding Semey's architectural legacy for future generations.

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Risk assessment of waste rock dumps in the development of highland deposits

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ABSTRACT

This article discusses approaches to the risk assessment of waste rock dumps during the development of highland deposits. Special attention is paid to analyzing factors affecting the stability of waste dumps, including natural (geological, hydrogeological conditions, climatic influences) and anthropogenic factors (mining activities and dump loads). The main risks associated with possible deformations and failures of dumps are considered, taking into account natural and technogenic factors. A risk matrix for dump formation during the development of highland deposits is presented, considering the influence of these factors.

Keywords: waste rock dumps, highland deposits, risk assessment, stability, mining operations.

1 INTRODUCTION

The development of highland deposits is associated with the formation of waste rock dumps, whose stability is a critical factor for both environmental and industrial safety. Under conditions of complex mountainous terrain, increased seismicity, intensive atmospheric precipitation, and other natural influences, the risk of deformation and failure of waste rock dumps significantly increases [1].

Modern methods for assessing the safety factor of dumps require consideration of the interaction between geological and geomechanical environments [2,3], as well as hydrogeological and climatic factors, which is particularly important in highland mining operations.

The development of reliable forecasting methods for dump stability and the implementation of effective engineering solutions help minimize the risk of dump failures, reduce economic losses, and ensure the safety of mining operations.

An integrated approach to studying the safety factor of waste rock dumps enables the development of practical solutions aimed at minimizing slope and dump stability losses and managing associated risks in high-altitude and seismically active regions.

2 METHODOLOGY

The formation of waste rock dumps in mountainous areas is associated with the risk of stability failures, which can pose a threat to both operational safety and the surrounding environment. During the development of high-altitude deposits, dumps are typically placed on slopes with inclinations ranging from 10° to 25° at elevations of 3500–4000 meters, which significantly complicates the task of ensuring their stability [4].

The diversity of lithological composition and the presence of groundwater significantly affect the mechanical properties of rocks, altering their stress-strain state. Groundwater can cause leaching, reduce rock strength, and increase pore pressure, contributing to the formation of cracks and faults. The fracturing of the rock mass and its mechanical properties depend on composition, strength characteristics, and the bearing capacity of the rocks, all of which must be considered during dump design [1].

Seismic activity substantially increases the risk of dump instability, especially in regions with high seismicity levels. Under such conditions, the stability coefficient is reduced.

Climatic factors such as precipitation, temperature fluctuations, and weathering processes impact the strength properties of rocks, causing deformation and reducing slope stability. Seasonal cycles of freezing and thawing of materials within the dump body contribute to the development of irreversible deformations [1,2].

Additional risks are created by filtration processes, where moisture penetrates the rock mass, reducing its stability and provoking slippage.

The placement of dumps adds extra loading on the slope, causing surface deformations and activating toe sliding. Mining operations also exert localized mechanical impacts, provoking settlement and collapses [3].

3 RESULTS

As a result of the research, a risk matrix for dump formation at highland deposits was developed, taking into account the influence of both natural and anthropogenic factors.

4 TABLES

The matrix presents a risk assessment for the formation of waste rock dumps at high-altitude deposits. It includes both natural (seismicity, moisture content, groundwater level, etc.) and man-made (load, dump height, loosening factor, etc.) factors that influence dump stability. For each factor, threshold values, possible consequences, and the corresponding risk level — low, medium, or high — are provided.

Table 1. Risk matrix for dump formation at highland deposits.

Test ID	Width (m)	Height (m)	Depth (m)
Natural Factors			
Seismicity of the highland area (points)	4 points	No observed failures or displacements	Low risk
	7 points	Formation of cracks, possible displacements	Medium risk
	9 points or more	Destruction and displacement	High risk
Moisture content of dump material due to precipitation (%)	2–5%	Dry, no deformation	Low risk
	6–15%	Moist, possible plastic deformations	Medium risk
	16% or more	Wet, viscoplastic deformations develop	High risk
Rise of groundwater level (m/day)	0–0.1	No influence on displacement at the base-dump boundary	Low risk
	0.12–0.3	Possible fading displacements at the base-dump boundary	Medium risk
	0.3 and above	Likely displacement within 20–25 days at the base-dump boundary	High risk
Correlation between internal friction angle φ and slope angle α (degrees)	φ > α	No deformation observed	Low risk
	φ = α	Possible deformation development	Medium risk
	φ < α	Dump displacement	High risk
Anthropogenic Factors			
Dump pressure P versus bearing capacity N (MPa)	P < N	No deformation under dump, stable condition	Low risk
	P = N	Subsidence under dump	Medium risk
	P > N	Subsidence and displacement under dump	High risk
Dump bench height h (m)	h < 50	No deformation	Low risk
	h = 50	Fading deformation	Medium risk
	h > 50	Progressive deformation	High risk
Looseness coefficient	1.5–1.4	Stable condition	Low risk
	1.3–1.2	Limit condition	Medium risk
	1.1–1.0	Unstable condition	High risk
Safety factor K*y	Ky > 1.3	Stable condition	Low risk
	Ky = 1.3	Limit condition	Medium risk
	Ky < 1.3	Unstable condition	High risk
Deformation rate v (m/s)	v < 1.2	Safe deformation rate	Low risk
	v = 1.2	Limit deformation rate	Medium risk
	v > 1.2	Dangerous deformation rate	High risk

5 CONCLUSIONS

The conducted research examined the key factors influencing the stability of waste rock dumps during the development of highland deposits. Dump stability depends on a combination of natural factors (seismic activity, precipitation, hydrogeological conditions) and anthropogenic factors (dump pressure on the foundation, dump parameters, looseness and safety factors).

Particular attention was given to seismic and climatic influences, which significantly reduce the stability coefficient, especially in high-altitude and seismically active regions.

The development of stability assessment methods and risk analysis systems enables more accurate forecasting of potential deformations and failures, thereby reducing environmental threats and ensuring personnel safety.

The proposed risk matrix and the recommendations for its application in assessing dump stability under highland conditions have practical significance for mining enterprises.

It allows for effective control of geotechnical risks, ensuring the safety of both workers and the environmental situation of the region.

The implementation of the proposed methods and approaches in the design and operation of dumps will contribute to reducing emergencies and ensuring the long-term stability of dump systems. Conclusions summarize the most important propositions concisely derived from the study.

Use of soil-cement elements to improve slope stability in geotechnical calculations

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ABSTRACT

This paper deals with the application of numerical methods for estimation of strength and deformation characteristics of soil. The study is devoted to a construction site in Kostanai, where the presence of a nearby gorge requires a careful assessment of the stability and bearing capacity of the soil. Using advanced modeling techniques in Plaxis software, the study investigates the effect of soil-cement elements on slope stability and deformation under load. The results show a significant improvement in soil stability, which ensures compliance with safety standards for construction projects.
Keywords: Ground strength, deformation characteristics, numerical methods, geotechnical design, foundation design, slope stability, soil-cement elements, Plaxis program.

1 INTRODUCTION

Assessing the bearing capacity of foundations and predicting the settlement of buildings and structures are fundamental tasks in engineering geology and geotechnical design. Determining the limit state allows us to establish whether the design load exceeds the bearing capacity of the soil. Failure to comply with this condition can lead to significant deformation or destruction of structures, especially in difficult conditions—on slopes, in water-saturated areas, and near ravines.

Modern approaches involve the use of improved calculation methods, including methods that do not require the use of a deformation modulus, which allows for weak and water-saturated layers, capillary suction, and groundwater level fluctuations to be taken into account.

Increasing attention is being paid to the mutual influence of adjacent foundations, especially during the reconstruction or superstructure of buildings. Ignoring this factor can lead to an underestimation of settlement values and the occurrence of uneven deformation.

This study examines a construction site in the city of Kostanay (Kazakhstan), where an extension to a building near a ravine approximately 9 m deep is planned. The geomorphological conditions of the site require analysis of both the deformation behavior of the foundation soils and the stability of the slope. The aim of the work is to evaluate the effectiveness of soil-cement elements in increasing slope stability and reducing foundation deformation. The modeling was performed in Plaxis 2D, taking into account the physical and mechanical properties of the soils and the influence of the nearby ravine.

2 METHODS

To simulate the reinforcing effect of soil-cement elements (SCE) in weak and potentially unstable soils, the finite element method was used in the Plaxis 2D program. SCEs were modeled using the Embedded Beam Row tool, which allows you to specify elements with axial, bending, and shear stiffness, which is suitable for columns and panels made by deep mixing or jet grouting.

A type B scheme was used—vertical inclusions installed at equal intervals along the slope profile. The geometric and strength parameters corresponded to the loess and loamy soils of northern Kazakhstan.

Boundary conditions: the lower boundary is fixed vertically and horizontally, the side boundaries are fixed only horizontally, which corresponds to flat deformation. The size of the calculation area exceeded the zone of influence of loads and deformations.

The slope stability was assessed using the Strength Reduction Method, in which the safety factor is determined based on the reduced values of cohesion c and internal friction angle φ.

Engineering geological surveys included drilling two wells 8 m deep with seven undisturbed samples taken from each. The section is represented by yellow-brown loam with layers of dense sand up to 4 m thick and dark green clay of semi-solid and solid consistency

with sandy layers up to 5 cm thick in the 4–8 m interval. The groundwater level is about 4 m.

The topographic model of the site (Fig. 1) includes a relief with an absolute elevation difference of 154.47–155.30 m and a ravine ~9 m deep near the building zone, which determined the need to use GCE.

The soil model in Plaxis (Fig. 2) reflects the geometry of the layers, their properties, and their interaction with structural elements, allowing the stress-strain state of the massif and the stability of the slope to be assessed.

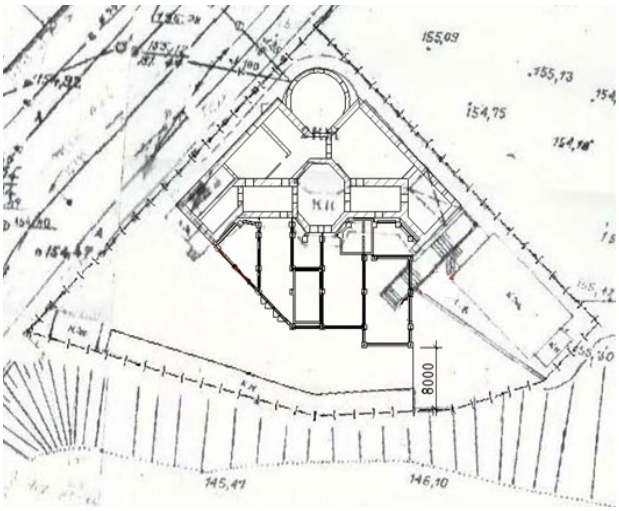


Fig. 1. Model of topographic survey of the object



Fig. 2. Soil model in the Plaxis program

3 RESULTS AND DISCUSSION

As part of numerical modeling using the finite element method in the Plaxis 2D environment, two calculation scenarios were performed: without foundation reinforcement and with the use of soil-cement elements (SCE) modeled using the Embedded Beam Row tool.

Fig. 3 shows the distribution of displacement vectors and deformation zones under full operational load. The reinforced version demonstrates a noticeable reduction in displacements at the foot of the slope and under the foundation heel, which indicates a redistribution of stresses and an increase in the spatial stiffness of the array.

Fig. 4 shows a map of shear deformations. When using GCE, the potential shear surface shifts deeper and is divided into several local zones, which increases the stability of the system. The slope stability coefficient was ΣM = 1.265, which exceeds the normative values.

The simulation results confirm that the use of GCE improves stress distribution, reduces deformation in the slope zone, and can be recommended for strengthening foundations in complex geomorphological conditions..

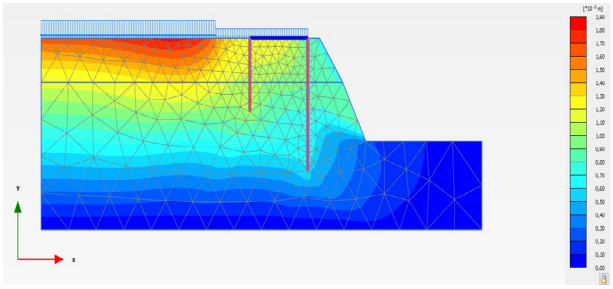


Fig. 3. Results of full load displacement calculations when soil cement elements are considered

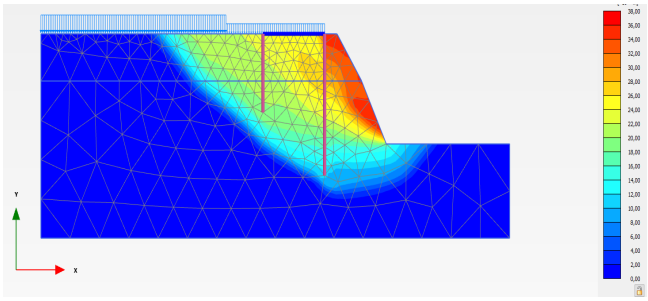


Fig. 4. Results of full-load stability calculations with soil-cement elements taken into account

CONCLUSIONS

During the study at a construction site in Kostanay (Kazakhstan) with loess-like and loamy soils located near a ravine with a depth of ~9 m, surveys and modeling were performed in Plaxis 2D. The results showed that the use of soil-cement elements reduces displacement at the foot of the slope and under the foundation, increases the stability coefficient to ΣM = 1.265, and shifts the potential shear surface deeper into the massif, confirming the effectiveness of the reinforcement.

Optimization of Pile Foundation Selection Using Artificial Intelligence

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ABSTRACT

Modern construction demands the integration of advanced technologies to enhance the efficiency of structural design and performance. One critical factor influencing the stability and durability of buildings is the selection of pile foundations optimized for specific soil conditions. This article explores the application of artificial intelligence (AI) methods to automate and optimize the process of selecting pile foundations.

The study proposes the integration of machine learning algorithms and deep neural networks to analyze geotechnical data, such as soil composition, load-bearing capacity, and water saturation levels. The article outlines the stages of AI model development, including data collection, preprocessing, training, and testing of algorithms. Special attention is given to the development of a predictive model capable of accounting for the geographic and climatic specifics of the construction region.

The findings demonstrate that AI significantly reduces the time required for engineering calculations while minimizing risks of design errors. Additionally, the proposed approach optimizes material costs and reduces environmental impacts by enabling the selection of sustainable solutions. The results hold practical value for civil engineers, designers, and geotechnical specialists. The conclusion discusses prospects for further development of AI technologies in construction and opportunities for their integration with other innovative approaches, such as digital twins and Building Information Modeling (BIM).

Keywords: pile foundation selection, engineering calculations, geotechnical data analysis, artificial intelligence (AI).

1 INTRODUCTION

The optimization of pile foundation design and selection has long been a critical area in geotechnical and structural engineering. Traditional methods, although robust, often involve conservative assumptions and manual iterations that may not yield the most efficient solutions. In recent years, Artificial Intelligence (AI) and Machine Learning (ML) technologies have begun to revolutionize foundation engineering, offering data-driven, adaptive, and highly optimized alternatives. This section presents a comprehensive review of relevant literature that underpins the integration of AI into pile foundation optimization.

2 TRADITIONAL APPROACHES TO PILE FOUNDATION DESIGN

Das (2015), in Principles of Foundation Engineering, offers a thorough exposition of conventional pile design methodologies. The author emphasizes the role of soil properties, pile material characteristics, and empirical correlations in determining pile bearing capacities. Traditional pile design heavily relies on static load tests, dynamic pile driving analyses, and empirical methods derived from field observations. Although these approaches have ensured safety and reliability, they are often limited by the complexity of soil-structure interactions and the inability to account for large datasets and multi-variable relationships effectively.

3 MACHINE LEARNING APPLICATIONS IN PILE ENGINEERING

The application of machine learning techniques in pile foundation optimization has been increasingly explored over the last decade. Liu and Li (2020) provide a comprehensive review of machine learning models used in pile engineering, highlighting how models such as Support Vector Machines (SVM), Random Forests (RF), and Neural Networks (NN) can predict pile behavior with higher accuracy compared to traditional methods. They conclude that ML models can handle complex nonlinearities inherent in soil behavior and provide rapid predictions once properly trained.

Zhang and Goh (2016) further advance this field by comparing the performance of Multivariate Adaptive Regression Splines (MARS) and Artificial Neural Networks (ANNs) for predicting pile drivability. Their

study reveals that data-driven models can outperform conventional predictive tools, especially when large historical datasets are available. Their findings underscore the importance of selecting appropriate features and model architectures for achieving reliable predictions.

The integration of optimization algorithms into pile foundation design has been another significant development. Li, Kong, and Zhou (2019) demonstrate the effectiveness of Genetic Algorithms (GA) in optimizing pile group configurations. By encoding potential design solutions into a population and iteratively applying evolutionary operations (selection, crossover, and mutation), GAs can efficiently search for cost-effective and performance-compliant designs. Their results show notable cost savings and structural improvements over conventional manual optimization.

One critical aspect of applying AI models in geotechnical engineering is ensuring their interpretability. Samui and Kim (2014) explore the use of Least Squares Support Vector Machines (LSSVM) for predicting pile bearing capacity, finding that not only do these models enhance prediction accuracy, but they also allow for a degree of model transparency through sensitivity analysis. Understanding the relative influence of input parameters, such as cohesion or groundwater level, is vital for building confidence in AI-assisted designs.

Emerging trends also point towards real-time optimization and monitoring of pile foundations. Yan and Chen (2021) discuss the convergence of Machine Learning and Internet of Things (IoT) technologies, enabling the continuous assessment of foundation performance. IoT sensors embedded within piles can feed real-time data into predictive models, allowing for dynamic reassessment and maintenance strategies throughout the structure's lifecycle.

Goodfellow, Bengio, and Courville (2016) in *Deep Learning* provide the theoretical backbone for many of the machine learning applications in pile engineering today. Their work details various architectures, including Convolutional Neural Networks (CNNs) and Recurrent Neural Networks (RNNs), which can be adapted for complex tasks such as time-series analysis of pile settlement or stress distribution patterns under variable loading conditions

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GIS Applications in engineering-geological investigations for optimizing foundation design

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ABSTRACT

In modern geotechnical and urban planning practices, Geoinformation Systems (GIS) have become a powerful tool for processing, analyzing, and visualizing spatial data across various fields. Their application is particularly valuable in engineering-geological investigations, both for scientific research and practical implementation in construction projects. The ability of GIS technologies to integrate and manage large datasets enables the efficient assessment of engineering and geological conditions, providing essential insights for urban zoning and foundation design.This study highlights the role of GIS technology in analyzing engineering-geological conditions and classifying urban territories based on soil types and foundation characteristics. The research focuses on cities in Kazakhstan, specifically Astana and Pavlodar, where rapid urbanization and increased construction activities require precise and cost-effective geotechnical solutions. By utilizing GIS-based methods, large volumes of geological data from engineering surveys can be processed to create zoning maps that categorize urban areas by foundation type, soil-bearing capacity, and other critical geotechnical parameters. One of the key advantages of GIS technology is its ability to optimize and accelerate engineering-geological investigations, significantly reducing the time and financial costs associated with fieldwork, data interpretation, and foundation design. Traditional methods of geotechnical assessments require extensive field surveys and laboratory testing, which can be expensive and time-consuming. In contrast, GIS-based models allow for a more systematic and comprehensive approach by integrating data from multiple sources, enabling engineers and urban planners to make informed decisions regarding land use and construction strategies.The results of this study demonstrate the effectiveness of GIS technology in improving the efficiency and accuracy of geotechnical evaluations, contributing to more sustainable urban development. The integration of GIS in engineering-geological investigations enhances the reliability of foundation design, ultimately ensuring safer and more cost-effective construction practices.

Keywords: geoinformation systems, engineering-geological investigations, engineering and geological conditions, soil properties, zoning maps.

1 INTRODUCTION

Urban development in Kazakhstan, particularly in key centers such as Astana and Pavlodar, is accompanied by large-scale construction of buildings and infrastructure. A critical factor in ensuring the stability and safety of these structures is the accurate assessment of engineering-geological conditions. Traditional methods based on fieldwork and laboratory testing are labor-intensive, costly, and often fail to capture the full spatial heterogeneity of soil conditions. In this context, the application of GIS—modern digital platforms for integrating, analyzing, and visualizing geological, hydrogeological, and geotechnical data—becomes especially significan (Demidenko, 2008).

2 GIS CAPABILITIES AND FUNCTIONS IN ENGINEERING GEOLOGY

2.1 Integration of heterogeneous data

GIS offers engineers and urban planners the following key functionalities:

Combining drilling results, laboratory tests, remote sensing data, digital elevation models, and archival maps.

2.2. Analysis of spatial variability

Visualizing differences in soil types, groundwater levels, and depth of strong soil layers.

Other GIS functions include:

- Creation of 2D and 3D subsurface models: modeling engineering-geological cross-sections and identifying weak zones;
- Automation of engineering map generation: producing maps of bearing capacity, seismic zoning, flooding risks, subsidence potential, etc.;

- Spatial classification of territories: zoning urban areas based on criteria for foundation suitability.

3 COMPARISON OF GIS WITH TRADITIONAL ENGINEERING-GEOLOGICAL METHODS

Traditional engineering-geological investigations (EGI) include drilling, sampling, and laboratory testing, which are time-consuming and resource-intensive. Additionally, they provide limited area coverage and may overlook local geological anomalies.
GIS-based approaches allow for:

- Utilizing both archival and current EGI data in a spatially-referenced format;
- Performing "what-if" scenario analyses to forecast soil behavior under loads;
- Reducing the number of boreholes through targeted investigations based on spatial models;
- Enhancing the speed of design decision-making (Alibekova, 2009).

4 PRACTICAL APPLICATION AND EFFICIENCY OF GIS IN KAZAKHSTAN

4.1 Practical application in Kazakhstan

GIS-based software solutions have been effectively applied in cities like Astana and Pavlodar to evaluate engineering-geological conditions, demonstrating their practical relevance in Kazakhstan (Zhussupbekov et al., 2012). These tools provide zoning based on soil types[1)] and depth of stable layers[2)], enabling a well-founded choice between pile and shallow foundations. GIS technology proves especially valuable in rapidly developing regions. Engineering surveys using GIS not only refine the geological understanding of an area but also allow for early risk prediction, reducing the need for rework and project adjustments.

4.2 Economic and strategic efficiency

GIS use contributes to:

- Reducing design time by 20–40% through automated data processing;
- Lowering fieldwork costs by optimizing borehole locations;
- Increasing the reliability of design decisions, thereby reducing the risk of structural failures;
- Supporting sustainable land development through rational land-use planning.

Thus, GIS becomes not just an auxiliary tool but a fundamental platform for comprehensive geotechnical planning.

5 CONCLUSION

The research confirms the high efficiency of GIS in engineering-geological investigations. GIS significantly improves the quality and accuracy of soil condition assessments during foundation design. Case studies in Astana and Pavlodar show that spatial data analysis enables the identification of high-risk zones, optimization of borehole placement, and selection of technically and economically justified design solutions. This approach reduces the volume of fieldwork, cuts costs, and increases construction reliability. GIS technologies are becoming a key component of modern engineering practice and require further development at the national level.

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Effect of polypropylene fiber and lime additives on enhancing the properties of swelling soils as foundation Soils of buildings and structures in Syria.

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ABSTRACT

The purpose of a study, presented in this paper, is to improve the methods of strengthening the foundation soils of buildings and structures in the conditions of swelling soils of the Syrian Arab Republic. During the work on this research, the methods of strengthening the foundations and bases of buildings and structures in the conditions of swelling soils were analyzed, and the possibilities of using mixtures of polypropylene fibers and lime to improve the properties of swelling clay soil were investigated. For this purpose, experimental studies were conducted on the effect of polypropylene fiber and lime additives on enhancing the properties of swelling soils as foundation Soils of buildings and structures in Syria. The experimental study establishes the possibility of using lime-containing mixtures to improve the construction properties of swelling clay soil in geological conditions of the Syrian Arab Republic. The results of this research can be used in the design and construction of foundations composed of swelling clay soils. This can lead to an economic effect in the design of foundations considering the improved properties of soils according to the developed methodology.

Keywords: geotechnical research, engineering research, soil properties.

1 INTRODUCTION

The Syrian Arab Republic is located on the west wing of the Fertile Crescent in an important location between the East and the West. Syria has seen some of the oldest and most important civilizations as evidenced by the enormous archaeological discoveries, some of which date back to over 8,000 years BC. In Syria, there are more than 4,500 archaeological sites dated back to different periods of history. In Syria were the first cities in history, such as the ancient kingdom of Ebla, from 2500 to 2400 BC, the Kingdom of Mary about 2900 BC, and Palmyra which flourished in the second half of the first century BC.

Additives of mixtures of polypropylene fibers with lime reduce plasticity index, increases workability and shrinkage limit, reduces shrinkage cracking, eliminates almost all swelling problems, and increases the soil strength (Al-Rawas and Goosen, 2006; Awwad, 2023; Corley, Marsh and Quiring; Dietz and Schürmann,2006; Muntohar and Jiun-Liao, 2001).

2 MATERIALS AND METHODS

Lime is an excellent choice for short-term modification of soil, and it can modify almost all fine-grained soils, but the most dramatic improvement occurs in clay soils of moderate to high plasticity. Modification occurs because calcium cations supplied by the hydrated lime replace the cations normally present on the surface of the clay mineral, promoted by the high pH environment of the lime-water system.

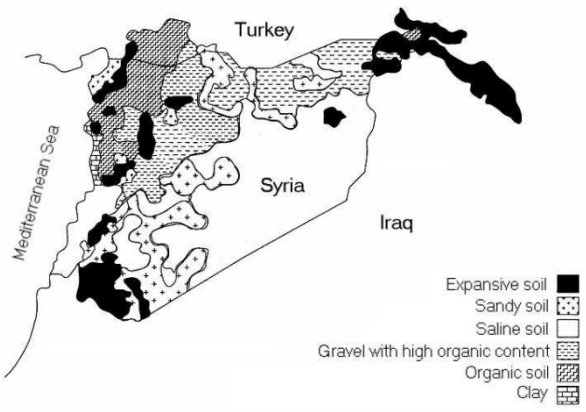


Figure 12 Location of swelling soils in Syria.

Thus, the clay surface mineralogy is altered, producing the following benefits Buhler and Cerato , 2007; Seco et al. 2011; Holtz Gibbs ,1956; Noorzad and Mirmoradi ,2010; Baglari, & Dash, 2013; Mazahir et al., 2020; Салим, Аль-Мансури, 2003).

- Plasticity reduction
- Reduction in moisture-holding capacity (drying)
- Swell reduction
- Improved stability



Figure 13 Laboratory experiments.

In this paper, soil improvement techniques with lime are recommended for restoration of historical building foundations because It is very useful and efficient to improve and stabilize the soil under and surround the foundation without harming its historical characteristics.

A series of laboratory experiments was made on the influence of additives of mixtures of poly-propylene fibers and lime on improving the building properties of swelling clay soil.

Lime was brought from the lime quarries in the province of Hama as limestone and ground with a ball mill at the Adra cement plant, and it was packed in tightly closed bags to isolate it from moisture and transported to the study site.

Experimental data indicates the potential of using soil stabilization techniques as a treatment or prevention method using local materials. Particular attention is paid to the investigation of the impact of a combination of polypropylene fiber and calcium oxide additives on enhancing the structural characteristics of expansive soils.

Experimental data analysis reveals that the incorporation of these additives into expansive soils assists in reducing their expansion. At the same time, a trend towards reduced swelling is observed with increasing concentrations of calcium oxide and polypropylene fiber. An optimum combination of 5% calcium oxide and 0.25% polypropylene fiber is identified.

Based on the results of laboratory experiments, the most suitable model was implemented in the Plaxis 2D program.

3 RESULTS

1. Using additives of mixtures of polypropylene fibers with lime to improve the soil is considered one of the important ways to support and strengthen without harming the historical value.
2. Experimental data analysis has shown that the addition of these additives to swelling soils helps to reduce their swelling. At the same time, there is a trend towards decreased swelling with increasing amounts of lime and polypropylene fiber.
3. The optimum combination is 5% lime and 0.25% polypropylene fiber.
4. The treatment process should not affect the historical mission for coming generations.

Swelling Soils as Foundations for Buildings and Structures in the Changing Climate of Syria: Forecast and Engineering Solutions

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ABSTRACT

Climate change in Syria is accompanied by rising average temperatures, shifts in precipitation patterns, and fluctuations in groundwater levels, significantly affecting the behavior of swelling soils. These processes intensify the deformations of the foundations of buildings and structures built on swelling soils, increasing the risk of damage. This study analyzes climate trends and their impact on the properties of swelling soils, as well as possible future scenarios of soil behavior. Based on field and laboratory data, the risks associated with uneven foundation settlements are assessed, and engineering solutions are proposed, including adaptive foundation types, improved drainage systems, and soil stabilization methods. The findings can contribute to the development of sustainable construction technologies that enhance infrastructure resilience under changing climatic conditions.

Keywords: swelling soils, foundation stability, soil deformation, soil stabilization, climate change, Syria
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1 INTRODUCTION

Swelling soils, particularly expansive clays, pose a significant geotechnical risk in arid and semi-arid regions due to their high sensitivity to moisture variations. In Syria, such soils are widespread and often underlie residential and infrastructure development areas, making them highly relevant to current and future construction practices. Projected climate changes — including temperature rise, irregular precipitation, and increased evapotranspiration — are expected to intensify ground deformations. Despite Syria's vulnerability, few empirical or numerical studies have addressed the interaction between future climate scenarios and foundation performance. This study presents a site-specific numerical simulation of shallow foundations on expansive clays, calibrated with regional laboratory data and projected moisture indices.

2 LITERATURE REVIEW

Climate change amplifies shrink–swell behavior in clay soils, primarily through elevated temperatures and changing rainfall regimes [1]. Studies utilizing the Thornthwaite Moisture Index (TMI) have shown that even moderate shifts in climate can significantly increase site reactivity and impact foundation stability [2,3]. Moreover, rainfall distribution, not just total volume, plays a crucial role in determining the depth and intensity of soil heave [4]. To mitigate these risks, foundation adaptations such as inverted T-shaped and stiffened raft systems have been proposed [5,6]. However, Syria lacks local studies and climate-informed design strategies specific to its geotechnical conditions. This limits the applicability of global findings and highlights the need for localized modeling and validation efforts

3 METHODOLOGY

Laboratory tests were conducted on clay samples from three locations in Syria: Daraa, Latakia, and As Suqaylabiyah. Geotechnical parameters including plasticity, porosity, permeability, and strength were obtained to characterize the swelling potential of the soils.

Climate data were extracted from NASA POWER, WorldClim, and CMIP6 datasets. Future scenarios were modeled using SSP2–4.5 and SSP5–8.5 pathways for the years 2050 and 2100. The Thornthwaite Moisture Index (TMI) was applied to assess climate-induced shifts in soil moisture and site reactivity.

A 3D numerical model was developed in PLAXIS 3D to simulate the response of shallow foundations on expansive clay under variable climatic conditions. The

upper clay layer was modeled as climate-sensitive and calibrated with site-specific data. Infiltration and drying were introduced through boundary conditions to reflect projected moisture extremes. The analysis included sequential phases: initial state, structural loading, wetting, and drying. The model geometry and material behavior were calibrated using both field-observed characteristics and laboratory-derived input parameters

4 RESULTS

An analysis of climatic data from 1985 to 2024 for selected regions in Syria revealed a consistent rise in average annual temperature and high interannual variability in precipitation. The calculated Thornthwaite Moisture Index (TMI) confirmed arid conditions (TMI < 0) and a high potential for shrink–swell activity in clay soils.

Fig. 1 presents a heatmap of relative humidity at 2 meters for the period 2001–2022 at a representative site in Latakia (Lat. 35.52, Lon. 35.78). The image illustrates a visible decline in relative humidity during summer months over the past decade, especially in July–September, indicating intensifying drying trends. These conditions contribute to increased suction and shrinkage in expansive soils, reinforcing the importance of climate-informed geotechnical assessments.

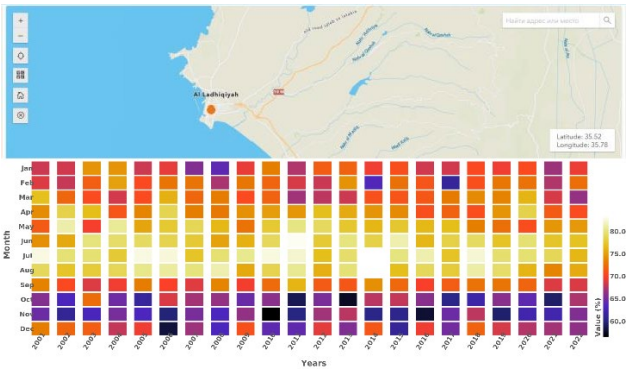


Fig. 1. Heatmap of daily relative humidity at 2 meters for the period 2001–2022, based on NASA POWER data.

According to climate projections based on SSP2–4.5 and SSP5–8.5 scenarios, a decline in TMI by 10–13 points is expected by 2050 and 2100, indicating intensified aridification and increased depth of the active soil zone.

Laboratory analysis of clay samples from climatically vulnerable areas revealed high sensitivity to moisture changes, a pronounced swelling potential, and the presence of montmorillonite minerals. These findings served as input parameters for numerical modeling in PLAXIS 3D.

Three scenarios were simulated: (1) stable seasonal moisture fluctuations, (2) prolonged drought, and (3) extreme rainfall. The results showed that the greatest vertical displacements and differential deformations occurred under sudden wetting following drought conditions. Additionally, zones of maximum stress were found to shift toward the corners and edges of the foundation.

5 CONCLUSIONS

This study confirms the critical impact of climate change on expansive soil behavior and foundation performance in Syria. Projected TMI declines and temperature increases are likely to intensify shrink–swell cycles, particularly under extreme weather variability. Numerical modeling showed that shallow foundations are highly sensitive to both drying and sudden rewetting. These findings emphasize the need for climate-adaptive foundation designs and support the integration of climate projections into geotechnical design standards. Future work should include time-dependent soil modeling, testing of foundation alternatives, and broader regional validation. Additionally, collaboration with planning authorities may help translate research findings into revised construction guidelines and adaptive policies.

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Dealing with Enhanced Expansive Soils in Foundation Construction: Features of Modeling and Designing.

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ABSTRACT

This paper discusses the issues of modeling swelling soils and their implementation in the finite element program. The main emphasis is on the choice of a mathematical model that best allows you to make correct and accurate calculations in such complex soils as swelling. Based on the results of the comparison, the most suitable model was implemented in the Plaxis 2D program. Despite the wide prevalence of swelling soils, design methods have many differences and are not reduced to a single standard that allows you to reduce costs at the design stage. The complexity and variety of conditions for constructing a foundation on swelling soils force engineers to resort to solving each problem separately. New global trends and conditions are changing the established order of things even in construction. The new approach outlined in the study will help to solve this problem with greater ease and accuracy. Based on laboratory data, modeling of foundations on improved expansive soils with lime and polypropylene fiber, allowed us to come to conclusions that significantly improve the behavior of foundations on swelling soils and simplify their design, which opens the possibility of using the research results and the calculation model in engineering practice.

Keywords: swelling soils, foundation stability, soil deformation, soil stabilization, Syria
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1 INTRODUCTION

This paper presents a study on the effectiveness of adding a mixture of polypropylene fiber and lime in order to improve the properties of swelling soil. The aim of this research is to reduce the vulnerability of the soil when it is stabilized using lime alone. In order to investigate the effect of the mixture on the technical characteristics of clay soil, twenty groups of treated samples were created and tested. Each sample consisted of five different proportions of fibers (ranging from 0.0% to 0.35%) and four different proportions of lime (ranging from 0% to 8%). It was discovered that the proportion of lime and fiber has a significant influence on the structural properties of the treated soil.

2 MATERIALS AND METHODS

To perform such calculations, the PLAXIS 2D software package was selected. This software is for numerical modeling of geotechnical processes in two-dimensional space. It is used to analyze the stability of soil structures such as foundations, dams, tunnels and other engineering structures. PLAXIS 2D allows you to model the behavior of soil under various loads and conditions, which helps in design and risk assessment. The program includes tools for analyzing deformation, stability and dynamic behavior of soils. The choice of the program is due to the fact that the foundation model we analyze in the study is quite simple and does not have an additional spatial load.

The main emphasis is on the choice of a mathematical model that best allows for correct and accurate calculations in such complex soils as swelling soils. Based on the comparison results, finite element modeling of the studied soil was performed using the popular and widespread Mohr-Coulomb (MC) model. The main advantage of this mathematical model is that it does not require additional input data, and the parameters obtained from triaxial tests are sufficient for its use [1-6]. These qualities of the model are fully suitable for solving our problem with swelling soils due to the small volume of the foundation, the flat calculation setup, and the absence of spatial loads. The Mohr-Coulomb model takes into account the main properties of the soil, such as elastic behavior under low loads, low material rigidity during failure, failure conditions, and elastic unloading after flow.

Despite the widespread use of swelling soils, design methods have many differences and cannot be reduced to a single standard that allows for reducing costs at the

design stage. The complexity and variety of conditions for constructing a foundation on swelling soils force engineers to resort to solving each problem separately.

The purpose of the simulation was to analyze the vertical movements of the foundation and determine the optimal depth of replacement or stabilization of the soil. Based on the calculated results, it was possible to visualize the active swelling zone and determine the stress state at the interface between the base and the foundation. Additionally, the analysis allowed us to assess the sensitivity of the model to changes in soil parameters.

The data obtained from the simulation served as a basis for engineering recommendations aimed at minimizing the risk of structural deformations and improving the reliability of the structures. These recommendations were based on a thorough understanding of the complex interactions between the foundation, soil, and external forces. This subsystem helps engineers to more accurately assess the behavior of structures in various geological conditions. New global trends and conditions are changing the established order of things even in construction. The new approach outlined in the study will help solve this problem with greater ease and accuracy

3 RESULTS

A series of laboratory tests were conducted to study the effect of adding a mixture of polypropylene fibers and lime to a swelling clay soil to improve its construction properties. Lime was mined from quarries in the province of Hama, ground into powder at the Adra cement plant using a ball mill and packed in sealed bags to eliminate the effects of moisture during transportation to the research site.

The model uses a modified version of the classical Mohr–Coulomb model that can take into account plastic deformations caused by increased humidity, as well as the effects of anisotropy and external constraints from non-swollen layers. In order to improve reliability, parameters that reflect the localization of water saturation and reduce the effect of swelling with depth are activated. This provides a more realistic distribution of deformations within the array.

Numerical modeling in the PLAXIS 2D environment allowed us to recreate the actual engineering conditions of the foundation's interaction with the subsoil as the humidity regime varies. The results showed that the largest swelling deformations occur in the upper part of the subsoil (up to 1.5 meters from the foundation base), where vertical stress is unable to restrict the expansion of the soil. Partial core replacement stabilizes the behavior of the foundation and reduces the risk of the foundation base lifting above regulatory limits.

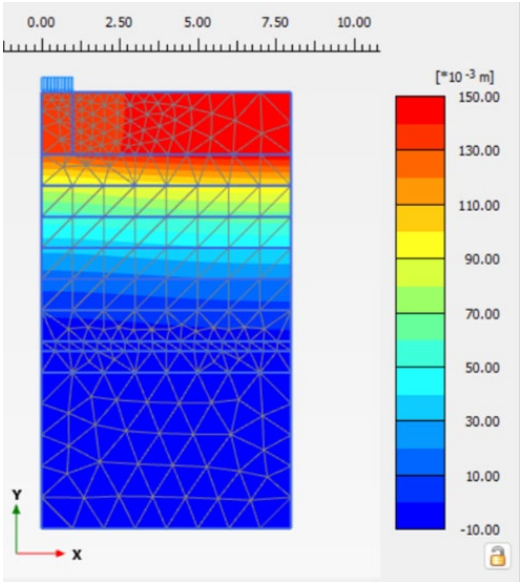


Fig. 1. Result of swelling simulation.

4 CONCLUSIONS

As a result of the modeling of foundations on improved expansive soils using lime and polypropylene fiber, we concluded that these methods significantly improve the performance of foundations on swellable soils and simplify their design. This opens up the possibility of applying the research results and calculation model in engineering practice.

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Model testing of columnar foundations on undermined reinforced foundations

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ABSTRACT

Construction of buildings and structures in undermined areas is among the special ones. Buildings constructed in such conditions as a result of underground mine workings, movement of soil into the excavated space and formation of a displacement trough on the ground surface are subjected to uneven settlements and horizontal deformations in the course of operation. This article presents the results of model tests of separately columnar foundations on a volumetric stand with and without reinforced bases. Model piles considering a scaling factor of 1/40 with a width of 100 mm, a length of 100 mm and height of 20 mm were adopted for the study. Static loading is applied to the model foundation and the corresponding displacements are measured during each loading phase. The test results were compared for load distribution curves and estimates of the maximum load-bearing capacity for both reinforced and non-reinforced bases. The load on the foundation was applied in steps of 78 N and up to a limit load of 624 N. According to the results of the tests, the settlement-load diagrams of the model foundation with and without a reinforced foundation were obtained. Settlement of the foundation with a reinforced base was -12.41 mm. The foundation without a reinforced foundation sank into the ground at a load of 391 N. As a result of the study, the effect of soil erosion on the bearing capacity and settlement of freestanding foundations was established, and the bearing capacity of the reinforced foundation was higher than that of the unreinforced foundation.

Keywords: *instruction, reinforced, geotechnical engineering, foundations, model test, piles*

1 INTRODUCTION

In recent years there have been some changes in the construction of buildings and structures in the undermined areas, as well as in the conditions of the impact of mine workings on the surface objects. The impact of mine workings on buildings and structures has increased, since repeated mining of buildings and structures has a different character of the stress-strain state in comparison with a single mine working. In large cities, the number of buildings with residential and public buildings of higher storeys has increased, including in the undermined areas. In addition, the share of buildings with buried premises has increased, but the existing building codes are not sufficient to address all the issues that arise, especially for sites that have to be developed when there is a shortage of available urban space (Zhukenova, 2018). In Figure 1 shows coal basins in the cities of Zhezkazgan and Kentau.



Figure. 1. Coal basins in the cities of Zhezkazgan and Kentau (Zhukenova, 2018)



Figure 2. Map of Kazakhstan

2 METHODOLOGY OF RESEARCH

Testing of foundations in the undermined areas and investigation of the stress-strain state of soil strata in full-scale conditions are difficult, and such tests are unique (Zhussupbekov et al, 2020). Therefore, in order to find out the influence of massif deformation on the bearing capacity and pliability of pile and slab foundations, in particular, the influence of the degree of horizontal and stepwise deformations of the foundation on the bearing capacity of foundations, laboratory modeling is practically the only experimental method that allows to study the influence of various factors in a wide range of their changes.

3 CONCLUSION

This investigation effectively demonstrates the significant benefits of using reinforced bases for foundations in areas affected by mining-induced ground deformation. Through model tests, it was observed that a reinforced foundation exhibited significantly less settlement (12.41 mm) and a higher bearing capacity compared to an unreinforced foundation, which completely submerged under a lower load (391 N). These findings underscore the critical role of reinforcement in mitigating the negative impacts of soil erosion and uneven ground movements, ultimately enhancing the stability and load-bearing capacity of foundations in challenging undermined regions. The research highlights the need for improved building codes and construction practices in such areas, particularly given the increasing development of multi-story and buried structures.

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Formation of environmentally friendly residential architecture in the cities of Northern Kazakhstan

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ABSTRACT

Abstract. The structure of the modern urban environment represented by a combination of two subsystems: anthropogenic and natural. In vast areas of Northern Kazakhstan, they actively consume natural resources. The range of influence of the city is determined by its scale, naturally, the larger the size of the city, the wider the range of its influence, the greater on the territory of the risks under the influence of the city of imbalance in the ecological situation of the second natural subsystem. Objective: to examine the principles of creating eco-architecture in the conditions of Northern Kazakhstan. To development of principles and requirements for the formation of residential buildings using energy-efficient and environmental systems in Northern Kazakhstan. The main ways to save energy and factors affecting the formation of the ecological environment of the city and residential buildings have identified. Various measures make it possible to improve the ecology of residential buildings and the urban environment. Using rainwater stored in tanks to water trees and lawns. Recycling and use of organic waste as fertilizer. The research methodology provides for the study and generalization of domestic experience in scientific development and design of buildings using energy-efficient systems and the principles of "green" architecture. An analysis of the development and environmental friendliness of residential buildings in Astana carried out. The main ways to save energy and factors affecting the formation of the ecological environment of the city and residential buildings have identified. The regularity and fundamental justification of construction solutions with low, medium and high-energy efficiency in Northern Kazakhstan, and the methods of design. Eco-architecture forms a love of nature, which consists in the transition to energy and resource-saving alternative sources of energy, building materials of natural origin and an increase in the plant mass.

Keywords: environment, ecology, residential architecture, comfort, energy-efficient systems.

1 INTRODUCTION

The paper considers the features of the formation of environmentally friendly residential buildings in an urban environment. The historical and modern experience of designing and building energy-efficient housing analyzed. The features of environmentally friendly buildings for the conditions of the North Kazakhstan region considered, domestic experience analyzed. Describes the basic design principles for environmentally friendly and energy-efficient buildings. The article tells about the relevance of environmentally friendly residential buildings for Kazakhstan, as well as about the role of environmentally friendly development in the development of new territories. Examples from construction practice given, proving the effectiveness of the use of environmentally friendly houses in the development of the territories of Astana.

Human impact on the environment, its change to comfortable conditions and the use of natural resources lead to an increase in the emission of greenhouse gases into the atmosphere. This negatively affects the stability of ecosystems, biodiversity and human safety. To prevent further undesirable consequences of human exposure to the environment, certain measures must take. An effective way to increase the sustainability of the environment can considered the development of the "Green" architecture.

Energy-saving objects today called residential buildings built using economically profitable and fully

justified technical, architectural and engineering solutions.

Energy-saving houses always built according to an individual project. Typical projects in this case will not help, since when planning construction, specialists have to take into account dozens, and sometimes hundreds of individual factors in the use of the new structure.

Energy-saving homes are a relatively new trend in the residential market. These facilities have many advantages and advantages, which provide an ever-growing demand for this type of construction.

The most significant advantages of modern eco-friendly buildings include:

1. Energy independence. Modern houses built using autonomous utilities, which ensure its complete independence from the central systems of heat supply, water supply, and sometimes power supply. Sometimes it is too expensive to do the full non-volatile of an object. In this case, if you have access to central communications, specialists can plan to equip the house with mixed systems. Such homes can use both external energy sources and autonomous ones.

2. Comfort. A person spends most of his life at home; therefore, it is extremely important to organize the most comfortable operating conditions in residential rooms. Energy-saving houses are not only financially profitable, but also very comfortable, cozy; meet all, even the highest requirements of owners. Autonomous engineering systems can provide any living conditions in rooms at minimal financial cost.

3. Energy efficiency. Modern houses can not only be equipped with autonomous energy sources and communications, but also use energy extremely efficiently.

4. Innovation. When building and decorating energy-saving houses, it is customary to use the most modern and sometimes innovative technical and engineering means. The competent selection of facing and building materials also affects the energy efficiency of the structure, as well as the most modern equipment. Therefore, each element of the building selected taking into account its purpose, owner requirements and operating conditions.

5. Sustainability. Modern homes are due to not only the rational use of resources and the use of new materials, but also because they do not harm the environment. Up to 40% of the pollution, entering the atmosphere generated by the combustion of fuel, which provides heating for residential buildings. Energy-efficient homes do not have such a devastating effect on nature [1].

2 THE MAIN PRINCIPLES AND DIRECTIONS OF "GREEN" CONSTRUCTION

The main principles and directions of "Green" construction reflected in many works by foreign and domestic authors, and implemented in many megacities around the world. However, in the realities of Kazakhstan, this principle still poorly implemented in real life.

"Green" architecture characterized by its ecology, safety for both people and the environment. The main principle of this type of architecture can considered to reduce the level of energy consumption, preserve comfortable conditions both for humans and for nature and reduce the environmental impact of the building throughout the entire period of operation. "Green" architecture also helps to avoid the appearance of "Heat Islands" and equalize the temperature of the building with respect to the environment.

The maximum effect of the "Green" architecture revealed when using local natural materials for construction, increasing the efficiency of engineering networks and equipment, and improving design solutions. Zero-energy building is a type of building capable of generating enough energy from renewable sources and using it in equal shares for the entire period of operation. This type of building shows its effectiveness due to the absence of harmful emissions into the atmosphere, a noticeable decrease in energy consumption and payback [2].

The next step to the "Green" architecture, perhaps, can considered the "Green" roofs. It is a multilayer insulation structure of the roof, including a slab, hydro and heat insulation, a drainage layer, as well as a soil and vegetable layer. Intense and extensive green cover can placed on such roofs. In the first case, tall plants with a large root system are mainly used and it will require a large layer of soil up to a meter, as well as constant care of specialists. In the second case, permanent care of the cover is not required, and the thickness of the soil layer can reduced to minimal values, which greatly simplifies the design of such a roof.

This type of design has many advantages both in environmental and energy terms:

- Significant enrichment of air with oxygen and purification of it from harmful impurities, which has a beneficial effect on human health and reduces the number of pulmonary diseases.
- Reduce noise pollution from the urban environment and create comfortable acoustic comfort.
- Significant reduction in the cost of heating the building during the cold period due to the low heat transfer of such a structure.
- Significantly reduce the cost of cooling the building during the warm period due to the natural evaporation of moisture from the surface.
- Significant mitigation of the “Thermal Islands” effect by equalizing the surface temperature with the environment [3].

The use of green facade technology to regulate the temperature and humidity of the building should also

mentioned. Due to the high dust absorption coefficient, noise reduction in large cities and the protection of building structures from atmospheric influences, this technology has a high development potential. This technology also helps to mitigate the effect of the "Heat Islands" and allows you to create a more comfortable temperature regime. In addition, a significant role in the energy efficiency of construction should paid to the thermal rehabilitation of the facades of buildings. A big problem in this direction is the unevenness of the heat protection of the elements of the building shell, the reduction of the thermal uniformity of the enclosing structures. In addition, a large source of heat loss can called translucent elements of facades, through which the internal heat of buildings almost freely leaves the building. The next problem in this direction is the masonry of small-sized structural elements. As is known, such a structure is to a certain extent breathable, especially in the area of vertical seams. Adherence to all the above principles of "Green" construction can lead to an improvement in the state of the urban environment both in terms of ecology and health, and in energy efficiency and payback of construction [4].

3 CONCLUSIONS

In energy-efficient buildings, energy consumption reduced due to improvements in utilities and structural elements. This plays a significant role in the search for architectural and planning solutions for buildings: layout, facades, and aesthetics.

Currently, there are also a number of problems in the practical implementation of energy saving projects with alternative energy sources. Investors themselves solve the training of qualified personnel for innovative enterprises under construction; the problem of the lack of domestic raw materials and components compensated by imports, while working out the possibilities of localizing the entire production process. However, despite all the temporary inconveniences, the implementation of projects for the construction of energy-efficient houses not only favorably affects the environmental situation in the country, but also demonstrates economic efficiency, and therefore attractiveness for private investment.

The home will be as economical as possible if it designed with all energy-saving technologies in mind.

It is better to use the most natural raw materials, the production of which does not require numerous processing stages. It is wood and stone. It is better to give preference to materials produced in the region, because this reduces transportation waste. Passive houses began to build from the products of processing inorganic garbage. These are concrete, glass and metal.

If you once pay attention to the study of energy-saving technologies, think over the eco project at home and invest in it, in subsequent years the cost of maintaining it will be minimal or even strive for zero.

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Evaluation of Strength Properties in Soils Enhanced by Large Crushed Stone Particles

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Abstract: This paper explores how the strength of soil structures relies on various factors affecting the strength characteristics of coarse crushed stone soil. Also presented the dependence of the strength of the soil structure on factors affecting the strength properties of coarse-grained stone soil, the size of soil particles, the shape of particles, the magnitude of normal stresses, the dependence of strength properties on the density of the structure, the influence of pressure on the displacement resistance, the dependence of the displacement angle on the laying density.

During this research the tests were carried out in a triaxial compression vacuum apparatus with a height of 500 mm and a diameter of 250 mm. The axial pressure on the sample did not exceed 7.5 kg/cm2 , and the lateral pressure did not exceed 0.9 kg/cm2, which describes methodology of this research.

The strength properties remain consistently stable within the specified range when the material is deposited within the dam body at densities aligned with the grain composition.

Regarding strength characteristics, the stone material exhibits minimal variation, with differences primarily observed in the coefficient of cohesion (maintaining a strength level within 5% of the absolute value).

The coefficient of cohesion demonstrates an increase as the soil composition of fine particles approaches its optimal composition.

Keywords: broken rock, rocky soil, soil structure, soil particles, magnitude of stress, pressure, displacement, strength.

INTRODUCTION

Based on information gleaned from research publications conducted both in the Commonwealth of Independent States and abroad, several factors influence the strength characteristics of large crushed rock soils. These factors include the integrity of the soil structure, the size and shape of soil particles, and the levels of normal stress [1-9]. Researchers have underscored the significance of structural density in determining the strength properties of large crushed stone soils and sands, particularly in the context of tunnel construction in mountainous regions. They unanimously assert that as the total volumetric weight of a given soil skeleton increases, thereby reducing its porosity, the angle of internal friction also increases correspondingly. This correlation mirrors findings by S.M. Atayya [10] in the examination of Nurek limestone.

METHODS OF RESEARCH

The tests were carried out in a triaxial compression vacuum apparatus with a height of 500 mm and a diameter of 250 mm. The axial pressure on the sample did not exceed 7.5 kg/cm^2 , and the lateral pressure did not exceed 0.9 kg/cm^2 .

Table 1. Effect of pressure on shear resistance

Material	Sizes of fractions, mm	Initial density, ρ, g/cm ^{3}	n, %	Lateral pressure, σ_3, kg/cm ^{2}	Axial pressure, σ_1, kg/cm ^{2}	Angle of internal friction,φ
Nurek limestone	5-10	1.50	45	0.37	2.45	51 °20′
			45.5	0.36	2.7	50 °30′
			45.5	0.6	4.5	50°
			45	0.6	4.6	50 °30′
			46	0.9	6.0	48°
			45	0.9	6.3	48 °40′
	10-20	1.50	45.5	0.3	2.4	51 °30′
			45	0.3	2.44	51 °15′

			45.5	0.6	4.6	50 °30′
			45	0.6	4.5	50°
			46	0.9	5.8	47°
			46	0.9	6.2	48 °30′
	20-30	1.50	45	0.3	2.75	53°
			45.5	0.6	5.0	51 °30′
			45	0.9	6.4	49°
	30-55	1.77	35	0.5	4.1	52°
			35	0.9	7.5	52°

By comparing (comparing) the results shown in Table 1, we get the following: for 5-10 mm fractions, when σ_1 increases φ from 2.45 to 6-6.3 kg/cm^2 · 2-2° decreases by 30, for 10-20 mm fractions, this value 2-2° was 30, and for fractions 20-30 mm it was equal to -4° .Thus, the effect of these pressures on the angle of internal friction was observed even at normal pressures that were not too large compared to each other.

Consists of the products of weathering of argillaceous shales filled with sandstones, limestones, and tufforphyritic rocks [11-26]. The strength of large crushed stone soils is determined in laboratory conditions in large-scale rigs. the strength of large crushed stone soils is rarely determined directly in natural conditions, because their methodology is not fully developed.

In the mining of rocky rocks, depending on the strength of the rocks being mined, the amount of cracks, and the layers of weaker rocks in the base massif, a rock mass with different coefficients of polyhedra is obtained. The recommended granular composition of the pile mass for laying on the base prism of the Piskom dam: granular composition No. 1; 2; It is divided into three zones by curves 3 and 4 (Fig. 1).

RESULTS

These compositions differ from each other in terms of the size of fine (fr.<5mm) particles of the mixture, large fractions, and coefficients of heterogeneity. Therefore, it is necessary to experimentally establish the calculation characteristics included in the project in each specific case . The task (problem) carried out includes the establishment of calculation characteristics of the rock mass for the most probable granular composition and the assessment of the change of the angle of displacement in the limit of 1-4 granular compositions. When conducting BPI (drilling and blasting operations), it is ~very difficult to obtain the minimum amount of 5% of small (fr.<5mm) particles of ground (limestones) in the grain composition of the rock mass, therefore, the range between 1-3 curves is adopted for the main composition. The curve of grain composition No. 2 is the average grain composition for this range, the calculation characteristics of which are determined by the appropriate amount of displacements according to the regulatory requirements for this composition.

For grain sizes 1-4, two tests were performed at each recorded load and density corresponding to the grain size. Such tests can only give approximate values of strength and allow evaluation of strength parameters within the limits of selected grain sizes of the stone. Experiments were carried out on large-scale devices, the use of such devices made it possible to conduct tests on the material containing 140 mm fractions fulfilling the $5D_{max} \leq d_{min}$ condition of the device. The presence of fractions of 500-900 mm in the natural material forces the transition to the model composition of the material. The granular compositions of the soil obtained on the basis of experiments and their model mixtures are shown in Fig. 1. The material presented for testing was divided into fractions that form model mixtures. Various compressive loads of samples: 2; 0.4; 0.8; Displacement at 1.2 MPa was carried out according to the guidelines. Based on the results of the displacement tests, Table 2 was created, according to this table, graphs of the displacement coefficient depending on the vertical load and density were constructed (Fig. 1).

By cutting these graphs with horizontal straight lines, it was determined that the displacement angle depends on the density of the rock mass and the vertical stresses . It follows from the graphs that it is possible to regulate the strength properties of the material at different stresses with the deposit density of the rock mass. Here $\rho_{d\leq140}$- the density of the model mixture, t/m^3 ; ρ_d^s- density of individual fractions (ρ_d^s=2.66 t/m^3); $m_{<140}$ – amount of fractions smaller than 140 mm in natural soil.

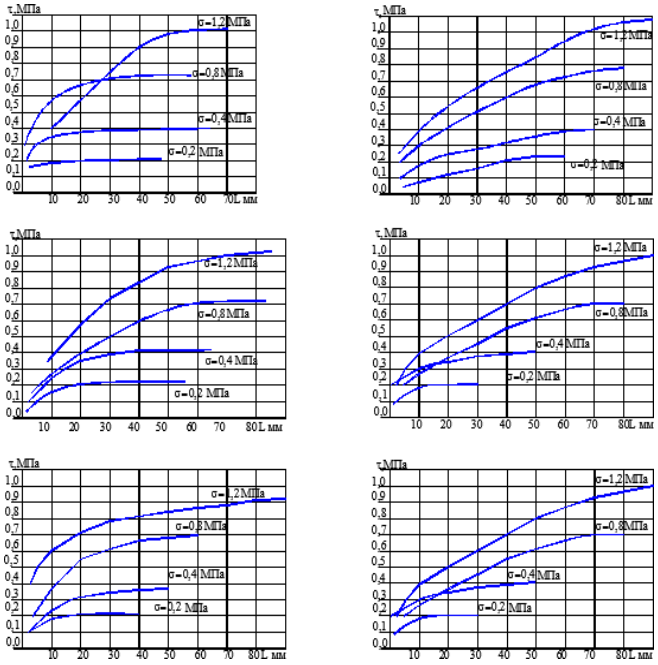


FIGURE 1. Granularity (m<5=18%) results of tests №1÷№24, carried out on the test of quarry soil № 7 with respect to displacement for curve № 2 of composition

Table 2. Results of the tests conducted to determine the displacement characteristics of pit soil No. 7

№	ρ_n -initial, ρ_k -final densities	Vertical loads, MPa							
		σ=0.2		σ=0.4		σ=0.8		σ=1.2	
		K sd	τ, MPa	K sd	τ, MPa	K sd	τ, MPa	K sd	τ, MPa
1	ρ_n	1,125	0.225	1.00	0.400	0.913	0.730	0.85	1.02
	ρ_k	1.92		1.94		2.07		2.05	
2	ρ_n	1,175	0.235	1,075	0.430	0.92	0.736	0.875	1.05
	ρ_k	1.96		2.05		2.02		2.10	
3	ρ_n	1.10	0.220	0.95	0.380	0.875	0.700	0.788	0.94
	ρ_k	1.87		1.87		1.94		1.99	
4	ρ_n	1.15	0.230	1,025	0.410	0.969	0.775	0.896	1.07
	ρ_k	1.93		1.94		2.02		2.08	
5	ρ_n	1,075	0.215	0.99	0.396	0.90	0.720	0.875	1.05
	ρ_k	1.89		1.90		1.96		2.06	
6	ρ_n	1.05	0.210	1.86	0.371	0.85	0.680	0.83	1,0
	ρ_k	1.85		1.99		1.93		2.03	

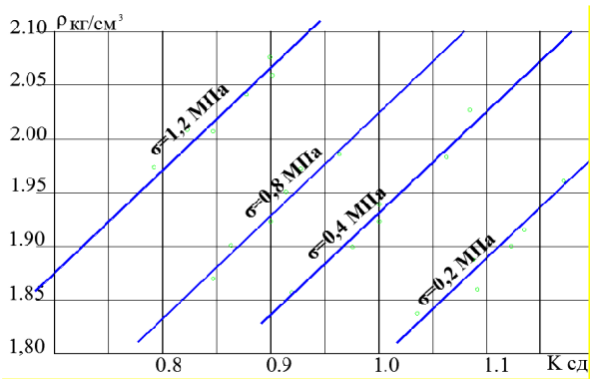


FIGURE 2. Graphs of dependence of angles on density and vertical loads (mixture No. 2, m $_{<5}$ =18%)

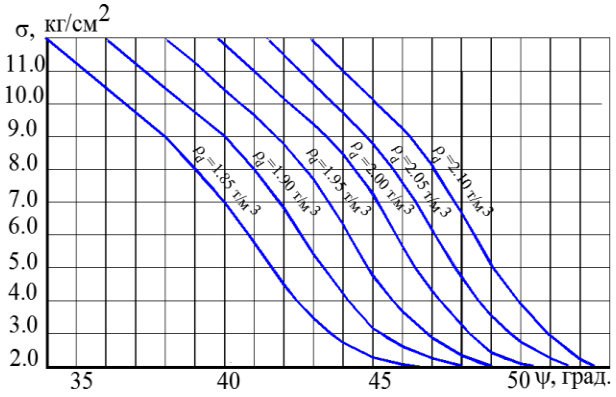


FIGURE. 3. Graphs of dependence of rock mass displacement angles of quarry No. 7 on normal stresses and density of deposited material (curve No. 2, mixture No. 2, m<5=18%)

Determining the required densities of laying the soil mass for natural soil αwas carried out at the time of determining the densities of stone laying depending on the grain composition and a reliable probability =0.95, mixture №4, m $_{<5}$ =5%, ρ_d^{mp}=1.75 t/m ^{3} ; mixture #3, m $_{<5}$ =10%, ρ_d^{mp}=1.84 t/m ^{3} ; mixture №2, m $_{<5}$ =18%, ρ_d^{mp}=2.02 t/m ^{3} ; smes #1, m $_{<5}$ =25%, ρ_d^{mp}=2.18 t/m ^{3} ;

tg φ^n i c ^{n} is presented in Table 4. The calculation of the normative and computational characteristics of τ=σtgφ+c was carried out according to the formulas of the method of least squares of linear correlations in the form of "tgφ" and "s" for the entire set of data obtained from experiments.

Table 4. Calculation of normative values

№	σ_i , kgs/cm ^{2}	σ_i^2, kgs/cm ^{2}	τ_i , kgs/cm ^{2}	$\sigma_i \cdot \tau_i$ kgs/cm ^{2}
1	2	4	2.20	4.40
2	2	4	2.30	4.60
3	2	4	2.15	4.30
4	2	4	2.10	4.20
5	2	4	2.35	4.70
6	2	4	2.25	4.50
7	4	16	3.80	15.20
8	4	16	4.0	16.40
9	4	16	3.96	15.84
10	4	16	3.71	14.84
11	4	16	4.30	17.20
12	4	16	4.00	16.00
13	8	64	7.0	56.0
14	8	64	7.75	62.0
15	8	64	7.20	57.6
16	8	64	6.80	54.4
17	8	64	7.36	58.88
18	8	64	7.30	58.40
19	12	144	9.45	113.40
20	12	144	10.75	129.0
21	12	144	10.50	126.0
22	12	144	10.00	120.0
23	12	144	10.50	126.0
24	12	144	10.20	122.4
∑	**156**	**1368**	**142.03**	**1206.26**

The calculated values of the strength were obtained by applying pressure to the soil reserve coefficient, the physical essence of which is that the actual (actual) values of the strength characteristics do not exceed the limits of the limit strengths of the soil strength with a reasonable probability. The soil reserve coefficient is calculated by summing up the measurements of vertical stresses and displacement loads in pairs from σ_{min} to σ_{max} , and it depends on the change (variation) of these quantities, i.e., the "V" coefficient.

For the average curve of granular composition No. 2, we obtain the following using the calculation characteristics of the rock mass strength and the methodical instructions for calculating the dependences on the strength characteristics in the statically stressed state:

$$tg\psi = tg\varphi^{P} + \frac{c^{P}}{\sigma} \quad (1)$$

Table 5. Results of calculation of strength characteristics of rock mass in statically stressed condition

σ, MPa	0.2	0.4	0.6	0.8	1.0	1.2
tgψ	1.103	0.932	0.874	0.846	0.828	0.817
ψ°	47.8	43.0	41.2	40.2	39.6	39.2

As can be seen from the table, according to it, as the stresses increase, the strength decreases, which, in turn, allows us to conclude that the support prism of the dam should be restored from the state of tension, that is, in order to obtain homogeneity in terms of physical and mechanical properties of the material, the material should be distributed to different zones of the dam body. laying of this material is subject to requirements that ensure homogeneity of strength. According to the results of the conducted tests, the predicted approximate indicators of strength in the range of granularity material were obtained. Tests were carried out with two repetitions for each load on the sum of even values of normal and shear stresses for each granular composition of "Rybki" bounded by curves 1-4.

For each recorded value of the normal stress, the ultimate yield stress was determined based on the results obtained by the shearing (displacement) method. Displacement tests were performed in duplicate for each curve of #1-4 grain compositions. In the strength tests of the samples (mixtures No. 1-4) at the shear strength, after applying all steps of the vertical load of 0.2-3 MPa, the soil density in the pile that is more compatible with the shear moment than the initial density or the density after the shear was adopted. Experiments were conducted at densities close to the allowable density for the selected average range of densities. When carrying out experimental experiments with respect to displacement, the sample is placed in the installation by manual compaction of the soil and the same sample is tested, so the compactions performed cannot have a constant strength. It is worth noting the improper compaction of the samples in the optimal size of overcompaction when the amount of fine-grained soil in the soil composition is small. Bringing the obtained values of strength properties to the limits of single-grain content and required density was carried out according to the empirical formula.

$$tg\psi_2 = tg\psi_1\left(\frac{\rho_{d2}}{\rho_{d1}}\right)^{2/3} \quad (2)$$

where tg ψ_1 ; tg ψ_2 – required and obtained in experiments ρ_{d1} ; ρdisplacement angles corresponding to ÷$_{d2}$. This calculation was performed for the required densities for each grain composition represented by curves 1 4 . The results of the displacement tests for each curve are shown in Table 6.

Table 6. Results obtained from tests against displacement of soil mass №7 for each granular composition represented by curves №1÷4

Numbers of mixtures with fractions of less than 5 mm	Vertical load, MPa			
	0.2	0.4	0.8	1.2
t/r $_d$ τ-horizontal load, ρ_d -density (t/m ^{3})				
Mixture №3 m $_{<5}$ =10%	2,35 ; 2,10 / 1,81 ; 1,80	4,10 ; 3,90 / 1,84 ; 1,81	7,80 ; 7,20 / 1,88 ; 1,84	10,10 ; 10,50 / 1,89 ; 1,90
Mixture №2 m $_{<5}$ =18%	2,25 ; 2,35 / 1,90 ; 1,96	4,00 ; 4,3 / 1,93 ; 2,02	7,30 ; 7,36 / 1,95 ; 1,98	10,20 ; 10,5 / 2,01 ; 2,07
Mixture №1 m $_{<5}$ =25%	2,52 ; 2,30 / 1,96 ; 2,01	4,10 ; 4,6 / 1,97 ; 2,03	6,8 ; 8,3 / 2,00 ; 2,07	11,00 ; 11,0 / 2,04 ; 2,10

According to the results of tests of different granularities of the soil against displacement and the parameters of the displacements to the densities required for laying soils of this composition, ÷ diagrams of displacements were obtained for the entire range of changes in the granularity (1- 4 curves) (Fig. 5).

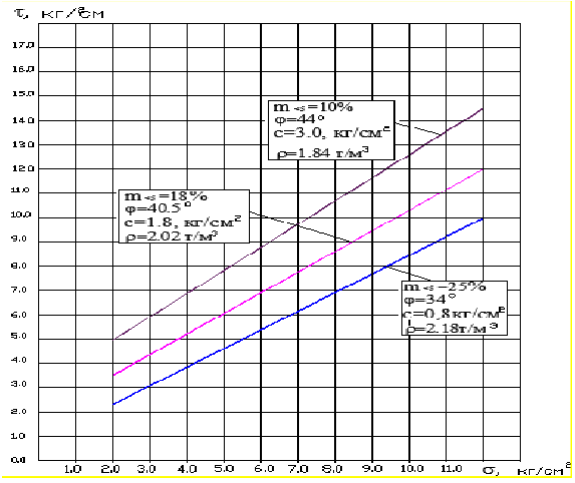


FIGURE 5. Dependence of t and s in different percent granularity of fine (fr. m $_{<5\ mm)}$ particle fractions

CONCLUSION

The strength properties remain consistently stable within the specified range when the material is deposited within the dam body at densities aligned with the grain composition. Regarding strength characteristics, the stone material exhibits minimal variation, with differences primarily observed in the coefficient of cohesion (maintaining a strength level within 5% of the absolute value). This coefficient demonstrates an increase as the soil composition of fine particles approaches its optimal composition.

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Using geoinformation systems (GIS) for mapping the groundwater regime of Astana

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ABSTRACT

Geographic information systems (GIS) are an important tool for studying and monitoring groundwater regimes. These technologies allow analyzing the spatial distribution of groundwater, its quantitative and qualitative characteristics, and predicting changes caused by natural and anthropogenic factors. In the context of active development of cities such as Astana, monitoring the groundwater level is of particular importance, since its fluctuations can lead to flooding, soil subsidence, and damage to infrastructure. In addition, the groundwater level affects the stability of buildings and roads, as well as the general condition of the urban environment. The use of GIS in hydrogeological studies allows for the integration of data from various sources, including field observations, hydrogeological maps, satellite images, and mathematical modeling results. This enables a more accurate assessment of the current state of groundwater and prediction of its future behavior. GIS allows for the identification of areas with an increased risk of flooding, modeling groundwater movement, and analyzing the possible impact of climate change on the hydrogeological situation. One of the key mapping methods is spatial modeling, which allows the creation of digital maps that display groundwater levels, their movement directions, areas of possible flooding, and areas with an increased risk of changes in the chemical composition of the water. Geostatistical analysis methods, such as data interpolation, are also used, which allows obtaining information about the territory even with a limited number of measurements. GIS also makes it possible to take into account climatic factors such as precipitation and temperature, which can affect the groundwater level. The research results confirm that GIS is an effective tool for monitoring and managing groundwater. It allows not only to track the current state of groundwater, but also to develop strategies to minimize the risks associated with their change. Data obtained using GIS can be used in the planning of new infrastructure facilities, the development of water disposal systems and the implementation of environmental protection measures. In addition, they help city authorities make informed decisions on the development of territories, taking into account possible hydrogeological risks. The introduction of GIS in water resource management processes contributes to a more rational use of natural resources and sustainable development of the urban environment, providing protection from the negative consequences of changing hydrogeological conditions.

Keywords: geographic information systems (GIS), groundwater, hydrogeological regime, mapping, monitoring of water resources, dynamics of changes, data visualization

1 INTRODUCTION

This section highlights the relevance of applying Geographic Information Systems (GIS) for mapping the groundwater regime in urbanized areas such as the city of Astana. It discusses the challenges of water resource management under intensive urbanization, the impact of urban development on the water regime, and the need for integrating GIS to address these issues. The aim of the study is to develop a GIS-based methodology for groundwater mapping to assess their condition and predict changes.

2 GLOBAL TRENDS IN THE DEVELOPMENT OF GIS FOR HYDROGEOLOGICAL STUDIES

This section covers global trends in the use of GIS for analyzing hydrogeological processes. It particularly examines innovations in aquifer modeling, the application of remote sensing, and high-tech tools for groundwater level monitoring. Examples of successful studies are provided, where GIS was used for forecasting changes in the water regime and managing water resources.

3 KEY DESIGN CRITERIA FOR GIS IN HYDROGEOLOGICAL RESEARCH

The section focuses on the fundamental principles of developing GIS for hydrogeological research. It discusses key design stages, including the collection and processing of initial data, selection of appropriate geoinformation tools and interpolation methods, and the creation of spatial layers for analysis. Special attention is given to data quality and the importance of accurate geospatial referencing.

4 DEVELOPMENT AND IMPLEMENTATION OF GIS IN KAZAKHSTAN

This section describes the experience of implementing GIS in Kazakhstan for solving engineering tasks, including in the field of hydrogeology. It provides an analysis of existing groundwater monitoring systems and their adaptation to national standards and specific conditions. It also discusses issues related to data shortages and infrastructure limitations, as well as measures for advancing GIS in the country.

5 PROSPECTS FOR THE DEVELOPMENT OF GEOGRAPHIC INFORMATION SYSTEMS IN KAZAKHSTAN

This section discusses the future prospects of GIS application and development in Kazakhstan, including improved data accessibility, enhanced spatial analysis quality, and the growing role of GIS in water resource management. In particular, it forecasts the expanded use of GIS for predicting groundwater level changes under climate change and urbanization, as well as for monitoring water supply and drainage systems in large cities.

6 CONCLUSION

The conclusion summarizes the work, emphasizing the importance of further development and implementation of GIS for sustainable water resource management in growing urban areas. Recommendations are provided for the practical use of the findings and proposals for improving groundwater monitoring in Kazakhstan through the use of modern geoinformation technologies.

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Testing of piles of modern complex in Kazakhstan

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ABSTRACT

This study evaluates the load-bearing capacity of pile foundations, with a focus on those used in the Diamond Multi-Complex. It presents the results of a field investigation conducted in accordance with current geotechnical standards for load-bearing capacity assessment. The processed data on the load-bearing capacity of piles is critical to ensuring the reliability of the structure and the long-term durability of the facility. Dynamic load tests were conducted using the impact wave equation theory, which provides an accurate understanding of the physical processes involved in pile driving. Three groups of pile foundations were subjected to dynamic tests, with the maximum recorded load reaching 860 kN and the corresponding settlement being 0.3 mm. The tests were conducted using the Pile Driving Analyzer (PDA) method in combination with the CAPWAP software package. This methodology allows for a detailed analysis of the behavior of piles under dynamic loading. Conducting dynamic tests during pile installation provides important data on the relationship between load-bearing capacity and driving depth, allowing real-time adjustment of design parameters and optimal selection of pile driving equipment.

Keywords: measure, test, pile, load

1 INTRODUCTION

Field testing of Piles at the “Diamond” complex, Astana.

The target of the tests is to determine the bearing capacity of two driven piles No.TR7, No.TR18 grade C9.30 immersed in the ground to a depth of 4.1–3.5 m.

Field work was performed in accordance with the normative of GOST 5686-12 "Soils. Methods of field testing with piles".

2 TESTING MATERIAL AND METHODOLOGY

Two driven piles No.TR7, No.TR18 grade C9.30 were subjected to tests, immersed in the ground to a depth of 4.1 and 3.5 m, respectively, to the abs. mark of their bottom 340.51 and 341.34 and one pile No. TR34 grade C11.30, immersed in the ground to a depth of 3.6 m to the abs. mark of their The bottom is 341.37 (Fig.1).

According to the assignment, two previously driven C90.30 test piles numbered TR7, TR18 and one C110.30 test pile numbered TR34 were tested. The above-mentioned piles were driven on 2021 by the Junttan PM-25 pile driving unit with an NNK-5A hydraulic hammer with an impact weight of 5 000 kg.

The installation of the test and loading stand and the loading of blocks was carried out in summer period.

The load on the pile was created using a 200-ton hydraulic jack "Energpred DG200P150" with a pumping station "Enerpac P462", resting on a test and loading stand with railway blocks weighing 120.0 tons. The load was recorded with a pressure gauge up to 1000 atm (kgf/cm^2) of the MA100VU100 type with a division price of 20 atm (kgf/cm^2) and a digital dynamometer of the "Load Cell" type connected to the SLT2 monitoring system.

The movements of each pile were measured by two 6PAO deflection meters with a scale division of 0.01 mm, and also by four digital electronic displacement converters of type 027DG1, 027DG2, 027DG3, 027DG4 working in conjunction with SLT2.

The monitoring devices of the SLT2 system are specially designed to monitor the testing of piles with static load in order to obtain results in accordance with Eurocode 7 standards. This system makes it possible to monitor the testing of piles with static load at a distance of up to 25 m. This system allows the personnel (testers) to remotely monitor the pile sediment and the current load on the pile without approaching the

potentially dangerous area of the experimental site structure, where the system is under high pressure and load.

The protocols and schedules of static testing of piles by load stages and time, respectively are given (Fig.2).



Fig. 1. “Diamond” complex.



Fig. 2. The testing process

3 TESTING RESULTS AND DISCUSSIONS

The piles were tested with static stepwise increasing loads, the first four stages of 123 kN (40 atm), then 61.5 kN (20 atm) and brought to a maximum load of 861 kN.

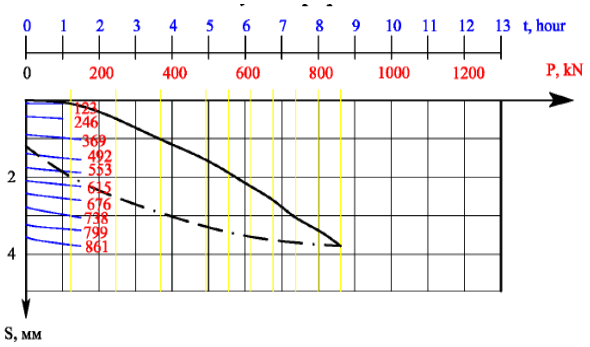


Fig. 3. Graph of pile TR7

Static tests of two driven piles No.TR7, No.TR18 grade C9.30 immersed in the ground to a depth of 4.1 and 3.5m, respectively, to the abs. mark of their bottom 340.51 and 341.34 and one pile No.TR34 grade C11.30 immersed in the ground to a depth of 3.6 m to the abs. mark of their the bottom is 341.37 in the summer of 2021 (Figures 3-5).

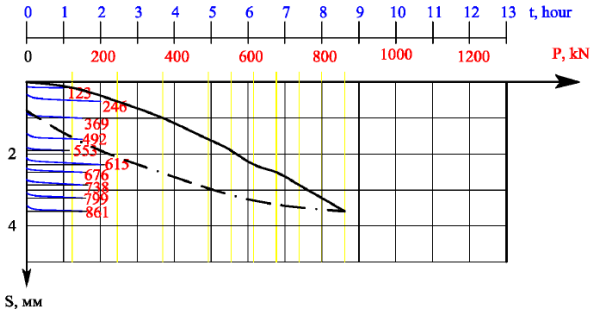


Fig. 4. Graph of pile TR 18

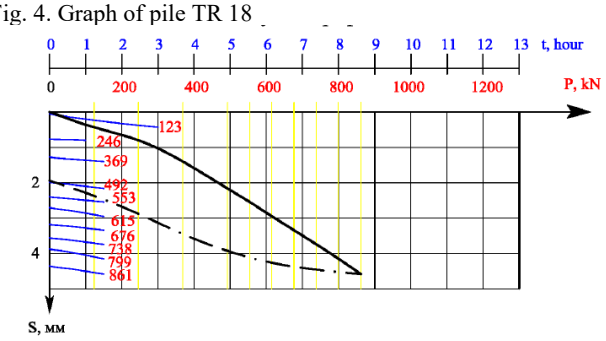


Fig. 5. Graph of pile TR 34

The bearing capacity of the piles according to the results of their static load tests at the above-mentioned construction site was 799 (seven hundred and ninety-nine) kN.

Permissible load on the pile, taking into account the reliability coefficient yk = 1.2 according to standard 5.01-103-2013 "Pile foundations" should be assumed to be equal to 666 kN.

4 CONCLUSIONS

In practice, the method of testing piles by static load, based on the application of a step load, was used. Using the load allows you to more accurately describe the processes occurring during pile driving. During the research work, an analysis of 3 pile foundations was carried out, which were considered for static testing. The maximum load on the piles is 861 kN the settlement is max 4.3 mm.

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Stages of formation of settlements in the suburban area of Astana

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ABSTRACT

In the context of rapid globalization and increasing urbanization, the processes of development and transformation of urban agglomerations are becoming particularly relevant. Kazakhstan, as a dynamically developing country, is also subject to these trends, which is especially noticeable in the example of the Astana agglomeration.
Suburban settlements within the Astana agglomeration play an important role in ensuring the sustainable development of the region. They serve not only as a place of residence for a significant part of the population, but also perform a number of economic, social and environmental functions.
Understanding the stages of formation of these settlements will help identify the key factors influencing their development and improve urban planning policy. At the same time, it is necessary to take into account not only economic, but also environmental, social and cultural aspects, which makes the study multifaceted.
In addition, the analysis of historical stages allows you to identify all the changes. The study of the formation of suburban settlements allows us to predict the trends of their development in the future and find solutions to emerging problems.

Keywords: agglomeration, suburban areas, settlements, stages of formation, Astana, urbanization, migration

1 INTRODUCTION

Almost twenty years have passed since Astana received the status of the capital of Kazakhstan in 1997, which has served as a powerful impetus for significant population growth both in the city itself and in the surrounding suburban areas. During this period, large-scale changes took place, including fundamental transformations in the planning structure and infrastructure of the entire region, which was the result of the construction of the new capital. This process caused an increase in construction and economic activity, increased the birth rate in the region and facilitated migration flows from all over the republic. The resettlement of the population to Astana was a complex and multifaceted process, covering the most important aspects of urbanization, migration processes, urban planning, development of social infrastructure and ensuring environmental sustainability of the region. The analysis of this stage requires a comprehensive approach, including an assessment of its impact on socio-economic development, adaptation of infrastructure to new challenges and the formation of a comfortable urban environment.

Despite the considerable interest in the study of urban agglomerations in the world scientific literature, the issues of development and transformation of suburban territories in the context of Kazakhstan, in particular the Astana agglomeration, remain insufficiently studied. Existing research often focuses on general trends in urbanization, without taking into account the specific features and local factors affecting the development of suburban settlements (Abilov&Kusainova&Makhrova et al.2017).

Scientists involved in modern urbanization processes study the transformation of suburban areas, the integration of suburbs into agglomeration, transport and infrastructure issues. Chikanaev A.S., Iskakov U.M., Zhunusov B.A. are prominent researchers who have made significant contributions to the study of the Astana agglomeration and related processes. Their work covers various aspects of urbanization, migration and economic development, which allows for a deeper understanding of the dynamics of socio-economic changes in the region (Daueshova et al.2014).

Sociologists Makhrova A.G., Safronov S.G., focuses on issues of migration, social dynamics and quality of life in suburban areas of Astana. In their research, they identify the following key points:

- migration processes to Astana contribute to a change in the social structure of the population, which creates a diversity of ethnic and cultural groups that influence socialization and interaction between residents (Makhrova & Safronov et al.2021);

- in conditions of accelerated urbanization, there are changes in the quality of life, which is associated with an increase in the need for social infrastructure, such as education, healthcare and leisure.;

- the level of social integration of migrants, as well as the availability of social services, are important factors determining the success of adaptation of these groups in the urban environment (Makhrova&Safronov&Abilov et al.2023).

In their work, researchers Akimov V.V., Moldumarova Zh.E., Muzyka O.S. analyzed various approaches to defining the boundaries of the Astana agglomeration and studied their features and prospects. Four options for delineating the agglomeration were considered, with special attention being paid to the third option, which requires clarification after the development of the project plan within the framework of the interregional scheme. The authors collected the initial data and prepared the materials in accordance with the regional development program until 2020. As a result of the study, conclusions were drawn about the need to focus on settlement channels and transport corridors when developing long-term plans until 2030 to attract investment and optimize migration flows (Afontsev& Zubarevich et al.2012).

The relevance of the topic "Stages of the formation of settlements in the suburban area of Astana". It is caused by a variety of factors affecting the socio-economic and environmental development of the region.

The purpose of the study "Stages of the formation of settlements in the suburban area of Astana" can be formulated as follows:

To study and evaluate the socio-economic, urban planning and ecological condition of suburban settlements of the Astana agglomeration in order to identify the main stages of settlement in suburban areas.

2 MATERIALS AND METHODS

The methodology for researching the development and transformation of suburban areas should be comprehensive and take into account the specifics of this facility. Within the framework of this study, it is proposed to use the following methods: analysis and study of suburban settlements based on statistical data provided by the Department of the Bureau of National Statistics of the Agency for Strategic Planning and Reforms of the Republic of Kazakhstan.

3 RESULTS AND DISCUSSION

The study of the division of the Astana agglomeration into different stages is important for understanding its development dynamics. Such an analysis allows us to trace how the structure and level of development of the agglomeration has changed over time, to identify the main trends and problem areas. In addition, it helps to determine which factors and initiatives have contributed to or, conversely, hindered its growth at each stage. This is important for developing more effective development strategies, as knowledge of the stages allows you to create targeted programs that take into account the characteristics and needs of each period. Such an analysis also helps to identify weaknesses and identify priority areas for improving infrastructure, improving living conditions and attracting investment. In general, the division of the agglomeration into stages contributes to a more accurate and meaningful management of its development, providing a strategic basis for the long-term and balanced development of Astana and the surrounding territories (Perkova&Perkova et al.2022).

4 CONCLUSIONS

As a result of the analysis of the evolution of the habitat, it was found that any changes in the expansion of suburban settlements are uniquely associated with the growth and decline of the socio-economic trend of a certain period. It is these conditions that have a direct impact on geography and habitat.

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The effect of excess pore pressure in sandy soil on the embedded pipeline during earthquakes

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ABSTRACT

This study investigates the seismic response of a layered soil profile incorporating an embedded steel pipeline, emphasizing the excess pore pressure ratio (r_u) behavior during a multiple fault rupture event. The numerical analysis was conducted using OpenSees with the PDMY02 constitutive model. A 4×4 m square steel pipeline was embedded 2 meters below the ground surface within a 100×30 m layered soil domain. The groundwater table was defined at a depth of 6 meters. The model was subjected to the Sakarya record from the 1999 Kocaeli Earthquake, which was applied as the base input motion. Before dynamic simulations, the PDMY02 material model was verified to ensure its reliability. The results revealed that the embedded tunnel influenced the spatial distribution of r_u, particularly in the vicinity of the structure, indicating localized changes in liquefaction potential.

Keywords: Excess pore pressure, pipeline, opensees, liquefaction, numerical modeling

1 INTRODUCTION

Liquefaction in saturated sandy soils poses serious risks to underground infrastructure during earthquakes. Multi-fault rupture events, which involve simultaneous or cascading fault segment ruptures, produce complex and intense ground motions, thereby increasing the risks to buried structures. Understanding the interaction between excess pore pressure development and embedded structures is crucial for resilient design. This study examines how a buried pipeline affects r_u distribution in a layered soil profile, using advanced numerical modeling to capture cyclic mobility and liquefaction phenomena.

2 NUMERICAL MODELING and VERIFICATION

The soil profile was developed using borehole data from the Gölbaşı region. A 2D plane strain finite element model was generated in OpenSees, with a 4×4 m square pipeline placed 2 m below the surface and the groundwater table at 6 m depth. The soil was modeled with PDMY02 and PIMY constitutive models, discretized using 9-4 node QuadUP elements with finer mesh near the pipeline. Free-field boundary conditions were applied, and the Sakarya record served as input motion. To verify the PDMY02 model, simulation results were compared with laboratory cyclic simple shear test data. Good agreement confirmed the model's capability in predicting stress-strain behavior and pore pressure generation under seismic loading.

3 INPUT MOTION RECORD

The Sakarya record from the 1999 Kocaeli Earthquake (Mw 7.4) was used, featuring maximum bedrock accelerations of 0.32 g (NS), 0.41 g (EW), and 0.21 g (vertical). This strong motion represents realistic conditions for assessing liquefaction potential and soil-structure interaction.

4 RESULTS

Results were analyzed at various depths and at two reference lines 1 m away from both sides of the pipeline. Excess pore pressure generation was most intense within the loose sand layer at ~11 m depth, leading to

liquefaction and settlement near the structure. Horizontal displacements indicated both sides of the pipeline moved in the same direction, while vertical displacements were notable up to ~11 m depth due to liquefaction, with negligible movements at deeper levels.

5 CONCLUSIONS

The study demonstrates that an embedded pipeline influences r_u distribution and associated deformations in liquefiable sandy soils during strong shaking. Liquefaction in the loose sand layer caused surface settlements, and the pipeline altered the local pore pressure build-up, emphasizing the need to consider soil-structure interaction effects for underground lifelines in seismic design.

Experimental studies of seismic resistance of slopes at modeling on a vibration test bench

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ABSTRACT

The paper presents the results of physical modeling of slopes behavior on a vibration bench at different levels of seismic impact. The aim of the work is to determine the dependence of the slope stability coefficient on the amplitude of vibrations and to validate the experimental data by numerical calculations. Modeling was carried out on loess soils with different moisture content, and failures were recorded visually and using accelerometer data. It was obtained that at accelerations above 0.3g the stability coefficient decreases below critical values (Fs< 1.0), indicating loss of stability. The results were compared with PLAXIS 2D calculations and showed a satisfactory coincidence. The obtained data can be used to refine the design methods for stable slopes in earthquake-prone regions.
Keywords: slope stability, vibration test, physical modeling, stability coefficient, PLAXIS 2D, loess soils, dynamic impact, geotechnical safety, earthquake-active regions, slip surface failures.

Keywords: slope seismic resistance, vibration test, modeling

1 INTRODUCTION

Seismic stability of slopes is one of the key problems in the design and operation of embankment structures in earthquake-prone areas. In many regions of the world, including South Kazakhstan, Kyrgyzstan, Iran, Japan, and Chile, earthquakes accompanied by landslides, loss of slope stability, and destruction of infrastructure are recorded annually [1-4].

The complexity of earthquake resistance assessment is due to the nonlinear behavior of soils in dynamic conditions and the diversity of engineering and geological situations. Despite the wide application of numerical methods such as PLAXIS, ABAQUS, FLAC3D an important role is played by experimental studies on vibration benches, which make it possible to simulate the behavior of slopes under seismic loads under controlled conditions.

The main objective of this study is to analyze the stability of slopes of different steepness and composition under the influence of harmonic and accelerogram vibrations, using physical modeling on a vibration test bench. The methodology is based on the principles of modeling by similarity (Froude and Coulomb criteria), which allow reliable scaling of real geotechnical processes.

Scientific Novelty

The novelty of this study lies in:

- construction of a physical model of a slope with controlled parameters of density, moisture content and geometry;
- registration of displacements, accelerations and shear deformations using high-precision sensors;
- comparison of modeling results with calculations based on the limit equilibrium method and shear modulus Gmax

2 MATERIALS AND METHODS

To model the behavior of slopes under seismic impacts, a laboratory-type linear vibration bench was used, which allows reproducing vibrations with acceleration amplitude up to 0.5g and frequency up to 3 Hz. The method of physical modeling by vibration is recognized as an effective way to assess slope stability in engineering geology.

The experimental model of the slope is based on the geometry of the slopes investigated during the construction of the M-32 Samara-Shymkent highway. The height of the prototype slope was 3 m, the slope angle was β=350. The model was made in scale 1:10.

3 RESULTS AND DISCUSSIONS

3.1 Tests results

As part of the present study, large-scale experiments

were conducted on a vibrodynamic rig to determine slope stability parameters under various seismic impact scenarios. The results were processed, analyzed and compared with the normative calculations.

Dynamic parameters of shear and deformation. Experimental models of slopes were formed from loess soils with different moisture content (15%, 20%, 25%). A 1:25 scale model was used. Seismic effects were simulated by sinusoidal oscillations with amplitudes from 0.1g to 0.4g.

Comparison with numerical simulations. The experimental data were compared with the calculations performed in the PLAXIS 2D program. The maximum deviations of the results were 8-12%, which confirms the validity of the laboratory modeling.

Discussion of Results. The results confirm the critical importance of considering the amplitude-frequency characteristics of seismic effects in slope design. The experiments show that irreversible deformations occur when the acceleration threshold value (in this case A > 0.25g) is exceeded. Such conclusions are consistent with international studies.

In practice, this indicates the need for additional reinforcement of slopes or geometry changes (steepness reduction) when designing facilities in zones with seismicity of more than 8 points.

4 CONCLUSIONS

The conducted experimental studies of stability of slopes from loess soils on the vibration test have confirmed the significant influence of the amplitude of seismic acceleration on the stability of the structure. It is established:

- When the acceleration amplitude increases from 0.1g to 0.4g, the slope stability coefficient decreases from 1.43 to 0.71, indicating linear degradation of stability;

- Fractures are observed predominantly along circular-cylindrical slip surfaces characteristic of underconsolidated loess;

- The stability threshold corresponding to the coefficient Kust<1.0 is reached at A>0.3g, which is critical for areas with seismic intensity of more than 7 points;

- The results of laboratory tests have a high degree of coincidence with the calculations in PLAXIS 2D (error ≤ 12%), which confirms the reliability of the physical modeling methodology.

Thus, the experimental setup and the proposed methodology allow us to effectively investigate the dynamic behavior of slopes and form recommendations for earthquake-resistant design in real engineering and geological conditions.

Prospects for Future Research

In order to further the understanding of earthquake resistance of soil slopes, it is planned to:

- Expand the range of modeled soil types, including loams and sandy soils of different densities;

- Investigate the effect of different reinforcement parameters (geogrids, anchors) on slope stability under seismic action;

- Conduct a series of multi-cycle tests with modeling of real earthquake accelerograms (El Centro, Kobe, Tohoku);

- Use digital strain capture techniques (DIC - digital image correlation) to analyze displacements along the slope surface in detail;

- Develop a generalized empirical relationship between acceleration amplitude and stability coefficient for practical application in regulatory calculations.

The results of future research will contribute to the creation of improved calculation methods and will provide improved safety of engineering structures in earthquake-prone regions.

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Determination of soil rigidity indices using Marchetti dilatometer in seismic regions of Kazakhstan

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ABSTRACT

The study is relevant in the light of the massive transition from laboratory to field soil testing, this is especially important in the context of the complexity of obtaining natural samples, while maintaining the importance of laboratory testing for large-scale projects and scientific research. The purpose of the study was to describe the Marchetti flat dilatometer, the data obtained and their use, and to present a new Marchetti seismic dilatometer with a module for measuring the velocity of transverse waves. The study used experimental, observation, and measurement methods, including data analysis and interpretation of the results, which provided practically significant conclusions about the properties of the soil and its behaviour under load. One of the main results is that the Marchetti dilatometer provides an opportunity to investigate various types of soils, ranging from dispersed soils to soft rocks. The results of the Dilatometer Marchetti Testing (DMT) measurements remain accurate even in very soft or over-compacted soils. DMT test results can be used for a wide range of engineering and geological applications. They can help in predicting foundation sedimentation, quality control of pavement compaction, and determination of cyclic liquefaction resistance. The Marchetti dilatometer can be an effective tool for monitoring and controlling the soil compaction process. The test results can be used to assess the quality of technical soil reclamation and control the process of increasing compaction and stress. The use of DMT data allows establishing correlations between various soil parameters, such as cyclic liquefaction resistance and compression modulus of deformation.

Keywords: geotechnical research, engineering research, soil properties.

1 INTRODUCTION

Modern trends in geotechnical investigations indicate an increasing importance of field testing methods, which in many cases become the dominant component of research. These methods offer a number of significant advantages, including rapid data acquisition, cost-efficiency, reproducibility, a large volume of obtained information, and reduced variability and expenses compared to traditional sampling and subsequent laboratory analysis. These advantages are particularly relevant when studying sandy soils, where collecting undisturbed samples in their natural state is often challenging. In contemporary construction practice, field tests are widely utilized, providing reliable and accurate data for engineering calculations. At the same time, laboratory methods remain essential, particularly in the context of scientific research and the implementation of large-scale infrastructure projects.

This study aims to investigate the operating principles of the Marchetti flat dilatometer, assess the reliability and in formativeness of the data obtained through its use, and compare the results with those derived from conventional geotechnical investigation methods.

2 MATERIALS AND METHODS

The Marchetti Flat Dilatometer Test (DMT), introduced nearly four decades ago, has become a widely adopted technique for in-situ soil characterization in various industrialized countries, including Kazakhstan. Its methodology has been formally recognized through inclusion in international standards, such as those of the American Society for Testing and Materials (ASTM) and the Eurocodes [6, 7]. The technical capabilities and applications of the device have been comprehensively examined by Technical Committee TC16 of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) [8]. Parallel efforts by the International Organization for Standardization (ISO) and the European Committee for Standardization (CEN) have focused on formalizing standardized procedures for its use.

A distinguishing feature of the DMT is its direct insertion approach, which enables testing without the need for preliminary drilling. This allows for efficient soil profiling and assessment of stiffness parameters that are typically not accessible through other penetration-based methods, which are predominantly focused on strength evaluation.

An advanced version of this technique, known as the Seismic Dilatometer Marchetti Test (SDMT), combines traditional dilatometer measurements with geophysical assessments by incorporating the measurement of shear wave velocity (Vs). This parameter is essential for seismic response analysis, especially in the context of *Eurocode 8: Design of structures for earthquake resistance – Part 1* [9], which mandates the determination of Vs within the top 30 meters of soil in seismic-prone regions. The SDMT enables the determination of the small-strain shear modulus $G_0=\rho Vs^2G$, where ρ is the bulk density of the soil. Additionally, it allows for the evaluation of stiffness degradation as strain increases, which is critical for modeling dynamic soil behavior.

In the present study, a flat dilatometer system was employed, consisting of a stainless-steel blade with a flexible membrane mounted on one face. The blade is driven into the ground and connected to a surface control unit via an electro-pneumatic cable. The control unit includes pressure regulators, gauges, and audiovisual indicators for managing gas injection from a pressurized tank. Upon reaching the target depth, the membrane is inflated, and two pressure readings are recorded: the lift-off pressure required to initiate membrane expansion and the pressure corresponding to a standard membrane deflection. These values are used to compute intermediate parameters, which are further interpreted to derive key soil characteristics, including the constrained modulus of deformation and undrained shear strength.

The methodology also includes the evaluation of pore water pressure. In sandy soils, the closing pressure of the membrane valve is assumed to approximate the equilibrium pore pressure, which provides valuable insight into consolidation behavior and filtration properties.

3 RESULTS

Standard geotechnical equipment, such as penetrometers typically used in static Cone Penetration Testing (CPT) or borehole drilling rigs, was utilized to advance the dilatometer blade into the subsurface. The most efficient approach involves using a 20-ton static pushing rig, which provides high productivity—reaching up to 80 meters per day.

The testing procedure commences with the insertion of the flat dilatometer into the soil to the desired depth. Once in position, compressed gas is injected behind the membrane. After an approximate stabilization period of 30 seconds, two primary pressure readings are recorded: pressure A, corresponding to the onset of membrane expansion, and pressure B, representing the pressure required to displace the center of the membrane 1.1 mm outward into the soil. Optionally, a third measurement—pressure C, indicating the membrane valve closure pressure—may also be recorded. The dilatometer is then advanced to the next test depth, typically in 20 cm increments.

The data obtained from the field are processed by converting raw pressure measurements into intermediate DMT parameters:

- I_D – the material index,
- K_D – the horizontal stress index,
- E_D – the dilatometer modulus.

These intermediate values are subsequently used to derive key geotechnical parameters through empirical correlations, including:

- M_{DMT} – the constrained modulus of deformation,
- cu – the undrained shear strength,
- K_0 – the coefficient of earth pressure at rest, particularly for cohesive soils [8].

The DMT system itself consists of a steel blade fitted with a thin, flexible membrane affixed to one face. In its unloaded state, the membrane lies flush with the flat surface of the blade (Figure 1). The blade is connected to a surface control unit through an electro-pneumatic cable routed inside the pushing rods, which enables precise regulation of gas pressure and data collection during testing.

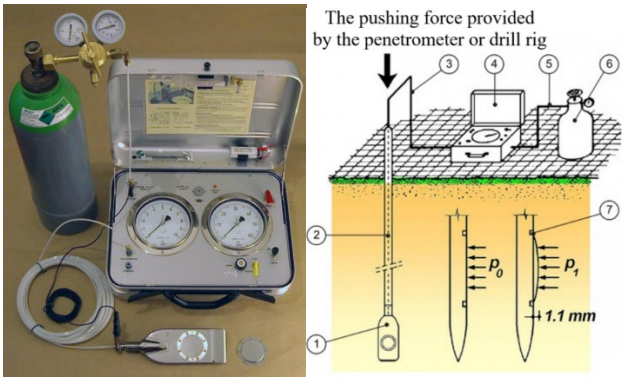


Figure 1. Marchetti Flat Dilatometer (DMT) Components and Test Setup.

Source: compiled by the authors.

The density of the soil, expressed by the bulk mass ρ, is related to the over-compaction coefficient OCR for clays and the effective angle of internal friction φ' for sands. The measurement of pore pressure allows estimating the coefficients of consolidation and filtration [8]. The value of the closing pressure of the valve C in the sand approximately corresponds to the equilibrium pore pressure. Examples of changing parameters with depth obtained using DMT or SDMT (with an additional V_s profile) are shown in Figure 2.

Pile integrity testing for light rail project in Kazakhstan using cross-hole sonic logging

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EXTENDED ABSTRACT

Title: Pile Integrity Testing for Light Rail Project in Kazakhstan Using Cross-Hole Sonic Logging

Objective:

This study aims to evaluate the integrity of concrete piles used in the foundation of a light rail project in Astana, Kazakhstan. Piles with diameters of 1000 mm, designed as C35 underwater cast-in-place piles with a length of 40 m, were tested to identify potential defects such as voids, inclusions, or discontinuities. The research focuses on applying Cross-Hole Sonic Logging (CSL) to assess pile quality under complex soil conditions, particularly in sandy and gravelly strata where polymer mud was used during construction to prevent borehole collapse.

Methodology:

Multi-channel ultrasonic testing (ZBL-U5700 system) was employed to measure acoustic parameters, including first arrival time (FAT) and signal amplitude (energy), across three pairs of embedded access tubes per pile. Testing was conducted after the concrete reached ≥70% of its design strength (per TB 10218-2008) or 15 MPa (ASTM D6760). The probe was raised incrementally at 100 mm intervals to record data along the entire pile length. Defect zones were identified by a ≥20% increase in FAT and reduced energy compared to adjacent sound concrete, following criteria from railway engineering standards (TB 10218-2019).

Key Results:

Pile B/5 and C/5-1 showed homogeneous concrete with consistent FAT (3900–4200 m/s) and high signal energy across all tube pairs (0.710–0.770 m spacing), classified as Class I piles (no anomalies).

Localized velocity drops below 3000 m/s were detected at 12–14 m depth in some sections, indicating potential defects like soil inclusions or honeycombing.

Waterfall diagrams and tomographic analysis visualized continuous wave propagation in sound areas, while disrupted patterns highlighted defect zones.

Conclusions and Significance:

CSL proved effective for detecting volumetric defects and assessing 3D concrete quality, with high sensitivity to large-scale anomalies like voids.

The method's reliability depends on proper tube alignment, water coupling, and concrete curing. Three-tube configurations (recommended for 1000 mm diameter piles) provided sufficient coverage for defect localization.

Limitations include inability to detect diameter variations or micro-defects outside tube paths, necessitating supplementary tests (e.g., low-strain integrity testing) for comprehensive evaluation.

Recommendations:

For piles ≥1500 mm in diameter, install ≥4 access tubes to improve defect detection in diagonal zones.

Integrate AI-based signal analysis or multi-frequency testing to enhance sensitivity to subtle defects.

Conduct follow-up static load tests on identified Class III/IV piles to validate structural performance.

Keywords: instruction, proceedings, geotechnical engineering, symposium

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Geotechnical support for the design and construction of a temple complex on a landslide slope

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ABSTRACT

The article presents the experience of comprehensive assessment of slope stability and organization of geotechnical monitoring during construction of a temple complex in the suburbs of Moscow. The object is located on the top of a slope with a complex geological structure and a significant thickness of fill soils. Additional engineering and geological surveys, calculation of slope stability, and optimization of design solutions for retaining walls were carried out. A monitoring system was developed and implemented, including inclinometric observations and geodetic control, the results of which were compared with predicted value. The obtained data confirm the effectiveness of the adopted engineering solutions in complex geotechnical conditions. A conclusion is made about the importance of an integrated approach to design and geotechnical monitoring during construction on slopes.

Keywords: slope stability, scientific and technical support, geotechnical monitoring

1 INTRODUCTION

An important aspect in the design and construction of buildings on complex terrain (slopes) is the solution of issues related to the assessment of the slope stability, as well as the control of horizontal displacements of the soil base in the area of the construction site.

This paper presents the experience of scientific and technical support of the design and construction of a temple complex in the suburbs of Moscow, which was built on top of a slope in conditions of possible landslide processes.

2 SLOPE STABILITY ASSESSMENT AND DESIGN REVIEW OF THE RETAINING WALLS

As part of the geotechnical expertise of design solutions, the geotechnical engineering surveys were analyzed and additional studies were recommended to assess the stability of the slope.

The authors of the article implemented an approach to calculating the slope stability under the conditions of existing uncertainty of physical and mechanical characteristics in plan and depth of a large layer of fill soils lying in the upper part of the slope. The strength characteristics of the fill soils of the slope were selected in such a way that the calculated sliding surface passed through their thickness, while the stability factor was within the range of $1.0 \leq k_{st} \leq 1.1$, since in natural conditions no landslide phenomena were observed in the fill soils. After this, "conditional fixation" of the fill soils was carried out in order to determine the sliding surface and the minimum stability factor in the underlying native soils with known mechanical characteristics.

Based on the results of the work, the previously adopted design solutions were optimized in terms of retaining walls of the projected facility.

3 GEOTECHNICAL MONITORING

The authors of the article developed a geotechnical monitoring program. It included observations of the surrounding development, the soil base in the area of the temple complex construction, the temple building, including retaining walls, and changes in the hydrogeological regime.

3.1 Monitoring System

In accordance with the geotechnical monitoring program, a system of observations of the slope soils was

implemented at the construction site of the temple complex, including measurements of planned displacements of surface soil plates, as well as horizontal displacements along the depth of the massif (inclinometric observations).

In the northwestern part of the slope, in order to control and confirm the predicted displacements in the layer of filled ground, two inclinometer pipes 6 and 9 m long were installed, with the depth of their immersion determined by the calculated sliding surfaces.

Surface plates were installed at a depth of up to 2.5 m over the area of the slope to observe the surface horizontal displacements of the soil mass.

3.2 Monitoring Data Analysis

During the installation of bored piles of retaining walls on the slope edge in the northwestern part of the construction site, a displacement of the top of the reinforcement cages of several piles by up to 28 cm towards the slope was recorded under the conditions of concrete strength gain under the load from construction equipment. At the same time, increases in horizontal displacements in the first inclinometer pipe installed near the retaining wall were recorded, reaching 12.0 mm towards the slope, as well as planned displacements of surface soil marks by up to 45.0 mm.

On the recommendation of the authors of the article, additional engineering and geological surveys were conducted, the results of which revealed a significant thickness of fill soils (up to 8 m) beyond the contour of the retaining walls. Control calculations taking into account these data confirmed the stability of the slope.

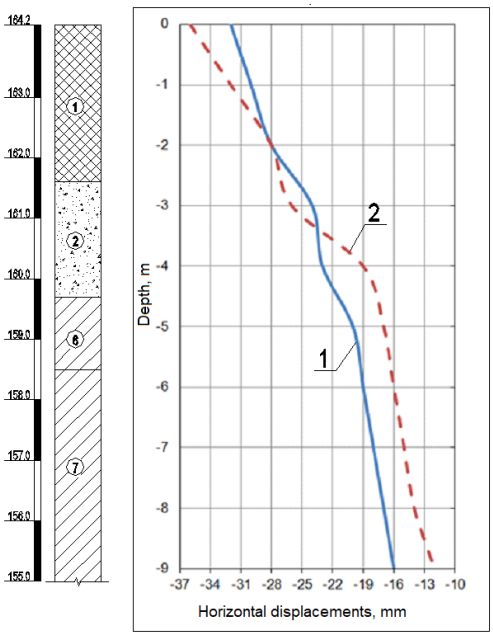


Fig. 1. Horizontal displacements along the depth of the soil massif: 1 - measured displacements of the first inclinometer pipe; 2 - predicted displacements.

At the time of completion of construction work at the site, including backfilling the area to design marks, the maximum horizontal movement along inclinometer wells was recorded at a depth of 1 m and amounted to 30 mm, which does not exceed the predicted value 32 mm (Fig.1).

The maximum planned displacement according to the results of inclinometric measurements reached 61 mm.

The monitoring results, taking into account corrections for the planned displacements of the inclinometer heads recorded by the tacheometer, are consistent with the predicted values.

4 CONCLUSIONS

From the presented case studies, it can be concluded that:

1. It is necessary to study the mechanical characteristics of backfill soils using field and laboratory methods during engineering surveys, especially if they have a significant thickness, in order to obtain more reliable initial data that are used in slope stability calculations.

2. The geotechnical monitoring program should be developed taking into account the requirements of current regulations and predicted geotechnical risks to prevent emergency situations.

3. Inclinometric measurements must be carried out with measurements of horizontal displacements of the head of the inclinometric pipes using a tacheometer for the purpose of possible correction of the obtained results, especially in the case of significant displacements, since the pipe may end up completely in the deformation zone of the soil base.

4. The use of an inclinometric system at construction sites allows for the control of horizontal displacements of soil in depth with high accuracy and the required volume, as well as the verification of calculated data.

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Advanced fiber optic monitoring of cracks in reinforced concrete for structural health applications

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ABSTRACT

Advanced fiber optic monitoring technologies are becoming essential tools in the field of structural health monitoring (SHM), offering accurate, durable, and real-time detection of damage in reinforced concrete structures. This study presents an experimental evaluation of a novel distributed fiber optic sensing (DFOS) approach that utilizes inner-fixed-point cables for enhanced crack detection performance. The proposed system is designed to provide high-resolution strain measurements while maintaining cable integrity under localized deformation conditions.

A laboratory test was conducted using a reinforced concrete pile embedded with the inner-fixed-point fiber optic cable. A gradually increasing load was applied to induce crack formation, while strain data were continuously acquired using a Brillouin Optical Time Domain Analyzer (BOTDA). The cable design, which incorporates predefined fixed points separated by free sensing lengths, enables the system to localize strain concentrations with high precision. This segmented structure allows for the detection of cracks without compromising the sensing fiber, distinguishing it from conventional continuous fiber optic systems.

The experimental results demonstrated the effectiveness of the advanced DFOS system in capturing the initiation and propagation of cracks in real time. The system's performance aligns with the requirements of modern SHM applications, especially in critical infrastructure monitoring, where early damage detection and long-term durability are paramount. Furthermore, the collected data can be integrated into Building Information Modeling (BIM) and digital twin platforms, supporting intelligent infrastructure management through real-time visualization, diagnostics, and decision-making.

This research highlights the potential of advanced fiber optic technologies in delivering precise, non-invasive, and scalable solutions for monitoring the structural integrity of concrete elements in contemporary engineering practice.

Keywords: distributed fibre optic sensing, fixed-point architecture, reinforced concrete, structural health monitoring, crack detection

1 INTRODUCTION

In recent years, there has been growing concern regarding the significant degradation of civil infrastructure, primarily caused by aging and operation beyond the original design capacities. To detect structural damage, various non-destructive evaluation (NDE) techniques have been developed, including ultrasonic testing, radiography, acoustic emission, and eddy current methods. However, these approaches often suffer from notable limitations, such as limited portability, vulnerability to electromagnetic interference, and the inability to provide continuous structural health monitoring. Structural Health Monitoring (SHM) is increasingly acknowledged in civil engineering as an effective approach to enhance structural safety and reduce operational and maintenance expenses. The data obtained from SHM programs enables informed decision-making regarding the operation, maintenance, repair, and potential replacement of structural components, relying on accurate and objective information. Continuous monitoring allows for the early detection of damage and deviations from expected performance, facilitating proactive interventions. Furthermore, SHM data can be integrated into structural management systems, improving the overall quality and reliability of maintenance and lifecycle management strategies. During the operation of high-rise buildings, structural malfunctions can lead to serious consequences. Undoubtedly, the most severe of these is an accident involving human casualties. Ensuring operational safety is the primary goal for both builders and developers. The

economic impact of structural deficiencies manifests in two forms: direct and indirect. Direct losses are associated with reconstruction costs, while indirect losses involve disruptions and financial setbacks in related sectors of the economy. Analyzing the behavior of residential high-rise buildings under real operating conditions contributes to the improvement of future design solutions. This, in turn, can lead to the development of more cost-effective, safer, and more durable structures with enhanced reliability and performance. Structural health monitoring serves as an effective tool for expanding our understanding of how buildings perform in real-world conditions [1].

2 MATERIALS AND METHODS

Many fully distributed measurement systems use time-domain scanning to determine the measurement point based on the distance that scattered light travels in a given time. A well-known method, Optical Time Domain Reflectometry (OTDR), is commonly used to measure the backscattered signal of a light pulse, which is modulated by the properties of the medium along the optical fiber. In such systems, a short optical pulse is launched into the fiber, and a small portion of the pulse is scattered back and detected. The spatial resolution is determined by the pulse duration. The measured parameters can be derived from the intensity or wavelength of the scattered light [2].

Rayleigh backscattering is widely used in OTDR- and OFDR-based systems for fault location, loss measurement, and distributed parameter sensing. Rayleigh scattering is an elastic scattering process in which the scattered light retains the same wavelength as the incident light. In Rayleigh-based backscattering systems, the intensity of the scattered signal is usually measured, which depends on environmental parameters such as temperature and strain.

The proposed layout for the fiber optic cable on the base slab is illustrated in drawings with the layouts reproduced in Figure 1 for the reader's convenience.

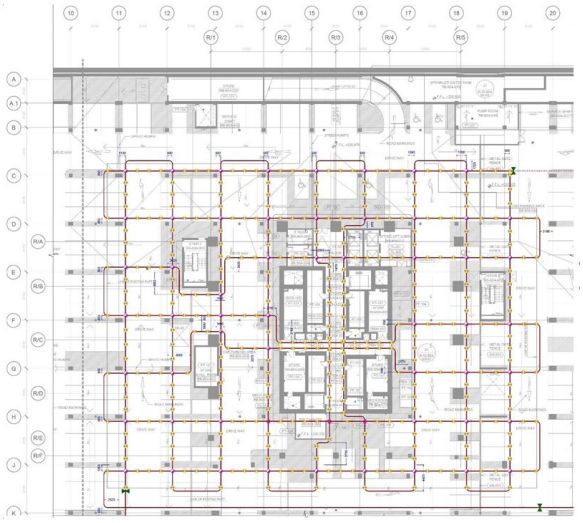


Fig. 1. Part plan of cable routing in Block R monitoring area

In the drawings the bending radii of the cable at turning points is drawn as 600 mm assuming use of the cable specified in Appendix B. If an alternative cable is preferred the bending radius must be at least one-and-a-half times the allowable bending radius specified by the cable manufacturer to allow for installation error.

3 RESULTS

Particular attention was given to the high-rise structures during the analysis of fiber optic monitoring data, with a focus on Block R, where the highest strain values in the fiber optic cable were recorded, as anticipated. The temperature sensitivity coefficient (CT) was determined to be 1.113 MHz/°C, and the strain sensitivity coefficient (Cε) was calculated as 0.054 MHz/με. During the monitoring period, the temperature distribution at the lowest level (B4) of Block R remained relatively stable, ranging from +4°C to +8°C. Areas with slightly lower temperatures were observed in the building's central core along the load-bearing walls, likely due to the higher thermal conductivity of reinforced concrete elements.

Given that the raft foundation is located at a depth of −19 meters and Block R occupies a central position within the overall building complex, the influence of ambient temperature on fiber optic strain measurements was considered negligible.

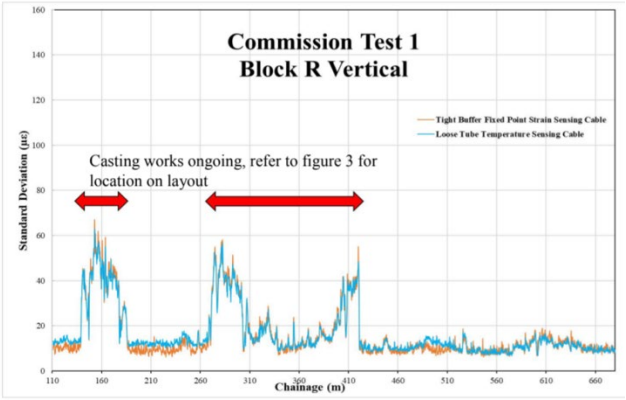


Fig. 2. Commission test 1 standard deviation block R vertical

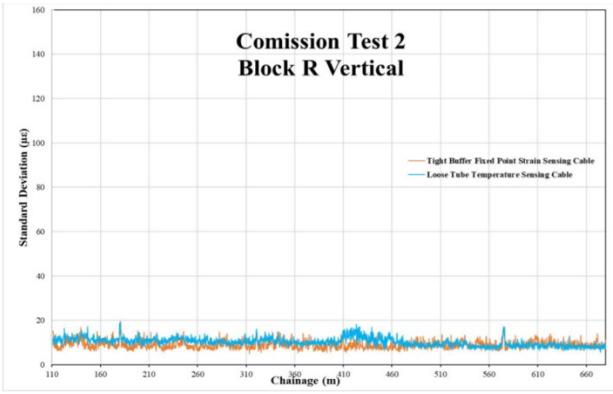


Fig. 3. Commission test 2 standard deviation block R vertical

4 CONCLUSIONS

The results of the tests captured temperature variations caused by concrete curing during the measurement period, as illustrated in Figures 2 and Figure 3. Notable temperature fluctuations were observed in specific zones of Block R, where slab casting was ongoing during the tests. Similar trends recorded by both the temperature and strain sensing cables indicate that the observed strain variations were primarily due to temperature changes rather than structural movements A comparison of the standard deviation between the first and second test showed nearly identical values, except in areas affected by significant temperature fluctuations. This suggests that, for the analyzer used, thermal drift of the electronic components does not impact measurement accuracy when the device is powered off between data acquisitions.

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