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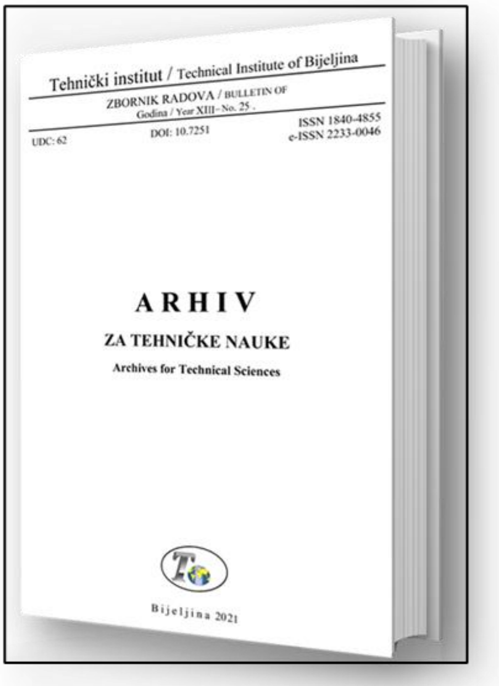

CONSTRUCTION

ENVIRONMENT

ARCHITECTURE

Architectural design

Editors

Prof. Ph.D. Viktorija Aladžić
Prof. Ph.D. Milan Kekanović

Preliminary report
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ARCHITECTURAL DESIGNING THE NURSING HOMES IN TEHRAN CITY (AS A SAMPLE)

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ABSTRACT

In recent years, there has been an increasing trend of entrusting the elderly to nursing homes, thus providing a suitable space that meets their physical, psychological, social, and spiritual needs has become necessary. The nursing homes is the first and simplest solution that can be considered for such individuals. However, considering the characteristics and temperament of this group, it is necessary to define a model that can meet the elderly's spiritual and physical needs, alongside maintaining their normal life, in the home of their hopes and memories. Therefore, there is a need for a model that defines a new space in accordance with the psychological and physical conditions of the elderly and can replace previous methods in order to improve the mood of the elderly.

The design of elderly homes, and certainly the creation of suitable decoration for them, should match the global standards. The concept of an elderly village is another model that has been introduced and implemented for the elderly in the West. In this model, housing units have been designed for the elderly to live alone or with their families. Basic and necessary services for the elderly are available in proximity and neighboring the housing units. These places will have a positive impact on creating a sense of usefulness, increasing hope in life, and uplifting the mood of the elderly.

The elderly have the autonomy to use these facilities and they fulfill their needs, such as suitable spaces for exercise and recreation, socializing with peers, preserving their dignity, and creating conditions for their independence during leisure time. They also provide counseling, teach necessary techniques for the elderly’s lives and more.

Keywords: *The elderly, nursing home, day care center, life expectancy, improving quality of life*

INTRODUCTION

The United Nations in 2006 estimated the global elderly population to be 687 million and 923 thousand individuals, and projected that this number would reach 1 billion and 968 million and 153 thousand individuals by 2050 [1,2]. This would make up approximately 21.4% of the population at that time. Iran has not also been exempt from the phenomenon of an aging population, as the average lifespan of Iranians increased by about 10 years between 1986 and 1996. According to the 2011 census, Iran has 6.2 million elderly people aged 60 and above, equivalent to 8.2% of the country's total population [3]. Based on population projections until 2049, the number of Iranians aged 60 and above will increase to 28 million, making up approximately 28% of the population. Old age is a period in which an individual gains experience, maturity, and wisdom after spending several years. They can take measured steps to navigate through future years and share their experiences with others.

In Iran and most Asian countries, the majority of elderly individuals live with their families, and those who are separated from their families for various reasons receive financial and emotional support from younger family members [4]. However, changes in the family structure, influenced by the development of modern lifestyles, increasing migration, and the movement of families, have led to the abandonment of the elderly and their social isolation [5]. Sometimes, this results in the elderly seeking refuge in elderly homes. Considering that many elderly individuals are capable of performing many tasks, there is a need for temporary residential centers that cater to their daily needs, providing recreational, welfare, and sports facilities. The elderly prefer to be independent and usually prefer to live in close proximity to their children. On the other hand, elderly individuals who live in warm and emotionally connected family environments have higher well-being compared to those who live alone. The elderly require spaces that suit their physical and emotional needs. If such spaces are not provided and they will not receive appropriate responses to their needs from the environment and they may also experience depression [6,7].

In recent years, there has been an increasing trend of older adults being placed in nursing homes and the number of such facilities is rising [8]. In addition, the physical and social conditions of the elderly’s living environment affect their well-being. Studies have shown that there is a higher disruption in some aspects of life for residents of nursing homes compared to elderly individuals living at home [9,10]. Additionally, the belief in the necessity of adapting the living environment of the elderly in these facilities is undeniable. Therefore, conducting research to evaluate the prevailing design conditions in elderly care homes in Tehran is highly necessary. According to the aforementioned information, this research was conducted in 2014-2015 with the aim of evaluating the conditions of elderly care facilities in Tehran in accordance with globally defined standards for the suitability of the physical environment of these facilities.

RESEARCH METHODOLOGY

This research was conducted based on theoretical studies and fieldwork. In the theoretical studies section, existing literature, internet sources, and interviews with experts were utilized to review and understand the regulations and standards. In the fieldwork section, site analysis, analysis of existing samples, and examination of current regulations were carried out.

Theoretical Section

In this section, the existing regulations and standards for designing elderly care homes were initially addressed. Then, based on these regulations, the design of the required spaces for the target site and its analysis were performed.

Fieldwork Section

In this part, the design of an elderly care home in District 10 of Tehran was addressed.

Geographical and Climatic characteristics of the Target City:

The city of Tehran is located between 51 degrees 2 minutes to 51 degrees 36 minutes east longitude and 35 degrees 34 minutes to 35 degrees 50 minutes north latitude. Its elevation varies from approximately 1800 meters in the north to 1200 meters in the central area and 1050 meters in the south, measured from the level of open waters. The climate of Tehran is moderately temperate and humid to some extent. The remaining parts of the city have a warm and dry climate, with slightly cold winters. The amount of precipitation in Tehran is generally low, measuring approximately 245.8 millimeters per year, and the number of frost days (with temperatures below freezing) is recorded as 36 days per year.

Characteristics of the Desired Site:

The selected site is located in District 10. District 10 is situated in the western part of Tehran and shares borders with Districts 17, 11, 9, and 2. On the right side of this site, there is Jeyhoon Street, which runs north to south. At the top of the site, there is Hashemi Street, intersecting with Jeyhoon

Street. The hierarchy of the access network around the site is indicated on the map, including main streets and side streets. The land use of this site includes medical, educational, and commercial purposes (Figure 1). Environmental pollution in this area includes noise and air pollution.

Figure 1. Surrounding Land Uses

FINDINGS AND DISCUSSION

Existing Regulations and Standards for Designing Nursing Homes:

Suitable Location for Nursing Homes

Experts believe that providing mental health care and creating a peaceful environment free from stressors can encourage the elderly to continue living and create a vibrant and self-sufficient life for them. It is also necessary to provide social interactions for the elderly and ensure a sense of security and protection in their external spaces. Therefore, recommendations have been made for the design of such places. For example, nursing homes should not be located near busy main streets, intersections, squares, waste disposal sites, places where animals are kept, slaughterhouses, factories, and the like. Additionally, the construction site should be away from pollutants and abnormal noises.

Nursing homes should be in direct proximity to chain stores, banks, mosques, libraries, healthcare centers such as clinics, hospitals, pharmacies, post offices, cinemas, parks, and the like, within a short distance that is within the maximum walking radius for the elderly. In other words, the accessible paths should be shorter than usual, with a maximum distance of 1000 meters. It is preferable for nursing homes to be within a 500-meter distance from a bus station, with a safe, flat, well-lit route, and the bus station itself should have shelter and seating areas.

The surrounding environment of nursing homes should be reasonably flat, with safe and secluded communication routes, surrounded by suitable trees and green spaces. The size of the building should be small, and the number of residents should be limited to provide comfort and allow better care. In terms of ensuring maximum safety, it is necessary for the nursing home building to be fire resistant.

Main Entrance

The main entrance of any building is of great importance and requires careful design consideration. It should be treated as a distinct space because part of it belongs to the interior and part to the exterior of the building. The main entrance of a nursing home is significant both in terms of accessibility for the elderly and visitors and in presenting an overall image of the facility. Therefore, creating security and easy access are fundamental design considerations that must be taken into account. The main entrance should provide direct access to the main elevator, staircase, as well as the common areas such as the living room and dining room [11,12,13].

Living Room

In a nursing home, the living room should be designed in a way that allows the elderly to gather and rest after daily activities. The architectural design of this space should be based on the specific needs of the residents and should be able to adapt to the mood of all the elderly individuals residing there. It should also provide easy communication and access to the bedrooms. It has been observed that large living rooms that give an office-like feel are less utilized by the elderly. Such rooms are only suitable for special purposes and limited times, such as meetings, discussions, concerts, and parties, while various activities should take place in the communal areas of nursing homes, such as watching television, playing intellectual games, reading, and sewing.

Therefore, it is recommended to use multiple smaller spaces clustered around a more public area to make it more desirable for the elderly. Each of these spaces should have a familial atmosphere and be capable of accommodating groups of more than 8 people engaging in specific activities comfortably. Hence, the minimum dimensions for such spaces are around 4.5×5.3 meters, and a standard area of 5.1 square meters per person is recommended for communal rooms. If the living room is separate from the main kitchen, a small kitchenette can be considered for the residents to make tea or coffee. Additionally, if the living room and dining room are adjacent, movable and foldable partitions can be used between the two spaces. Since living rooms in nursing homes should create a cheerful and exciting atmosphere for the elderly, bright and warm colors should be used, while libraries and study rooms should be painted with soothing and gentle colors [14].

The windows in the living room should be short so that individuals sitting on comfortable chairs can have a view of the outside. Therefore, the height of the window sill from the ground should be between 30-90 centimeters, and the height of the window frame should be 1.8 meters for easy opening [15]. To create uniform and suitable lighting conditions for the special visual needs of the elderly, parallel lighting fixtures should be used. It is also recommended to provide lighting equivalent to 200 watts in the living room. The space, which is continuously used by groups of two or three people for relaxation and conversation, should be suitable for seating and watching television. Quiet spaces suited for intellectual games and reading should be included, as well as spaces for group activities, sewing groups, and art classes. A small room should be designed as a prayer room.

It is recommended to separate the smoking area from the non-smoking area. The furniture in the living rooms and common areas should be lightweight and easily movable, and fabrics should be used that can hide stains and dirt and can be easily cleaned, providing comfort and tranquility to the elderly when sitting on them. Additionally, the total surface area of furniture and objects in the living room should be such that the sound reflection time is relatively short to prevent noise disturbance. It is recommended to establish visual connection to the green and open outdoor space, which is desirable for the elderly, through the creation of covered terraces and balconies in front of the living rooms.

Library

If creating a separate space for reading is economically challenging, a dedicated space for this purpose can be considered alongside the living room. This space is considered quieter than other areas of the nursing home and should be equipped with comfortable chairs, temporary tables, and shelves for storing magazines and newspapers [16].

Dining Rooms

Elderly individuals who live alone do not fully understand the social importance of shared meals. Therefore, they may experience less favorable emotional and social conditions compared to those living in communal elderly care centers or those who have not yet separated themselves socially. It is better for these spaces to have connections to terraces or outdoor green spaces so that the elderly can enjoy open air while having their meals in the summer. The flooring of the dining rooms should be selected in a way that prevents slipping and reflects light, and it should be easy to clean. It should also have sound-absorbing properties. Additionally, the lighting should be more intense and individualized on the dining table surfaces. It is recommended to have windows with a height of 70 centimeters from the floor so that individuals can enjoy the outdoor view while eating. Dining tables should be designed in various sizes for 2, 4, or 6 people, in square or rectangular shapes with rounded corners. The height of the dining table depends on the height of the seating area, but it is suggested to be 70 centimeters to provide enough space for leg movement and accommodate wheelchairs. The surface area of the dining table should preferably be 90×90 centimeters (minimum absolute size is 75×75 centimeters), as smaller sizes are usually insufficient. Lightweight, easily cleanable chairs with non-slippery bases are recommended. According to the design recommendations, a standard space for dining for 4 to 6 people, considering the needs of the elderly with disabilities and wheelchair users, would be around 2.2 square meters (Figure 2) [17].

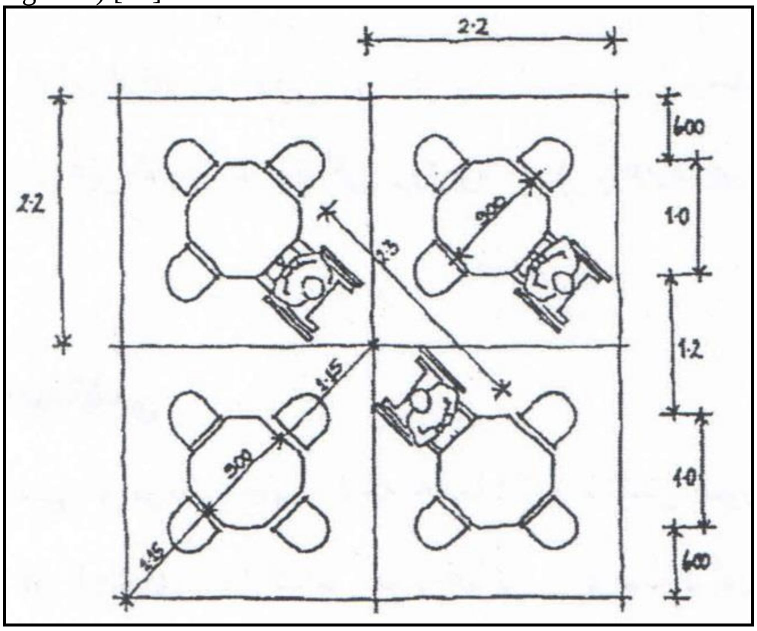

Figure 2. Dining Area for Suitable 4 to 6 People

Staff Room

The employees of the nursing home should have suitable facilities for sleeping and resting. Additionally, a separate room for rest and dining should be provided for them so that they can relax during non-working hours. Therefore, for each employee, a room with an area of 8 square meters with bathroom and toilet facilities can be considered [17].

Management Room

For administrative purposes, a space of approximately 9 square meters should be designated. The room should be located near the entrance and include an area for storing residents' files, comfortable seating, and a reception desk for receiving and making payments. The facility manager should also have welfare facilities such as sleeping area, sitting area, kitchen, toilet, and bathroom to attend to their daily needs even during non-office hours [18].

Therapy and Rehabilitation Rooms

In nursing homes, rehabilitation and therapy measures are necessary to provide services to residents. The presence of a physician, physiotherapist, social worker, nutritionist, and nurses is essential for the elderly throughout the whole time. Therefore, the existence of suitable spaces for providing these services on the ground floor of nursing homes is necessary. The purpose of the rehabilitation program is to assist the individual in need of rehabilitation to maximize their abilities, adapt to society, and prepare them physically, mentally, and socially for a type of life that is within their capacity. In the rehabilitation section, efforts should be made to strengthen the elderly's muscle forces and provide guidance on the correct use of walking sticks, crutches, or wheelchairs if they use them. If physiotherapy is performed along with the assistance of social workers or psychiatrists and continues throughout the treatment, the elderly will achieve results more quickly and easily and regain their abilities [19,20]. The examination and treatment room should be equipped to facilitate the following activities: 1) Examination of the elderly by a physician, even if it is performed once; 2) Injection and medication. It is suggested that the examination and treatment room have double doors with a total width of 120 centimeters, and the recommended area for the room is approximately 18 square meters.

Storage Room
For storing cleaning items such as blankets, sheets, towels, etc., a storage area of about 1 square meter per resident is necessary. This space can often be located next to the bathroom, and it is recommended to have tiled flooring for easier cleaning. For storing equipment and items such as wheelchairs, walkers, etc., a room with an approximate area of 4-10 square meters is recommended, and it is preferable to have such rooms on each floor of the nursing home [12].

Sanitary Facilities and Toilets
Since the elderly need access to toilets during the night, it is necessary to have toilet facilities adjacent to their bedrooms. The minimum suitable dimensions for toilets used by elderly individuals in wheelchairs should be 170×170 centimeters. The height of the toilet bowl (referring to Western-style toilets as the use of squat toilets is not suitable for the elderly, even if they are capable of using them) should not exceed 38 centimeters from the floor, and grab bars should be installed on the walls on each side, with a distance of 70 centimeters from the floor, to assist with sitting down and standing up [12].

The toilet used in nursing homes should be without a pedestal and installed at a height of 85 centimeters, but for individuals using wheelchairs, this height can be reduced to 75 centimeters. A minimum width of 80 centimeters is recommended for sanitary facilities. The floor, walls, toilet, and bathroom should be covered with suitable tiles. The water faucets should be lever-style and easy to open and close, so that elderly individuals suffering from arthritis and joint swelling can use them comfortably. The minimum dimensions for the toilet bowl should be 65×40 centimeters, and the rim should be at the height of the forearm and at least 85 centimeters above the floor. In countries with cold climates, it is preferable not to have the toilet against an external wall (empty space behind it) [18].

Communication Spaces in Nursing Homes
Considering the physical conditions of the elderly, it is preferable for nursing homes to have a single floor, and if necessary for buildings with more than two floors, the use of an elevator is mandatory. The elevator should be placed near the main entrance of the building and accessible on each floor without the need to go up or down stairs. Since the elderly move very slowly, the automatic doors of the elevators should be adjusted to allow sufficient time for individuals to enter and exit the elevator. Additionally, the elevator stops should be relatively longer than usual.

The minimum dimensions of the elevator cabin should be 1.1×1.4 meters, and it should have an opening width of 80-90 centimeters. Inside the elevator cabin, a railing should be installed 90 centimeters above the floor to maintain the balance of the elderly. It is necessary to use a fixed folding seat inside the elevator. The elevator cabin should be equipped with an emergency bell, a telephone line for external communication, control buttons, and other auxiliary equipment installed on the cabin wall for use inside the elevator, which should be placed between 90 and 130 centimeters.

Hallways
In nursing homes, hallways should have sufficient width and appropriate lighting to allow individuals in wheelchairs, individuals with walking canes, and pedestrians with companions to move in both directions. The width should be 70 centimeters for individuals using a cane and between 100 to 120 centimeters for individuals using a wheelchair. If the width of the hallway is considered as 180 centimeters, it will allow two wheelchairs to move side by side. Therefore, it is recommended that the width of hallways in nursing homes be a minimum of 150 centimeters, and it is always advisable to avoid narrow and winding corridors that may confuse the elderly.

Garden Area
One of the most valuable activities that can be considered for the elderly is gardening, even if only a few of them are interested in this activity. Gardening can be seen as a tool to showcase the abilities and achievements of the elderly. Two types of gardens exist in most elderly homes: 1) Private patios and balconies, and 2) Yards and designated areas. Elderly individuals should be given the opportunity to engage in gardening and flower cultivation in their own environment and bring their favorite flowers to the garden assigned to them.

Bathroom
For the elderly and individuals with disabilities, the bathroom becomes more important and should be designed for maximum comfort. The bathrooms should be fully accessible for wheelchair users, either without handrails or with one or even two handrails for assistance. It is advisable to have a small number of bathtubs in nursing homes. The essential and necessary fixtures in the bathroom include a bathtub or shower, toilet, bidet, first aid cabinet, and mirror. These fixtures should be arranged in a way that individuals with full mobility, users without assistance, and users with assistance from individuals in wheelchairs can easily access them.

However, regardless, individuals with severe disabilities who are unable to bathe even with assistance, as well as those with skin problems who find relief by immersing in water, require a bathtub for bathing. Therefore, a small number of bathtubs should be designed in nursing homes.

The essential and necessary fixtures in the bathroom include a bathtub or shower, toilet, bidet, first aid cabinet, and mirror. These fixtures should be arranged in a way that individuals with full mobility, users without assistance, and users with assistance from individuals in wheelchairs can easily access them, Figure 3.

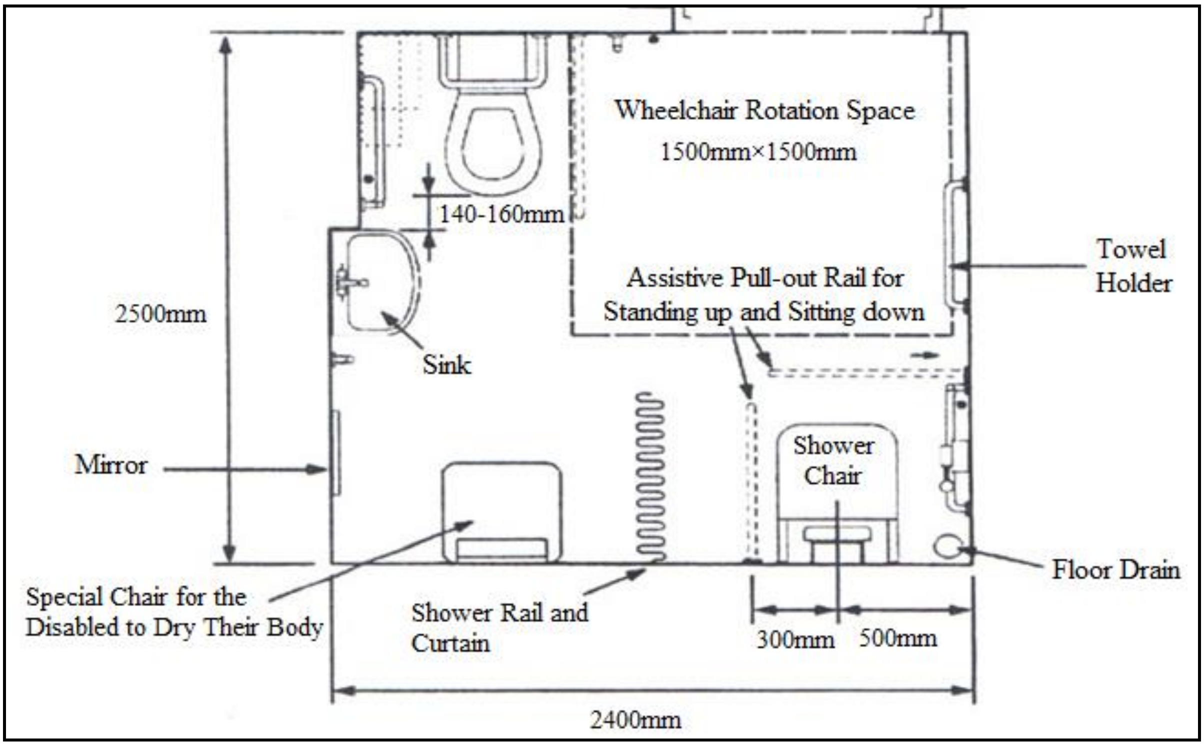

Figure 3. Appropriate Dimensions for Accessible Restroom and Shower for Disabled Individuals

Doors and Windows
Usually, doors create difficulties for the elderly, as most of them have joint swellings and may have trouble bending their fingers. Therefore, the door handles should not be round-shaped; instead, "D"-shaped handles are recommended. It is advisable for the handle to be approximately 90 centimeters above the floor of the hallway to allow for easy movement in both sitting and standing positions. Since most elderly individuals lower their feet while walking, there should be no protrusions on the threshold of the door that could cause them to fall.

In the bedroom, the height of the window should be low enough for a person lying on the bed to easily see outside. This height should be considered between 30-60 centimeters from the floor. For dining rooms, a height of 70 centimeters is recommended for windows so that elderly individuals in a seated position can comfortably enjoy the view outside. The window sills should be at a suitable height to accommodate small decorations and flower pots. Window handles should be designed to operate with minimal force and be easily accessible at a lower level, preferably between 90 to 120 centimeters from the floor.

Staircases

The design of staircases should ensure safety and have appropriate dimensions, sufficient lighting, and suitable handrails. Straight flights of stairs may appear more economical, but they are usually intimidating for the elderly and individuals with reduced mobility due to the long flight of stairs. It is better to divide them into two half-flights, although this increases the number of steps, it is more suitable for the safety and better balance of the elderly.

The maximum recommended number of steps for each flight is nine steps, and for a greater number of steps, the creation of landings for resting is necessary. For the elderly, an appropriate step surface should have sufficient width for them to stand on with some safety and catch their breath (Figure 4).

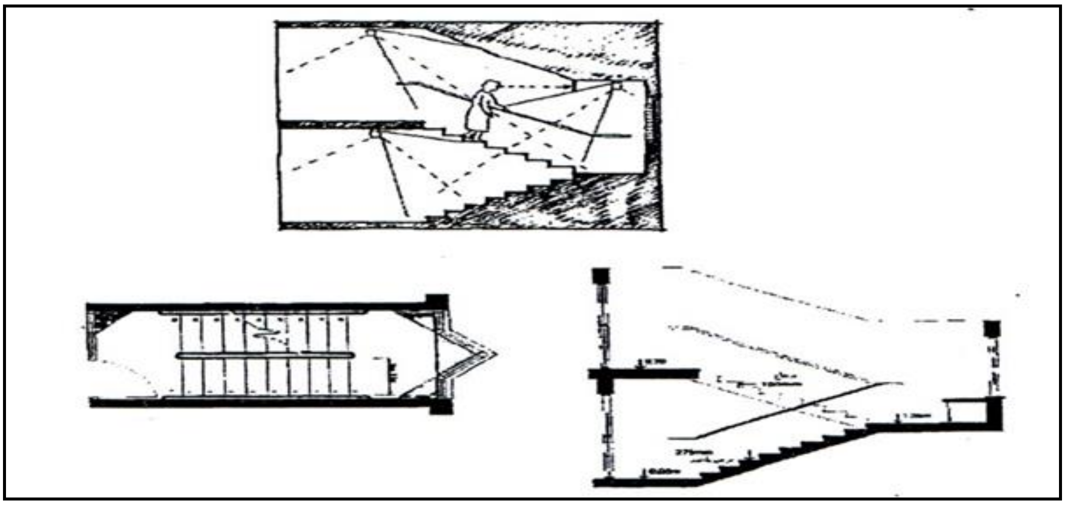

Figure 4. Appropriate Dimensions for Staircase

Suitable Flooring

The hallways and rooms of elderly homes should have non-slippery, sturdy, and consistently smooth flooring to prevent wheelchair obstruction or loss of balance. Inappropriate flooring selection, especially in hallways, hinders the ease and safety of elderly individuals' movement and reduces accessibility. The most suitable types of flooring are vinyl or soft coverings. Selecting flooring materials that are susceptible to waxing or any similar treatment that may cause the surface to become slippery should be avoided; as such materials may lead to discomfort and reduced balance.

Assistive Handrails

Assistive handrails should be placed on both sides of the elderly’s paths of movement to ensure easy access. These handrails provide both sensory and real security to the elderly and are usually used in places where they provide the most assistance to individuals with disabilities. For example, they are essential in areas with changing angles and slopes, in bathrooms and restroom facilities, and anywhere that may require a momentary lapse of attention, such as transitioning from a dark space to a well-lit area.

Two handrails should be considered for both groups of elderly individuals, those with and without disabilities using wheelchairs, with one rail at a height of 90 centimeters and the other at a height of 65 centimeters. Additionally, to ensure that the elderly can easily grip the assistive handrails with maximum safety and confidence, the handrail section should be rounded and have an approximate diameter of 4 centimeters, with a distance of about 5 centimeters from the wall.

Physical Programs for Designing Different Spaces of the Desired Site.

Administrative spaces

Fine-tuning the design of administrative spaces in elderly homes is a crucial aspect. Research findings indicate that the majority of office spaces are allocated to locations for Administration and meetings (50 meters), while the least amount of space is allocated to pantries and restroom facilities, Table 1.

Table 1. Physical Program of Administrative Spaces

	Space Name	Unit Area (square meters)	Number of Spaces	Total Area (square meters)	Descriptions
Administrative	Administration and Meetings	50	1	50	
	Deputy Office	20	1	20	
	Accounting	12	1	12	
	Archives	12	1	12	
	Security	12	1	12	
	Staff Break Room	14	2	28	
	Staff Cafeteria	50	1	50	
	Reception	12	1	12	
	Office Kitchenette	10	1	10	
	Restrooms	5	2	10	
	Total Sum: 216 square meters				

Educational Spaces

Regarding the design status of elderly homes in Tehran, specifically focusing on the fine-tuning of the physical program for educational spaces within the facility, research findings indicate that out of the total area considered, which is 762.8 square meters, a portion of 50 square meters is allocated to educational classes such as interpretation and Quran studies, music, literature and poetry, English language, and literacy campaigns, Table 2.

Additionally, 20 square meters are designated for workshops including sewing and embroidery, carpet weaving, ceramics, and vocational workshops.

Table 2. Physical Program of Educational Spaces

	Space Name	Unit Area (square meters)	Number of Spaces	Total Area (square meters)	Descriptions
Educational	Classrooms	50	1	50	Quran Interpretation - Music - Literature and Poetry - English Language- Basic Education
	Workshops	20	1	20	Weaving (Sewing and Embroidery) - Carpet Weaving - Pottery – Technical and professional Production Workshop
	Computer Lab	12	1	12	
	Faculty Room	12	1	12	
	Restrooms	12	1	12	
	Total Sum: 762.8 square meters				

Service Spaces

In this study, 1047.8 square meters of area is allocated to service spaces, as shown in Table 3.

Table 3. Physical Program of Service Spaces

	Space Name	Unit Area (square meters)	Number of Spaces	Total Area (square meters)	Descriptions
Service	Elders' Dining Room	400	1	400	Special Menu with Dietary Foods
	Kitchen	100	1	100	
	Food Storage	30	1	30	
	Cleaning Room	6	2	12	
	Women's Hair Salon	40	1	40	
	Men's Hair Salon	30	1	60	

	Bank	50	1	50	
	Tea House	30	2	60	
	Taxi Waiting Room	12	1	12	
	Prayer Room- Ablution Room	60	2	120	
	Supermarket	100	1	100	
	Bookstore	50	1	50	
	Restrooms	6/6	8	52/8	
	Engine Room and Facilities	400	1	400	
	Technical Equipment Storage	50	1	50	
	Security and Control	10	1	10	
	Total Sum: 1047.8 square meters				

Recreational and Cultural Spaces

Table 4 presents some items related to the design status of comprehensive recreational and cultural spaces in elderly homes. The results indicate that out of 1434.8 square meters of recreational and cultural spaces, the majority of the areas are allocated to the main hall and gallery for displaying artworks.

Table 4. Physical Program of Recreational-Cultural Spaces

	Space Name	Unit Area (square meters)	Number of Spaces	Total Area (square meters)	Descriptions
Recreational-Cultural	Library				
	Study Hall	100	1	100	
	Loan Office	10	1	10	
	Library Supervisor Room	12	1	12	
	Lounge	150	2	300	
	Meeting Hall				
	Main Hall	500	1	500	
	Control Room	10	1	10	
	Waiting Area	100	1	100	
	Backyard	50	1	50	
	Gallery for Displaying Artworks	2	200	400	
	Office for Elderly-related Associations	30	5	150	
	Restrooms	6/6	8	52/8	
	Total Sum: 1434.8 square meters				

Detailed Physical Program for Therapeutic, Rehabilitation, and Sports Spaces

In this project, 1851.8 square meters are dedicated to therapeutic, rehabilitation, and sports spaces, categorized as shown in Table 5.

Table 5. Therapeutic, Rehabilitation, and Sports Spaces

	Space Name	Unit Area (square meters)	Number of Spaces	Total Area (square meters)	Descriptions
Therapeutic	Reception	12	1	12	
	Waiting Hall	50	1	50	
	Doctor's Office	16	2	32	
	Examination Room	16	2	32	
	Nursing	30	1	30	
	Inpatient	30	2	60	Three Beds per Room
	Counselor’s Room	20	1	20	

	Pharmacy	90	1	90	
	Speech Therapy	20	2	40	
	Therapy Group	35	2	70	
	Physiotherapy Hall	150	2	300	
	Chess Room	45	1	45	
	Yoga Hall	100	2	200	
	Pool - Hydrotherapy				
	Reception and Waiting Area	45	2	90	
	Shoe and Towel Delivery Area	12	2	24	
	Locker Room	30	2	60	
	First Aid	12	2	24	
	Shower	5	8	40	
	Staff Room	30	2	60	
	Buffet	20	2	40	
	Swimming Pool	120	2	240	
	Rest Area	60	2	120	
	Massage Room	60	2	120	
	Restrooms	6/6	8	52/8	
	Total Sum: 1851.8 square meters				

HVAC System of the Project

The comprehensive design status of the HVAC system for elderly homes in Tehran is indicated. An evaporative cooler as well as a mist system are used to reduce dry air temperature. However, the process of water interacting with air in this system is different.

In the evaporative cooler, water is pumped under high pressure towards nozzles arranged in multiple rows along the airflow path. As a result of the pump pressure and passing through narrow nozzle openings, the water turns into a powder form, increasing surface evaporation. Water spraying from the nozzles not only reduces the passing air temperature but also removes dust and purifies the air to some extent.

If needed, to increase capacity and reduce the outgoing air moisture, a coil can be installed and circulated with cold water produced by a cooling tower to provide better conditions. By installing a hot water coil in the evaporative cooler and supplying it through the central engine, this equipment can also be utilized in winter. Installing a return air duct to recirculate a portion of the warm air can be highly beneficial in terms of energy efficiency.

DISCUSSION

Activities that can be effective in improving the mental health of the elderly include socializing, recreation, staying active, having a warm family environment, engaging in conversations, and providing recreational facilities. Therefore, a place that can meet all these needs and allow the elderly to live alongside their families can be referred to as elderly care centers. This place can be built with the best facilities and specific standards for the elderly and the disabled, providing necessary equipment and amenities for suitable exercises according to their age.

The need for gathering and socializing spaces for the elderly and counseling centers - emotional needs of the elderly always exist, and as individuals age, their needs increase accordingly. Considering lounges as places for sitting and resting and creating spaces that can facilitate effective communication among the elderly is essential. These spaces can include sitting and viewing areas.

It means creating diverse spaces where nature is harnessed to meet the cooling and heating needs of the elderly, which are highly sensitive. The sound of water and chirping birds can be the best environment for tranquility and creating new connections. Therefore, having a clean and attractive environment significantly contributes to this matter. Color, lighting, and the surface texture are among the most important features of creating a suitable space for the elderly. This set demonstrates that a high percentage of the spaces are allocated to social spaces and lounges.

The best suggested activities for the elderly (based on their literacy and individual preferences) include reading books and newspapers, strolling in green spaces, socializing and conversing with friends, watching movies, buying from buffets, sitting on benches and observing the streets, watching children play, and listening to music. Additionally, extraordinary events such as religious ceremonies, celebrations, concerts, and light games have a positive impact on the mood of the elderly and prevent monotony in their daily lives.

Furthermore, since the elderly spend most of their day in such environments, it is recommended to provide services such as hair salons and commercial stalls that cater to their needs. Moreover, if they have handicrafts, they can offer them for sale in these places. Service spaces should be designed in a way that allows the elderly to independently use them.

Bathroom facilities in such spaces should be separate for men and women. Additionally, washing and drying equipment should be easily accessible, and access to the bathroom from outside should also be provided for necessary occasions. The bathroom for those with physical disabilities should be designed in a way that enables independent use, as it is essential for the self-esteem of the elderly [9].

The restaurant is equipped with a large, well-equipped kitchen. Providing group catering services for a large number of individuals in such communal spaces requires mechanization, cost-saving work processes, electronic data processing (DP), and automated units known as "programmed kitchens" (from menu planning to provisioning and distribution of food and dishwashing). Preparing meals for over 800 to 1000 people involves different table settings and dishes. Preparation tables and serving areas are heated either by steam or electricity. The surface temperature of the table should be kept around 60°C [15].

Depression is more prevalent among elderly individuals who live alone and also experience physical discomfort. Therefore, it is necessary to create a suitable and better environment for their living to reduce the depressions caused by loneliness. Elderly people who, despite their old age, participate in social activities and have relatively good social relationships are less prone to various diseases, especially mental illnesses.

Another important issue is work. Work is the best way to spend one's old age. Work not only meets a real need in relation to the elderly, but with proper management, the results of the work of the elderly can address many aspects of societal needs. Therefore, self-employment workshops are recommended in this residential facility.

These workshops include revitalizing activities that are made available to the elderly upon request, including greenhouses, pottery workshops, and handicrafts. If these activities yield results, they can be sold in commercial booths within the same complex so that the elderly can see the results of their work and experience a sense of usefulness and productive living.

The elderly need physical and mental security, and this should be taken into account in the design of spaces related to the elderly. The elderly should feel secure that they can seek help in times of need or receive prompt treatment during illness. They should feel calm and free from harassment in the spaces they occupy. These examination rooms should have double doors with a total width of 120 centimeters, and it is recommended to consider a room area of about 18 square meters.

Due to the high age of the residents in this complex, the probability of accidents resulting from old age increases. It is necessary to have a short distance space for this purpose, along with a necessary medical practitioner. However, since it is a temporary space, it should have a parking area equipped with an ambulance for necessary transfers to hospitals.

Elderly individuals can follow exercise programs similar to those designed for young people in terms of principles and adaptability. Therefore, open-air or covered halls suitable for sports such as volleyball, table tennis, etc., are considered in this complex. Furthermore, diverse pathways for walking and cycling can be created. Due to the elderly's age and the feeling of fatigue during the journey, there should be resting areas for sitting and relaxing, accompanied by a buffet and beautiful scenery. Boating and water activities offer a sense of vitality and freshness to the elderly. If there are suitable environmental facilities, this recreational sport can be utilized. Otherwise, aquatic physiotherapy and hydrotherapy can be used.

On the other hand, activities with lower energy costs, such as golf, bowling, billiards, and gentle stretching exercises, have less impact on physical fitness but can be effective in creating a sense of vitality in the elderly. For this purpose, spaces with suitable views and ventilation can be used for non-aerobic games such as billiards, chess, etc.

In conclusion, considering the mentioned topics and the fact that Iran is among countries with a large elderly population. Also due to the increasing number of active elderly individuals living alone and independently in their own homes in developed countries and Iran, the adherence to the mentioned guidelines for the elderly is essential in architectural designs, especially in green spaces. Designers and architects should incorporate them into their designs to create a suitable, usable, and enjoyable environment for the elderly, as they will be part of this segment of society in the future.

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GEOTECHNICS

Contemporary dynamic processes,
Geotechnical environments separated during the research

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MODELS OF CONTEPMPORARY GEODYNAMIC PROCESSES ON THE EDGE OF THE SARAJEVO DEPRESSION

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SUMMARY

The rim of the Sarajevo depression was exposed to a strong construction expansion, which was not followed by spatial planning documentation. This work presents models of modern geodynamic processes, which are mainly caused by technogenic effects. The characteristics of these processes mainly depend on the morphological properties of the terrain, geological structure, engineering geological and hydrogeological properties, as well as technogenic factors. The aim of this work is to clarify the specificities of modern geodynamic processes in the areas of different lithological formations, and also to show their classification and special treatment considering process's dynamic, the development method of the moving rock mass, causes of its movement; all with the goal of preventing or reclaiming the negative effects of these processes. The key factors that affect modern geodynamic processes are: terrain morphology, lithological structure, groundwater level, rock mass structure, as well as the technogenic processes.

Key words: *modern geodynamic processes, landslips, landslides, Sarajevo depression.*

INTRODUCTION

The problems of landslides, as well as other modern geodynamic processes, have been present since ancient times, with the fact that sudden urbanization and the construction of high-rise buildings and road infrastructure led to their significant activation. The first steps in solving and studying this issue on the territory of the former state were taken by professor M.T. Luković [1,2].

„Landslide is a complex slope process of movement, usually of disintegrated and weakly bound rock masses, along a clearly expressed shear surface, which is called a sliding surface. Part of the rock masses that, due to various factors, move by shearing down the slope, without separating from the ground, is called a landslide, i.e. the body of a landslide.“ [3].

The basic question that is posed to the experts is: how to forecast the tendency of the terrain to appear unstable [4] explained that matter in great detail and for practical purposes. According to Professor J. Perić [5] the most widespread is the lithological criterion, that is, defining geological environments“ [3]. This criterion is the basis of this work.

Engineering geological research in the area of the city of Sarajevo and its surroundings began after the World War II, and intensified to a significant extent, with the sudden urbanization and construction of the city of Sarajevo. Numerous investigative works were carried out, for individual structures, with the aim of defining the engineering-geological properties of the rock masses from which the terrains were built, on which future construction objects will be based, whether they are road infrastructure objects or high-rise buildings.

RESEARCH METHODOLOGY

During the research of the terrain, that is, unstable slopes around the edge of the Sarajevo depression, the most modern methods were applied, both in the field and in laboratory conditions. Based on these results, their synthesis was performed and characteristic types of instability were isolated.

A number of shallow boreholes, exploratory cuts and exploratory pits were carried out, certain geophysical investigations were also applied, as well as a significant number of laboratory geomechanical, mineralogical and petrological tests, as well as a numerous engineering geological and hydrogeological maps have been created for specific areas of Sarajevo and its surroundings, all aimed at defining the conditions for the construction of certain structures or addressing the stability conditions of specific slopes, including already activated landslides.

During the 1970s and 1980s, extensive regional geological research was carried out, the final result of which was the „List Sarajevo, OGK SFRJ“. With this paper, all doubts about the age of individual lithological formations, their position in the geological column, and the physical-mechanical properties of individual lithological members, their hydrogeological properties and function in a broader sense were defined to a significant extent.

RESEARCH RESULTS

The geomorphological characteristics of the terrain and its rezoning

In view of the expressed very large height differences both in the area of the city of Sarajevo itself and in the area of its wider surroundings, we can single out two more significant regions in a general sense:

- The area of the mountain rim (Mountain region), where several different levels can be distinguished, which have special lithological, hydrogeological, structural and hydrological properties,
- The flattened area together with the depression of the Sarajevo field (Quaternary Sediment Area), where there are mainly Quaternary sediments on the surface, whose engineering-geological and hydrogeological properties are precisely dependent on the thickness of these sediments.

High mountainous area

It is an area with an altitude of more than 1000 m and includes all the surrounding mountains which are interconnected and almost completely enclose the Sarajevo basin (Igman-Bjelašnica-Trebević-Jahorina-Crepoljsko, etc.). Only in Bjelašnica can we state larger amounts of denudation-glacial origin, while the erosion process (karst) takes place mainly in the inner part of the rock massif in the form of chemical disintegration of the rock mass.

Middle mountainous region

The area, which in terms of geomorphology is significantly more complex and covers an area of 700 m above sea level. up to 1000 m above sea level On the northern slopes of Trebević, Jahorina, Romanija, and on the southwestern slopes of Crepoljsko, we can see isolated and large massive limestone blocks of the Anisian layer T_2^1, which lie in a concordant relationship over the Verfen sandstones (Sarajevo sandstones) T_1. It is at the contact of these two lithological units that numerous larger or smaller wellsprings appear (Spring of Bosnia, Spring of Miljacka).

Lowland belt

In morphological terms, this area significantly differs from the previous ones, as in addition to the Middle Triassic limestones and dolomites, as well as the Verfen sandstones, Tertiary and Quaternary sediments are present, represented by terraces (river and lake), occasionally with deluvial sediments, as well as fossilized alluvial fans.

Sarajevo field depression

From the morphological aspect, it is a terrain that has gentle, sometimes steep slopes around the edges of Sarajevo itself (at the contact with the previous morphological unit), up to a completely leveled area in the area of Sarajevo Field. The slopes are covered usually with a thin deluvial cover, in some places we can see smaller-sized sinkholes, while the Sarajevo field depression itself is built of fluvial (Quaternary) sediments of the rivers Bosna and Miljacka, which are somewhat thinner around the edges, while they are significantly thicker along the river courses themselves. In the area of Čengić Vila, the thickness of fluvial sediments is about 2 meters, while their thickness increases significantly towards the open area of Sarajevsko field. Fluvial sediments are represented by alluvial terraces (alluvium), then dominantly by the gravel fraction, as well as by occurrences of lenses of sand, clay and silt. For this area, the engineering-geological properties of the terrain depend to a significant extent on the level of groundwater oscillation.

From the aspect of this paper, and with the aim of defining the engineering-geological properties of the terrain, which are important for the construction of buildings and infrastructure in Sarajevo itself, the low-lying areas and the area of the Sarajevo field depression are important to us.

Geological structure of the terrain

The geological structure of the terrain of the city of Sarajevo and its wider surroundings can be classified into four geological units, namely:

a) Lower Triassic or Verfan sediments (T_1), in which we distinguish three lithological units:
   - A complex of sandstones, marls and clays, where from a geotechnical point of view we can state that this series represents an unfavorable working environment, especially if the structural position is unfavorable in relation to the slope. In terms of hydrogeological function, this lithological series represents a hydrogeological insulator. In the geological past, it was exposed to strong tectonic processes and strong influences of exogenous factors, primarily the denudation process. (Sedrenik, northern slopes of Trebević, Pale, Mošćanica). From the engineering-geological aspect, it can be classified as rocks with unfavorable properties, where the terrain is conditionally stable on slightly inclined slopes, while on steep slopes it is conditionally stable to unstable.
   - The complex of layered white and red sandstones (colorful Sarajevo sandstones), which are more favorable than the preceding lithostratigraphic unit from an engineering-geological point of view, and the slopes built from these sediments are stable to conditionally stable (Trebević, Sedrenik, Hreša, Pale, Mokro).
   - The complex of marls and thin-layered marly limestones is less widespread and can be found in the area of Kijevo, as well as in places on the northern slopes of Trebević. According to their engineering-geological properties, they are similar to pure limestone rocks.

b) Kompleks krečnjaka i dolomitičnih krečnjaka srednjeg trijasa ($T_2^{1,2}$)
   - Massive limestones of the Anisian floor of the Middle Triassic (T_2^1) they build the lowest parts of the rim of the Sarajevo depression, they are characterized by a high degree of cracking, large systems of cracks along which shearing and tearing of large blocks occurred

along the rim. As a working environment, they are very favorable, these rocks can be used as a quality building material, while the natural slopes are stable.

- Bedded to stratified limestones of the Anisian and Ladinian stages of the Middle Triassic, characterized by a high degree of karstification, significant presence of fractures in larger dimension fissures. Alluvial fans have formed at the bases of these rock sections, natural slopes are stable, and these rocks are less commonly used as construction materials due to the presence of chert.
- Layered thin-plate limestones and marls of the Ladina layer, significantly twisted and folded, natural slopes are conditionally stable to unstable. They are rarely used as building material.
- Volcanic sedimentation complex of the Ladin layer, where in addition to thin-plate limestones with cherts and marls, rocks of volcanic origin include tuffs and spilites. Depending on whether they were discovered or not, then on the structure of the rock mass, they behave differently according to erosion-denudation processes.
- The first three formations in the hydrogeological sense function as conducting collectors, while the last formation functions as an insulator.

c) Jurassic-Cretaceous J,K (Tithonian-Valendian) flysch

This lithostratigraphic unit is represented by breccias, breccia limestones, marls and clays that alternate in the geological column, and each of the individual facies of this complex has its own engineering geological and hydrogeological properties. In general, we cannot observe them individually but as a complex, and conclude that this complex has different engineering-geological, geotechnical and hydrogeological characteristics that depend on the individual facies of this flysch complex. In places, this lithological formation functions as a hydrogeological collector, and in places as an insulator. It is spread around the perimeter of the Sarajevo depression and is isolated in the localities of Hreša, Ljubna, Koševski potok, Zujevina, Rakovica and some other localities.

d) Tertiary complex of multifacies freshwater sediments

This complex of freshwater sediments in engineering geological terms is very colorful, depending on which facies is dominant. In the area of the Sarajevo depression and its rim, three series are distinguished:

- A series of conglomerates, sandstones with thin layers of marl occurs in the area of Sedrenik, Vrace, Lukavica and Kasindol. This series is characterized by a general dip towards the Sarajevo field (west-southwest), so the stability of the slopes essentially depends on the orientation of the interlayer surfaces as well as the cracks that cross them.
- Facies of weakly bound sandstones developed along the stretch from Podhrastovi—Veliki Park—Mejtaš—Hambaina carina. They have a distinct collector function, which is of crucial importance for the stability of slopes.
- A series of thin-layered marls, sandstones and clays (Koševska series) make up the area of Koševski potok, Čengić vila, Lukavica, Kobilje glave. The lithification process is not complete, i.e. these rocks in places change to their unbound equivalents, and this series can be characterized as unfavorable in terms of engineering geology. In the hydrogeological sense, different types of outcrops were formed in this formation.

e) Quaternary cover, represented on the slopes by mostly thin, less often thicker deluvial cover, then by sipar, and by alluvial sediments along the river courses, of different thicknesses.

Contemporary geodynamic processes on the edge of the Sarajevo

Landslides can be found practically in all geological formations in our country: from the oldest to the most recent [2]. Contemporary geodynamic processes, which take place along the perimeter of the Sarajevo depression, were previously caused by the physical-mechanical disintegration of primary rock masses which, under the influence of certain factors, then moved down the slope, and then in recent geological history, the consolidation and binding of these rock masses took place.

This is how a primarily deluvial cover was formed, which can be of significant thickness in places. This deluvial cover represents a relatively "suitable" environment for the formation of landslides, that is, for the detachment and uncontrolled movement of this mass down the slope if certain assumptions have been made beforehand (gravitational conditions, water saturation of the terrain, disturbance of the primary geostatic conditions in the terrain, etc.). Landslides are most common in Tertiary sediments, followed by Flysch Jurassic-Cretaceous (J,K), while contemporary geodynamic processes in the Triassic are represented by landslides, landslides and seeps, as accumulative forms.

a) Landslides in Triassic sediments (T_1, T_2^1)

These landslides were formed on the steep slopes of Triassic limestones, in the bottom of which lie white and red (Sarajevo) sandstones. Previously, on the slopes, sluices of significant dimensions were formed, partially cemented with clayey material. In natural conditions, these slopes are relatively stable, while in the process of building road infrastructure, as well as high-rise buildings, and during the exploitation phase of roadways, there was an occurrence of instability and the activation of sliding and sliding processes (Lapišnica landslides, landslides on the road from Hreša to Sumbulovac, as well as landslides on the northern slope of Trebević, the Vraca–Brus road), as shown in Figure 1. Uncontrolled dumping of soil and other waste material on a naturally unstable slope, in the area of Lapišnica, led to the occurrence of landslides of enormous proportions.

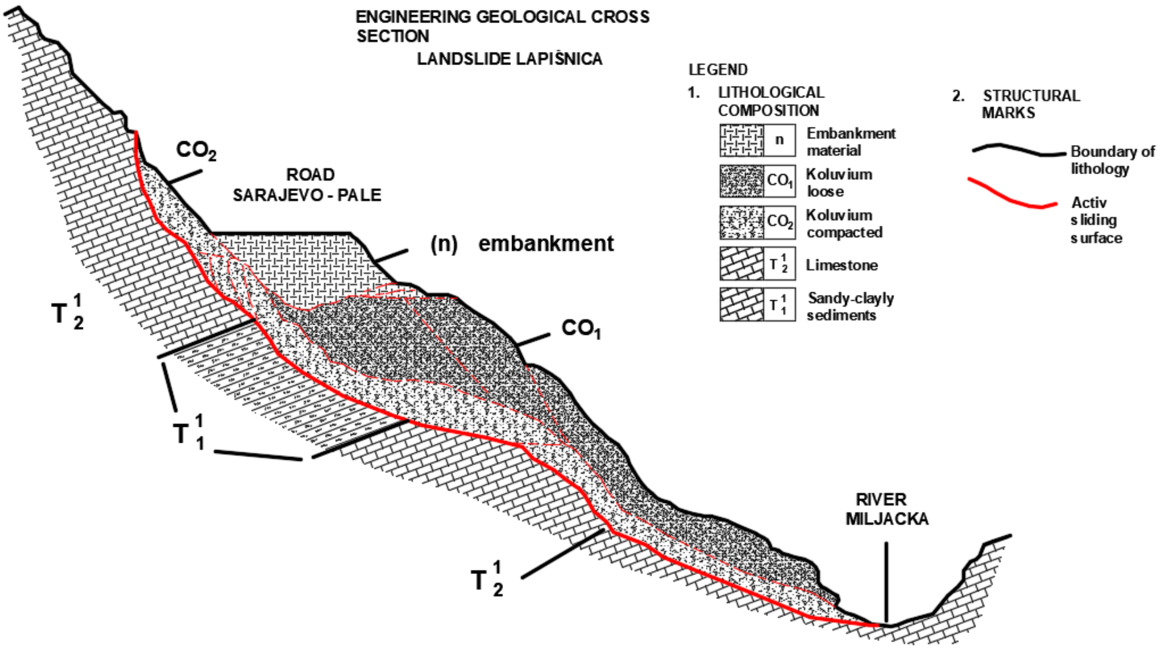

Figure 1. Lapisnica landslide

At the level of the road, an embankment several tens of meters thick was formed in order to create a leveled plateau, on which the construction of the building was planned. Backfilling was carried out over the colluvial cover on the steep terrain, which is located on the Middle Triassic, heavily crushed limestones, and in the deeper parts of the terrain, below them is the Lower Tiacian sandstone-clay complex.

As a result of the heavy load from the filled mass and the large amount of water, both surface and underground, the slope was moved and the road was washed away on a length of 170 meters. The moved earth mass also threatened the bed of the river Miljacka, and there was an objective danger of the formation of a dam and a lake in the upstream part.

„The occurrence of the Lapišnica landslide was predisposed by the heterogeneous geological structure of the terrain (in complex spatial relationships), and the impact of the newly formed landfill in conjunction with specific hydrological and hydrogeological conditions was decisive for the loss of balance and radical displacement of the terrain on 19-20.11. 1999“ [6].

b) Landslides in Jurassic-Cretaceous flysch (J,)

In the higher parts of the terrain, there is a flysch polyfacies complex built of: marls, calcarenites, sandstones and clays, and in the lower parts, the middle Miocene complex, represented by marls, sandstones and clays, and clays with carbonaceous interlayers, lies transgressively and discordantly over it. The flysch complex is intensively harvested, tectonically deformed, blocky and superficially decomposed, and the Miocene sediments are weakly diagenetically consolidated and subject to relatively rapid changes [7,8]. In the flysch complex, decomposition products are gravitationally moved towards the lower parts of the terrain, creating colluvial deep curtains as shown in Figure 2. These are slow, shallow to deep landslides. The causes of landslides are related to the natural morphogenetic development of slopes, surface and underground water, as well as technogenic activity.

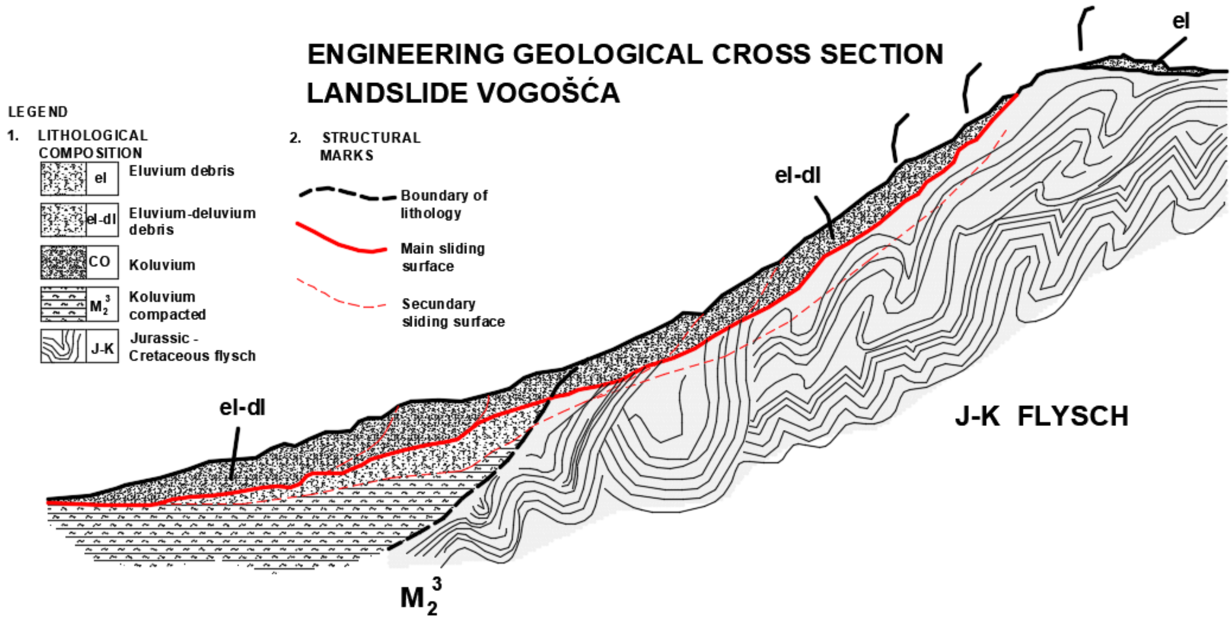

Figure 2. Landslide in the flysch complex

c) Landslides in the tertiary complex (Tc)

Landslides that are formed in the geological substrate of tertiary deposits where there is a different thickness of the decaying crust, then different lithological composition and different hydrogeological properties. In this lithological complex, landslides are formed according to the stated conditions, the position of the Quaternary sediments on the slope, the water saturation of the terrain, the disturbed natural geostatic condition in the terrain, the "favorable" position of the structure of the tertiary sediments for the formation of the sliding surface. In the area of Koševski potok and Sušica potok, a large number of smaller landslides were formed due to the disturbance of the primary geostatic condition in the terrain, and a typical representative of this type of landslide is the Crni Vrh landslide, which is shown in Figure 3.

The Crni Vrh landslide is not unique, but because of its specific the structure of the substrate is actually divided into three landslides [7]. The lithological complex of tertiary sediments does not have an extremely unfavorable structural orientation and the landslides were formed in the first phase at the substrate-cover contact, so that in the further phase of landslide development, the substrate would also be partially affected.

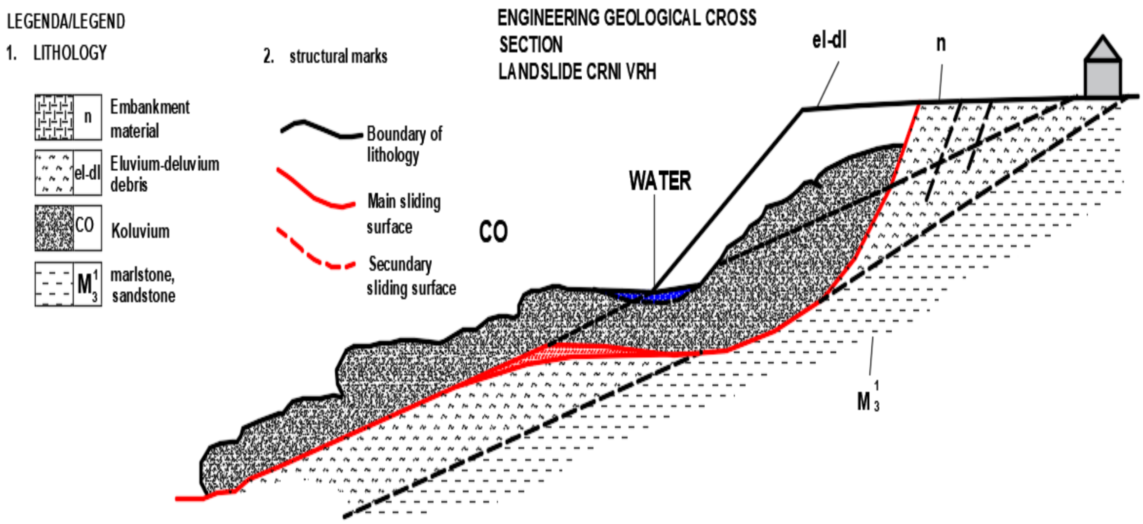

Figure 3. Landslide Crni Vrh

Deformations, i.e. the slope process of moving the mass takes place in this way, partly where the orientation of the structure of this lithological formation allows, they are also transferred to the substrate, i.e. the base rock. In the sloping parts of the city, landslides of this type are very common and are found on high embankments next to the streets, on local garbage dumps, etc. According to the activity of the sliding process, these are relatively fast landslides that require urgent interventions to prevent their further uncontrolled spread.

d) The slope in the Pofalići-Velešići area is built of Upper Miocene sediments (M_3^1) the so-called "Koševo series" of marls, marly clays, clays, sands and marls shown in Figure 4. The complex is layered with the position of the structure down the slope, which further worsens the stability of the slope in natural conditions. The inclination of the slope varies from 15^0 to 30^0.

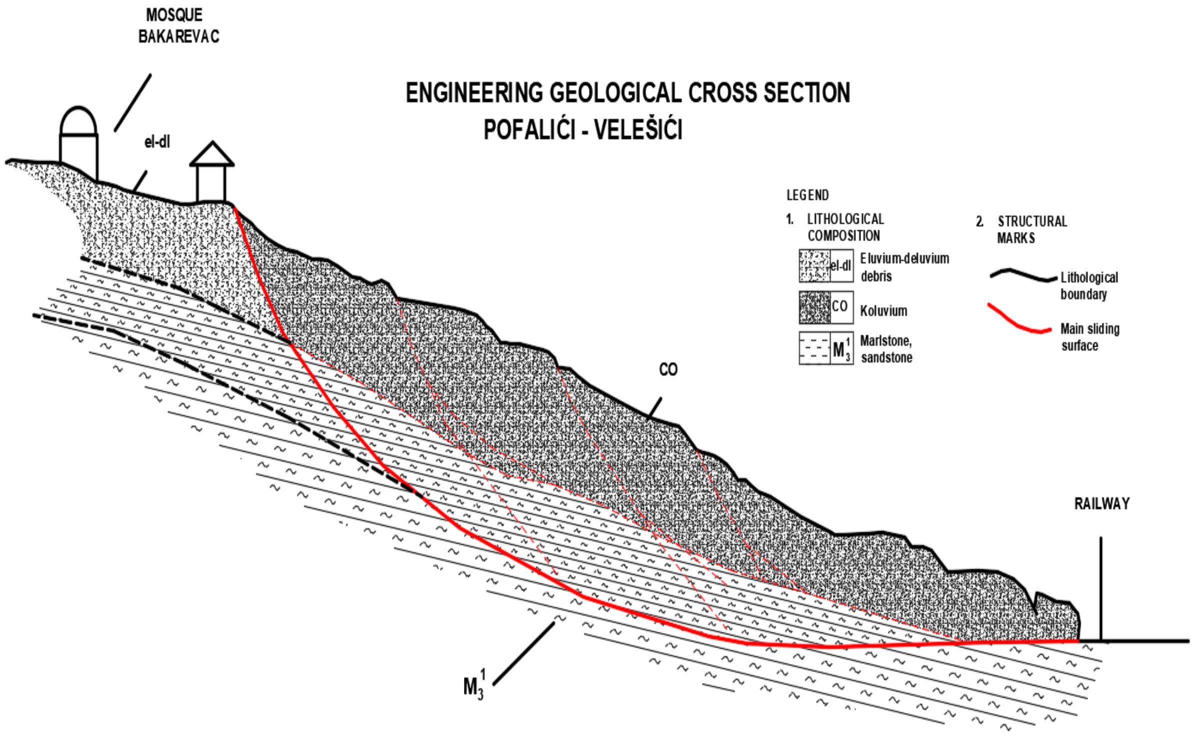

Figure 4. Landslide Pofalići - Velešići

The initial movements of instability occurred at the foot of the slope, so that the process of mass movement down the slope would successively spread towards hypsometrically higher parts. In addition to the natural predisposition of the terrain, additional man-made human activities, primarily the illegal construction of buildings, there was a significant activation of the movement of the rock mass down the slope.

e. Quaternary landslides (Q)

Landslides in the Quaternary were formed as a result of gravitational movements of rock masses, mainly in a loose cover as shown in Figure 5. These landslides are connected to the sources of streams and rivers on the edge of the Sarajevo depression and are located at high elevations, and in the lowest parts of the basin where streams flow into larger watercourses (mainly in plavine fans). The slope is built of eluvial deluvial cover in the surface part of the terrain and Neogene sediments, the so-called Koševo series.

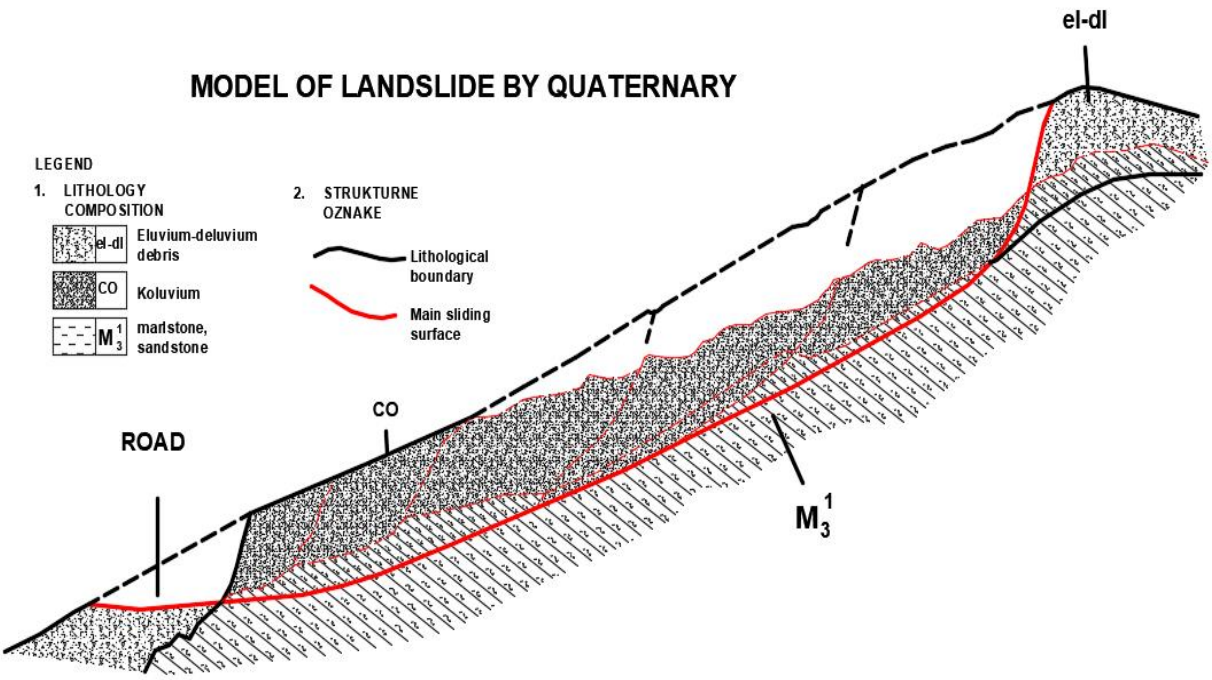

Figure 5. Landslides in quaternary formations

In the case of inadequate cutting of the base of the slope, when making frontal excavations for linear objects or foundations of stable objects, the initial movements occur immediately behind the formation of the cut, and then the sliding process spreads successively towards the hypsometric parts of the terrain until local equilibrium is established.

THE RESULTS ANALYSIS

In this paper, five characteristic types of landslides are presented, with clearly defined conditions of their occurrence. The causes of the occurrence, i.e. activation of the sliding process, can be reduced to: geological, geomorphological, physical and technogenic.

The geological conditions for the occurrence of landslides are found in the rock masses themselves, in the manner and conditions of their formation, physical-mechanical properties that reflect their ability to react faster or slower to all external agents of destruction.

Geomorphic conditions are related to the morphogenetic development of slopes, i.e., the shaping of the terrain under the influence of contemporary exogenous processes.

Physical or climatic conditions are related to the amount of precipitation, sudden freezing and melting of snow.

Technogenic activity or human activity is the most common cause of landslide activation. These landslides, in addition to their frequent occurrence, are characterized by rapid and uncontrolled processes, which often have serious consequences for residential and infrastructure facilities.

CONCLUSION

The area of the Sarajevo depression abounds with numerous occurrences of landslides and rockfalls, primarily due to the geological structure of the terrain as well as their very complex tectonic structure.

The basic geological environments that favor the formation of landslides are limestones in a morphologically scattered relief, which lie over Verfen sediments, then Neogene sediments, flysch sediments and the Quaternary cover.

The area surrounding the Sarajevo Depression has been exposed to strong construction expansion for the past 40 years, mainly the construction of individual buildings, then the construction of the infrastructure that had to accompany the construction of these buildings (road, water and sewerage, electricity lines, etc.). There are very few areas where there was spatial planning documentation, i.e. illegal construction is present to the greatest extent, which was almost nowhere followed by geological surveys, assessments of the stability of cut slopes, assessments of the bearing capacity of the terrain when building foundations, etc.

Thanks to the extremely thin decomposition crust (in places we also have a somewhat thicker decomposition crust), then the favorable orientation of the rock mass structures, the lithological structure of the substrate, during the execution and construction of these constructions, the investors did not face any significant problems, which required expensive rehabilitation (the exception is landslide Lapišnica).

The paper presents models of contemporary gliding processes that took place along the peripheral parts of the Sarajevo depression, mostly initiated by technogenic processes, i.e. human work. In places, there are several smaller landslides right next to each other, but essentially they do not represent a single whole, primarily due to the position of the rock mass structure in relation to the terrain surface (Crni Vrh landslide).

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SEPARATED GEOTECHNICAL ENVIRONMENTS DURING THE RESEARCH OF THE TERRAIN AT THE LOCATION PLANNED FOR THE CONSTRUCTION OF THE SILOS IN KOZLUK NEAR ZVORNIK

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ABSTRACT

The construction of high-rise objects in solid rocks characterized by frequent changes in its vertical profile requires a more detailed study of lithological layers, tectonic characteristics and analysis of physical and mechanical parameters. Layers of different strength and characteristics in all elements make it possible to separate it by geological environments, in order to choose the most favorable one for the foundation of objects. As part of the construction of additional silos at the existing "Molaris" Mill complex in Kozluk near Zvornik, research was conducted at the central points of the future facilities and the immediate surroundings.

Investigation works up to a depth of 12.0 m defined all the complexity of the terrain structure in a small area. Conducted research on rock samples from exploration works and from the open terrain profile showed differences in the physical and mechanical characteristics of softer rocks susceptible to solubility in water. Separated environments show that the foundation of objects cannot be done only in one environment, given the folds of the layers in a very small area.

Key words: *geotechnical environment, object foundations, characteristics of the rocks*

INTRODUCTION

At the location of the complex of facilities of the "Molaris" Mill, the construction of additional silos is planned on the plateau between the existing facilities and the cut part of the slope. In that area, the slope used to be characterized by a steep fall towards the Zvornik - Bijeljina regional road. Over time, it was cut and buildings were built on the plateau, which is at the level of the regional road. The present day appearance of the slope in the back of the buildings is of the type of subvertical incision of about 6.0 m.

The bedrock on the plateau is characterized by solid rocks, which can be connected to the rocks on the open part of the slope. Conducted field research showed that the terrain in the vertical profile is characterized by a folded shape with a synclinal dip of layers around 45^0. Softer marly rocks alternate with fairly solid sandstones of different colors in the profile. By comparing the rocks from the vertical profile obtained by exploration works and the same rocks on the open terrain profile of the subvertical cut, the differences in the softer marl rocks are shown.

The diversity of layers or geological environments in the part of the foundations of future facilities will require a specific way of connecting the construction of the building with the construction of the terrain. The degree of cracking of rocks that have been separated into geological environments is quite present. By implementing the results obtained by field and laboratory tests through the RockLab system, the parameter values reduced to the natural estimated state are significantly lower.

BASIC CHARACTERISTICS OF THE TERRAIN

The terrain of the immediate location has not been specifically studied, except as part of the preparation of the Basic Geological Map of SFRY. sheet Zvornik 1:100 000. Basic data on geological, geomorphological and structural-geological characteristics of the terrain are given in the Map Interpreter for the mentioned sheet, which are not sufficient for the details of the location where the objects are being built [1]. Previously built buildings also did not have the necessary documentation, but the construction was carried out on the basis of direct observation of the terrain during the arrangement of the plateau and the cutting of the slope.

By studying the terrain at the location of the future facilities and the immediate surroundings, certain characteristics of the terrain were recorded. It represents the extreme marginal parts of a slightly steep to steep slope with a fall of about 10-20%, and a drop angle of about 90 in the slightly steep, or 18^0 in the steep parts of the slope. The orientation of the slope is in the east-southeast direction, that is, with an azimuth of about 100^0. The edge of the slope is contoured on the western side by the terrace sediments of the Drina River.

According to the aforementioned BGM sheet Zvornik, R 1: 100 000 and its interpretation, the structure of the slope includes Upper Eocene (E_3) deposits, which in the milder parts are covered by thin deluvial (d) curtains. This unit includes bank quartz sandstones, conglomerates, friable gray, greenish, sometimes ferruginous sandstones, clays, clayey marls and very rarely clays. Rhythmic sedimentation is observed, with a pronounced presence of sandstone, and to a lesser extent conglomerate and fine-grained sediments of clays and clayey marls. The general direction of deposition of the layers is north-northeast (the angle of deposition ranges from about 355^0 to about 10^0), and the dip angle of the layers is from about 23^0 to about about 28^0.

During the engineering geological mapping of the core, all lithological members were followed in detail. The sampling schedule was adapted to the position of the lithological members in depth in relation to the exploration works [1,2]. Due to the synclinal deposition of the layers, the sampling depths are different, but they represented the continuity of the same layers between the exploration works. At the same time, the position of the lithological members was defined according to their vertical occurrence and horizontal extent. The layers of clastic rocks of the substratum of the terrain, which according to the engineering geological classification belong to semi-stony rocks [2], were separated:

Fine grained to medium grained sandstone

- Medium grained sandstone (1)
- Fine grained to medium grained sandstone (1)

Clayey marl (2)

- Sandstone fine grained to medium grained (1)

Clayey, sandy marl (3)

- Sandstone medium grained (1)

The layers of lithological members alternate rhythmically, forming an incomplete flysch sequence with the absence of certain members in the vertical column [2,3,]. The relationship of the mentioned rocks in the vertical profile is shown in Figure 1.

The characteristics of the lithostratigraphic assemblage classify the terrain and the immediate surroundings as conditionally favorable to favorable terrain. From a geotechnical point of view, terrains with a permissible load of > 300 kN/m2 belong to terrains of favorable stability, if other factors such as the disintegration or cracking of rocks, the inclination of the slope and strata are favorable [1,2].

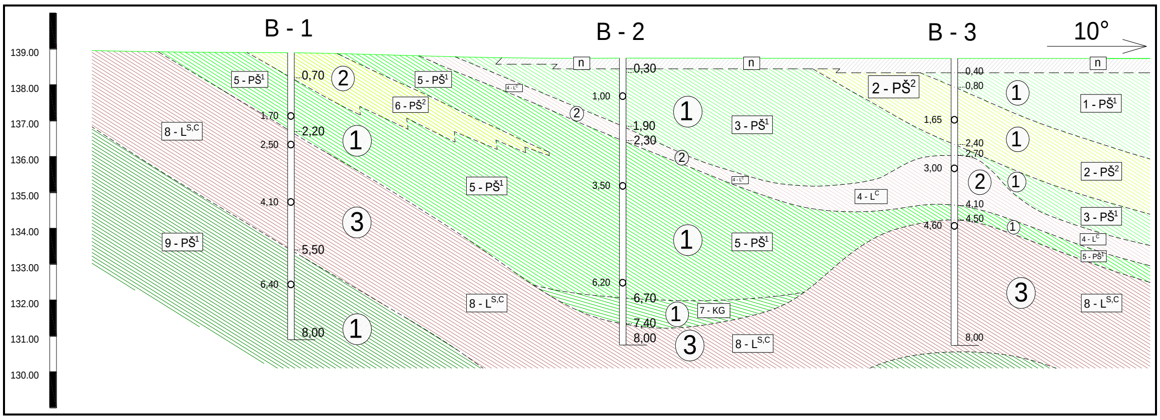

Figure 1. Terrain profile along the axis of future objcts

Directly at the location of the future buildings, the terrain in the vertical profile is characterized by layers of variable characteristics due to the presence of soft marly rocks [3]. The steep slopes of the layers make it difficult to build foundations in the same layer, which is why it was necessary to separate geological environments with the same or similar characteristics.

RESARCH METHODOLOGY

For a more detailed study of the terrain, a research methodology was chosen that will supplement the existing knowledge and study the area of the location itself in detail. It included:

- Field research was carried out that supplemented the data of the Basic Geological Map of SFRY, Zvornik sheet, 1:100 000.
- Research works along the axis of future buildings up to the depth of the influence of their load defined the lithological members and their occurrence in depth.
- Laboratory tests included the determination of physical-mechanical and resistant-deformable characteristics of rocks in conditions of 18 – 20^0 C.

During tests in the field and in the laboratory, the data of quasi-homogeneous sediments of the first and long marly layers were correlated, for the possibility of observing them as one layer.

RESEARCH RESULTS

The geotechnical characteristics of the terrain were determined from the aspect of the terrain's properties as a working environment in which the facilities will be based. For the analysis of the geotechnical conditions for the design and construction of silo facilities at the location "Molaris" in Kozluk, the City of Zvornik, the construction of the terrain was analyzed in detail in relation to the lithological types of rocks, their position within the studied depth of the terrain as well as their mutual position, then their condition, composition, engineering geological and hydrogeological characteristics and physical - mechanical and resistant - deformable characteristics [4,5,6,7,8,9]. Correlation of the data obtained by exploratory drilling was carried out with the data of the open terrain profile located a few meters from the planned facilities, Figure 1. The subvertical cross-section of the slope shows all the complexity of the geological structure, which is also confirmed by exploratory drilling.

The terrain of the researched location, as well as the immediate surroundings to the depth of the research investigation, are built by sediments - lithological members with different physical - mechanical and resistant - deformable characteristics. Through a detailed analysis of their parameters, it was concluded that the construction of the terrain of the researched location generally consists of three (3) geotechnical environments, Figure 1. With stricter detailing, some sub-environments could also be separated. For this level of research and foundation of objects, the separated environments are characterized by lower mean values of the same or similar layers. The separated environments are:

- Geotechnical environment 1, represents a complex of layers of sandstones in more horizons of different colors.
- Geotechnical environment 2, clayey, worn out, brittle marl of gray color.
- Geotechnical environment 3, clayey, worn out, brittle marl of reddish brown color.

Geotechnical environment 1 is a complex of layers built from large clastic rocks sandstone deposited in several horizons. Sandstone rocks are fine grained to medium grained, poorly weathered to hard and well cemented material, varying in color from light gray to gray with occasional shades of reddish brown. The higher horizon is registered at different depths due to the synclinal alignment of the layers. It starts from a depth of 0.40 m and goes up to max. 2.7 m.

The lower horizons are at greater depths, although due to the cyclinal deposition of the layers, they appear in one part from the surface of the terrain. On the terrain profile, Figure 1, the middle is marked with numerical code 1, and the lithological layers with geomechanical sandstone codes as 1-PŠ1, 2-PŠ2, 3-PŠ1, 5-PŠ1, 9-PŠ1. At the bottom of the synclinal part, a layer of well-cemented conglomerate, with a lenticular character, was identified as 7 KG. The values of the parameters are in a smaller range, depending on the horizon from which they were taken, and lower mean values were adopted, table 1.

Average RQD of the environment 1 ranges from 63 to 76%.

As an environment for the foundation of objects, it is a favorable medium, stable, incompressible and has a permissible load that corresponds to individual silo objects.

Table 1. Parameters of geotechnical environment 1.

Profile mark	Lithological type	Geomechan. mark	Physical - mechanical parameters	Adopted parameters of the environment
1-PŠ^1 2-PŠ^2 3-PŠ^1 5-PŠ^1 9-PŠ^1	Fine-grained to medium-grained sandstone, poorly worn out to hard and well-cemented material	Weakly petrified to strongly petrified rock	γ = 23,66 – 26,30 kN/m³ φ = 27,0 – 32,0^0 c = 2,10 – 2,70 MPa/m^2 σ = 12,80 – 29,97 MPa ν = 0,20 – 0,26 E_{din} = 22403 – 36816 MPa	γ = 24,00 kN/m³ φ = 29^0 c = 2,40 MPa σ = 21,00 MPa ν = 0,23 E_{din} = 29000 MPa

Geotechnical environment 2, clayey marl, worn out, brittle, gray in color with the characteristics of a quasi-plastic environment built of fine clastic rocks - clayey marl. It appears from the surface of the terrain and dips synclinally up to 4.5 m, with a thickness of about 3.0 - 4.0 m, and in the extreme part of the syncline it decreases to 0.4 m, Figure 1. On the terrain profile, it is marked with the number 2 and geomechanical with the designation of layer 4-LC. The parameter values are in table 2.

Table 2. Parameters of geotechnical environment 2.

Profile mark	Lithological type	Geomechan. mark	Physical - mechanical parameters	Adopted parameters of the environment
4-L^C	Clay marl, worn out, brittle	Weak petrified rock	γ = 24,06 kN/m³ φ = 26º c = 2,10 MPa/m^2 σ = 5,23 MPa ν = 0,22 E_{din} = 15782 MPa	γ = 24,00 kN/m³ φ = 25^0 c = 2,0 MPa σ = 5,0 MPa ν = 0,21 E_{din} = 15500 MPa

Average RQD of the environment 2 ranges around 55 %

Environment 2. is a conditionally favorable environment for the foundation of objects. In its natural state, it is stable and has a satisfactory load capacity. Engineering activities may come into contact with water from atmospheric precipitation, when it very quickly destroys its physical and mechanical characteristics to the limit of soft marly clay. At the same time, it is transformed into an unfavorable environment for the foundation of objects. If the foundation of the buildings is carried out in the middle 2, it is necessary to isolate them from contact with water.

Geotechnical environment 3, with the characteristics of a quasi plastic environment, is made of fine clastic rocks marl of clayey, sandy, brittle, dark red-brown color with rare inclusions of gray hard sandstone. It appears from the surface of the terrain and goes to a greater depth from 2.2 m to 7.4 m, that is, it follows the synclinal deposition of contact layers.

The middle has an average thickness of 2.5 - 3.5 m in the area of the silo facilities, Figure 1. On the terrain profile, it is marked with numerical code 3 and geomechanical code of layer 8-LSC. The parameter values are given in Table 3.

Parameters of geotechnical environment 3.

Profile mark	Lithological type	Geomechan. mark	Physical - mechanical parameters	Adopted parameters of the environment
8-L^{SC}	Clayey, sandy, weathered, brittle marl	Weak petrified rock	γ = 23,57 – 25,50 kN/m³ φ = 25º c = 2,30 MPa/m² σ = 3,64 – 4,91 MPa ν = 0,25 – 0,31 E_{din} = 11552 – 11661 MPa	γ = 24,50 kN/m³ φ = 25^0 c = 2,2 MPa σ = 4,0 MPa ν = 0,27 E_{din} = 11600 MPa

Average RQD of the environment 3 ranges around 69 %.

Environment 3 is favorable for the foundation of buildings in their natural state. It represents a stable environment, where the rock is weakly compressible to incompressible with a permissible bearing capacity that corresponds to the designed silo facilities. In contact with water, the physical and mechanical characteristics are reduced and it passes into a conditionally favorable environment. During the execution of the works, it is necessary to take measures to protect the rocks from contact with water.

By analyzing rock strength using RocLab, lower parameter values were taken for input data. The degree of reliability of the input data of field and laboratory research in the part of investigative works is satisfactory. The data for the rock mass taken from the RocLab program package are quite well chosen [10,11,12]. Characteristics of environments recalculated in RocLab are given in table 4.

Tabela 4. Parameters of geotechnical environments calculated in RocLab

Parameters	Geolog. environment 1		Geolog. Environment 2		Geolog. environment 3	
	Intact rock	Massive in RocLab	Intact rock	Massive in RocLab	Intact rock	Massive in RocLab
γ (kN/m³)	24	24	24	24	24,5	25,4
φ = (^{0})	29	37	25	31	25	28
c = (MPa)	2,4	1.343	2,0	0.255	2,2	0.178
σ = (MPa)	21	1.559	5,0	0.268	4,0	0.179
GSI	-	59	-	48	-	45
m_i	-	19	-	12	-	9
σ_{RM}	-	5.418	-	0.911	-	0.594
E_{din} (MPa)	29000	6.510	15500	1.250	11600	1.000
ε_{RM} (MPa)	-	2.518	-	339	-	224

γ – volumetric weight	mi – a constant that depends on characteristics of the rock
φ – angle of internal friction	σ_{RM} – total strength of the rock mass
c – cohesion	E_{din} – Modulus of elasticity dynamic
σ – uniaxial compressive strength	ε_{RM} – Modulus of deformation
GSI – geological strength index	

DISCUSSION

The values of adopted parameters for separated environments are based on laboratory tests and rock quality assessment during its mapping. Taken lower average values are realistic for tests on monolithic samples [14,15,16]. The rocks in the area of the future silo facilities do not represent continuity, but there are certain mechanical discontinuities along which there may be a complete absence of cohesion forces. Therefore, there is a significant difference in strength between monolithic parts of the mass and real rock massif, as well as between different varieties of lithological members and their mechanical discontinuities.

When defining geotechnical environments that are engaged as a working environment for the foundation and construction of objects, the selection of physical-mechanical parameters relevant for geostatic calculations was made on the basis of:

- results of laboratory tests of solid rock samples, taking into account the degree of their representativeness and test conditions
- data on the real characteristics of the rock masses of the marly complex and the sandstone complex (lithological heterogeneity, structural textural characteristics, degree of cracking and characteristics of cracks, degree of surface degradation, etc.)
- existing empirical correlation links between physical mechanical characteristics, structural characteristics and rating of rock mass (Analysis of Roc/Soil Strength using RocLab)

The values obtained by the analysis in RocLab for the analyzed environments are significantly lower and they reflect the actual situation in the complete block of the rock massif. The degree of detail of the data research for this analysis was not sufficiently observed in a wider area, although in the immediate vicinity there is an open terrain profile above the height of the foundation of the building.

This method shows us the general state of the rock massif in conditions of a well studied state of the lithological members and the degree of their cracking. The results of the state of stress in the rock massif as a whole and the rocks taken in the exploration works always differ, considering that the exploration works show the real state in the point section along the vertical. It is more or less different from the situation in the immediate surroundings, which depends on the structural geological characteristics of the terrain.

CONCLUSION

The complexity of the geological structure in the small area where characteristic facilities such as silos are being built required a more detailed study of the terrain at the micro-location. Geological research carried out, which included investigative works and laboratory tests, provided basic data on the characteristics of the terrain. The presence of an open terrain profile in the immediate vicinity made it possible to correlate data with relevant works in macroscopic observation. Correlations of laboratory tests are not relevant because the rocks on the open terrain profile have been exposed to different climatic changes for years.

Frequent lithological shifts of smaller or larger layers in a small area and their synclinal bedding made it impossible to choose an adequate layer for building foundations. That is why layers with approximately the same characteristics were grouped into geological environments. Three geological environments were separated, which, in addition to the parameters of laboratory tests, were also processed through the RocLab system.

A more detailed look at the lithological composition, especially on the open terrain profile, shows that the rocks in this zone have no continuity, but a lot of mechanical discontinuities. By translating field and laboratory data through the RocLab system into the real state of the rock massif, the obtained values for the geological environment are significantly lower, but more reliable for defining the foundation conditions of the silo structures.

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MINING

Protection at work in mining

Editors

Prof. Ph.D. Božo Kolonja
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ANALYSIS OF OCCUPATIONAL INJURIES IN AN IRON ORE MINE IN BOSNIA AND HERZEGOVINA IN THE PERIOD FROM 2002 TO 2021

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ABSTRACT

By designing protection measures, the occupational safety system should provide the best possible working conditions for all employees in every economic activity, including in mining production. The analysis of occupational injuries in a certain period of time should enable the collection of data on certain etiological specificities of injuries and in this way ensure the selection of adequate preventive measures of occupational safety. Monitoring the indicators of occupational injuries within the analysis, which is presented in this paper, provides information about the change in the state of working conditions and enables the determination of the general tendency of its decreasing or increasing change.

One-factor ANOVA analysis was applied to determine the variability of the results within the established groups of selected etiological factors and to determine the impact on the observed occurrence of the number of occupational injuries in the mining company. The application of Pareto analysis to determine the most dominant causes of occupational injuries is of crucial importance for determining the direction of action of preventive actions and corrective measures in the occupational safety system.

Keywords: occupational safety, occupational injury, etiological factors, Pareto analysis, occupational safety measures

INTRODUCTION

According to the report World Mining Data for 2022, it is stated that the world mining production in 2020 amounted to 17.2 billion metric tons of ores, and the largest part of the production was carried out in Asia (59.8%), North America (15.4%), Oceania (7.3%) and Europe (6.8%) [1]. Mining is a high-risk industrial activity and the main risks that arise are: risks to the safety and health of employees, environmental risks, social risks, land use risks, legal and financial risks and technical risks [2]. The imperative that has no alternative is the efficiency of production in mining production systems with a satisfactory level of safety and protection at work for employees. The occupational safety system in every production system should provide employees with working conditions in accordance with safety norms. The management of the protection system depends to a large extent on effective planning and timely and effective design of occupational safety measures, and for this purpose, the analysis of occupational injuries in the previous period should serve as a guide for changing safety strategies, developing new models of risk assessment, information base for the guidance and correction of protection measures, as well as the improvement of legal regulations. In any case, the effective functioning of the occupational health and safety system for the company also implies certain positive economic effects in the form of a reduction in production costs: fewer lost working days (working hours) due to occupational injuries, costs of repairing the consequences of occupational injuries (sickness of workers, employment and training of new workers, disability benefits, etc.), stoppages and delays in the production process, etc.

An occupational injury is a change in the working environment on a person's integrity (physical, psychological and social), as a consequence of the sudden and immediate occurrence of inconsistencies between the behavior of a person in the performance of work activities and the elements of the working environment [3]. Damage to health, reduction or loss of working ability, material costs due to compensation for sick leave, treatment, rehabilitation, disability, reduction of life activities, disturbances in the family, disruption of the work process, decline in productivity and quality of work caused by occupational injuries make the problem of occupational trauma very topical [4]. The first prerequisite in the analysis of injuries and the use of such data for the improvement of working conditions and improvement of occupational safety measures is their recording. It is crucial to keep a record of injuries, to analyze the conditions that caused the injuries, and to use the information gathered and the experience gained in order to prevent future injuries and loss of life [5]. Injury analysis involves the application of various statistical techniques to determine the etiology of injuries and the factors that lead to their occurrence. Statistical tools have been developed in the light of epidemiological principals to see whether there is any role of some personal and impersonal factors in the occurrence of coal miner's injuries [6].

The goal of the research presented in this paper was to determine which etiological factors had an impact on the frequency of injuries in the mine in order to design appropriate occupational safety measures aimed at a given category of workers. Also, determine the dominant causes of occupational injuries, in order to direct the effect of corrective measures to the observed causes in order to avoid similar injuries in the future.

LITERATURE REVIEW

Analysis of the state of occupational health and safety in the mines of the Republic of Srpska in the period 2005-2009. show that 1 out of 30 employed workers are injured annually, that is, for every 1 million tons of ore material produced, 13.3 workers are injured [7]. In Bosnia and Herzegovina, research dealing with this issue was conducted at "Arcelor Mittal" Zenica in the period 2005-2012, on a sample of 574 injuries. There were no fatal injuries during that period. Only men were injured, in the first shift (46.6%). The injured workers were between the ages of 50 and 54 (26.6%) and qualified workers - KV (67%). Injuries affected the extremities in 60% of cases. Injuries most often occurred on Tuesdays (26.6%) and Saturdays (20%) [8]. Based on the available data on fatal injuries in mines in Serbia, it is concluded that changes in the legal regulations on occupational safety and health can significantly reduce fatal injuries. Fatal injuries were most common in excavations, and less educated workers were more likely to die [9]. Numerous researches in Europe have shown that younger workers are more often injured in similar jobs and under approximately equal working conditions, while older workers' injuries are of a more severe nature [10].

In addition to research that included the analysis of etiological factors of injuries in mining, numerous authors dealt with determining the causes of injuries. Determining and analyzing the causes of injuries in any activity, including mining, is the basis for establishing an effective occupational safety system. Research on the frequency of injuries in surface coal mines in India singled out the most significant influencing factors: unsafe behavior of workers, years of work experience and age of workers [11]. Research in China that analyzed 320 accidents in coal mines showed that 74.3% of accidents were the result of unsafe conditions and rules: unsafe operator behavior, unsafe equipment, and unsafe working environment conditions [12]. In an analysis of injuries in mines in Spain from 2003 to 2012, it was determined that collisions with moving objects, physical exertion and radiation were the main causes of injuries [13]. According to data from the Australian mining sector, 79% of injuries occur as a result of falls, trips and slips, body contact with moving or stationary objects and handling materials [14]. The most common causes of deaths in mines are motor vehicle and powered haulage accidents, falls from a height and machinery according to data for 2010 in Canada (Ontario state) [15]. According to an analysis of injuries in mines in Ghana, 85% of all injuries and 90% of deaths in mining are related to mining equipment, while less skilled workers were more often involved in fatal accidents [16]. In surface mines in Turkey (71 surface mines), miners under the age of 40 were at greater risk of injury than older ones, with the most common types of fatal injuries occurring during blasting operations, motor-driven transport, and falls [17]. According to research [18], for the period 1995 to 2004, in mines in the USA, hand tools without drive were the equipment category most often associated with non-fatal injuries, while off-road ore transport was the most common source of deaths, younger employees had an increased risk of injury, while workers over the age of 55 had an increased risk of death.

MATERIAL AND METHOD

The analysis of occupational injuries was carried out in the iron ore mine "Arcelor Mittal" Prijedor, in the Republic of Srpska - Bosnia and Herzegovina for the period 2002-2021. year, and are based on the database on occupational injuries. The injury database was created based on the recording of occupational injuries. All injuries are recorded: incidental minor injuries and injuries with lost time. Injuries are recorded using a special document (injury report), which contains all the etiological data about the occupational injury. The analysis in this study includes injuries with lost time i.e. injury with sick leave (’injury with time burden’ - open injury list). In the period from 2002 to 2021, an average of 747 workers were employed in the iron ore mine. In the iron ore mine, the ore is mined in an open pit, and the ore preparation technology is magnetic-gravitational separation. For transport and exploitation, standard mining machines are used (conveyor belts, loading machines, rotary excavators, dumpers, bulldozers, etc.). An occupational safety service is organized in the mine, a risk assessment has been carried out at all workplaces and all workers have prescribed (standard) protective equipment. The occupational safety system in the mine organizes occupational safety tasks by performing the following activities: hazard identification and risk assessment, monitoring and measuring the effect of occupational safety, inspection and control, creation and management of occupational safety documentation, communication with workers (education and training) [19].

Etiological analysis of occupational injuries includes analysis of: severity of injury (light, severe, fatal, collective), place of injury (at the workplace, at another workplace, on the road, etc.), time of injury (day, month, season , work shift, etc.), characteristics of the worker who suffered an injury (gender, age, work qualification, injured body part, etc.), the source and cause of the injury [20]. For a more complete overview of the state of health and safety at work, in the production and business system, in addition to the absolute indicators of the number of injuries, relative indicators are also used for the analysis of changes in the number of injuries from the aspect of the trend over time, most often, the injury frequency index and the injury severity index. Relative indicators of injuries are calculated in relation to the number of employees or the number of effective working hours, such as the index (coefficient) of the frequency of occupational injuries and the index (coefficient) of the severity of occupational injuries as a ratio of lost working days and the number of injuries, i.e. number of effective working hours. Index relative indicators were also used for the analysis, which show the percentage change in the state of occurrence (in this case, the number of injuries at work) in successive time periods: base (in relation to the selected base period) and chain (in the current period in relation to the previous one). Anova analysis is used to analyze variance (variability of results) with the aim of determining the character and strength of the influence of one or more factors on the observed object or process (in this case, the number of occupational injuries). In this paper, one-factor Anova analysis was used to process etiological data on injuries done in the Excel software package. The graphical Pareto method or ABC diagram is used to analyze phenomena in a way to rank sizes/phenomena or errors and their causes in descending order. The theoretical basis of the Pareto principle is based on the claim that most of the problems in a certain area (up to 80%) arise from the 20% most significant causes. This method is used with the aim of identifying dominant causes in order to undertake corrective activities to eliminate them. The Solidworks software package was used to create a 3D model of the graphical interpretation of the analysis of injuries in the mine according to the etiological factor of the injured part of the worker's body.

RESULTS AND DISCUSSION

Monitoring of the number of injuries in the iron ore mine in the period from 2002 to 2021 is shown by the values of absolute and relative index indicators in Table 1. In the observed period, the structure of injuries according to severity that occurred in the mine is: 81% minor, 16% serious and 3% fatal injuries. The highest number of injuries was in 2006 (19 injuries), and there were no injuries in three years: 2008, 2013 and 2014. The largest number of minor injuries occurred in 2006 (17 injuries), and the largest number of serious ones in 2009 (5 injuries). In that period, three fatal injuries occurred in 2002, 2004 and 2006. The annual average of the total number of injuries at work is 5.30, minor injuries 4.3, serious injuries 0.85 and fatal injuries 0.15. Table 1 shows an overview of the number of employees in the mine. Based on the review of base indices (Table 1), for the selected base period of 2009, we see that the largest percentage change (increase) in the number of injuries occurred in 2006 (increase in the number of injuries compared to the selected base from 5 injuries to 19 injuries in year). Also from the review of the chain indices (Table 1), we see that the biggest percentage change (increase) in the number of injuries occurred between 2017 and 2018 (increase in the number of injuries from 1 to 7 injuries per year).

Table 1. Relative index indicators in an iron ore mine
in the Republic of Srpska (B&H) for the period 2002-2021.

Year	Number of occupational injuries, average number of employees and effective working hours*						Index	
	Number of injuries				Number of employees	Effective working hours	Base	Chains
	M	S	F	Total**				
2002	10	0	1	11	845	-	220.00	-
2003	4	0	0	4	424	-	80.00	36.36
2004	11	0	1	12	570	-	240.00	300.00
2005	13	2	0	15	639	-	300.00	125.00
2006	17	1	1	19	717	1313675	380.00	126.67
2007	9	1	0	10	723	1301101	200.00	52.63
2008	0	0	0	0	731	1320590	-	0.00
2009	0	5	0	5	696	1171361	100.00	-
2010	2	1	0	3	705	1302555	60.00	60.00
2011	1	4	0	5	824	1541360	100,00	-
2012	0	1	0	1	843	1565904	20.00	20.00
2013	0	0	0	0	854	1569103	-	0.00
2014	0	0	0	0	854	1568943	-	0.00
2015	1	0	0	1	848	1555526	20.00	-
2016	1	0	0	1	813	1368091	20.00	100.00
2017	1	0	0	1	803	1364028	20.00	100.00
2018	6	1	0	7	788	1364480	140.00	700.00
2019	4	1	0	5	767	1345911	100.00	71.43
2020	2	0	0	2	743	1228206	40.00	40.00
2021	4	0	0	4	746	1317719	80.00	200.00
Ukupno	86	17	3	106	14933	22198553		
Annual average	4.3	0.85	0.15	5.30	746.65	1109927.65		

*Internal records of the mining company
**Total number of injuries with lost time (Minor, Serious, Fatal) in the iron ore mine: injuries at the workplace + injuries from/to the place of work.

Figure 1 shows the trend of minor and serious injuries at work in the mine for the period 2002-2021. A decreasing trend of light and serious injuries is noticeable. The trend of minor injuries is decreasing at an annual rate of 10.35%, and the trend of serious injuries is decreasing at an annual rate of 4.33%. Figure 1 shows that despite the declining trend, the end of the observed period is characterized by an increase in the number of injuries. The applied occupational safety measures that have given results in a certain period should be reviewed in order to increase the efficiency of their effect.

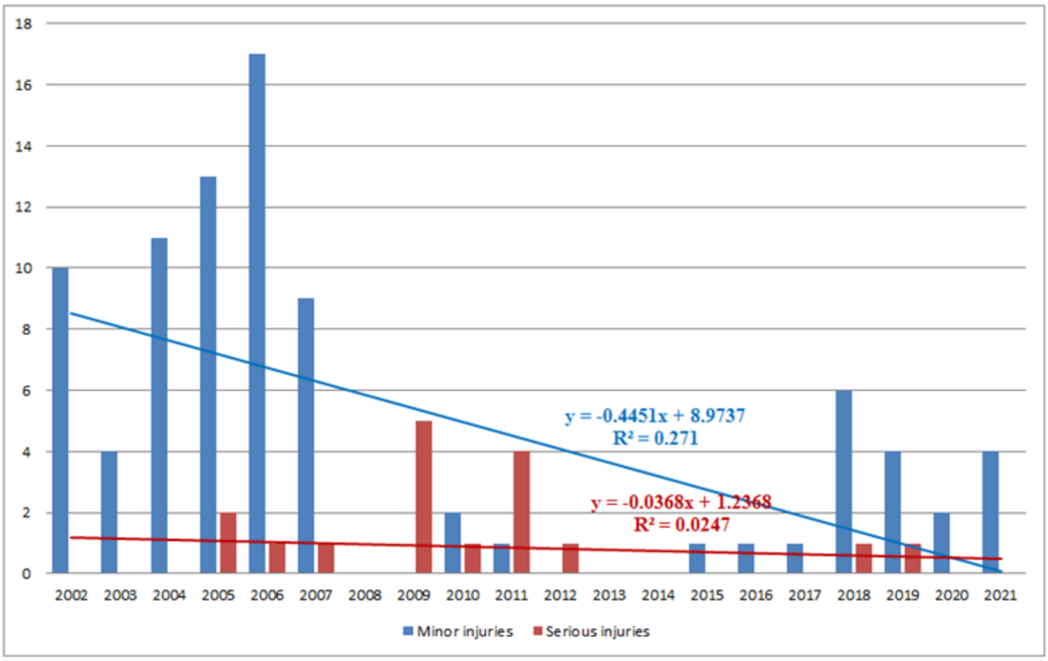

Figure 1. Trend of minor and serious *occupational* injuries in an iron ore mine i
n the Republic of Srpska (B&H) for the period 2002-2021.

From the analysis of injuries in the iron ore mine in the RS for the period 2002-2021. we conclude: that the male workforce is most injured (90%), that workers aged 41 to 50 are most injured (47%), that injuries occur at the workplace (42%), according to the level of professional education, most are injured workers with a secondary education and qualified workers (89%), workers with 21 to 30 years of experience (41%), in the 1st shift (59%), in the 3rd-4th hour after starting work (32 %), in April and May (11%) and on Mondays and Tuesdays (19%).

The result of the analysis by gender is expected, because the mining industry employs a predominantly male workforce (the percentage of employed women in mining companies does not exceed 10%, at the world level). Also, this industry has the highest percentage of employed labor force with secondary vocational education and qualified workers, so the highest percentage of injured workers is among the workers with these qualifications. Figure 2 shows the injury frequency index of workers with secondary education and qualified workers employed in the mine. Figure 2 shows that in certain years (2006, 2009, 2010, 2012, 2015, 2016, 2017, 2019, 2020 and 2021) that index is higher than the injury frequency index of all employed workers in the mine.

In relation to the work shift, there are the most injuries in the 1st shift, because the organization of work determines the number of workers engaged in other shifts because the exploitation of ore is carried out during the daylight. During the shift, injuries are most common after 3 to 4 hours of work, when workers are already tired before the daily break. According to the day of the week, there are more injuries at the beginning of the working week, due to 'getting in' into the working rhythm. The analysis of injuries according to the place of occurrence includes 25% of occupational injuries that occurred on the way from/to the place of work, because they are also treated as occupational injuries according to national legislation, which is in line with European standards. The percentage of 18% of injuries that occurred at another workplace indicates the practice of assigning workers to other workplaces before getting to know the dangers specific to that workplace. The highest number of occupational injuries, 90%, occurs in the production and technical sector, which is expected because in that sector work operations of mining, preparation and transport of iron ore are carried out, while the other sectors support the primary production sector.

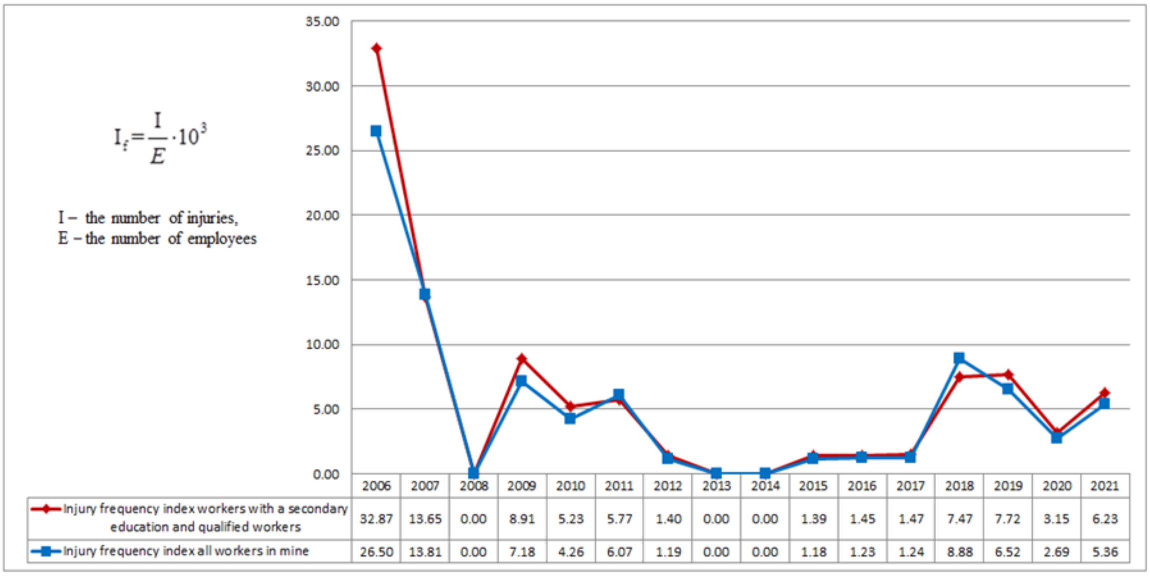

Figure 2. Index of the frequency of occupational injuries of workers with a secondary vocational education and qualified workers in an iron ore mine in the Republic of Srpska (B&H) for the period 2006-2021.

ANOVA ANALYSIS OF ETIOLOGICAL FACTORS INJURIES

One-factor ANOVA analysis showed which analyzed etiological factors influence the number of occupational injuries in the analyzed company, namely: age of the worker, injured body part and work shift.

Workers are divided into three groups by age (group 1: 40 or less; group 2: from 41 to 50; group 3: over 50). A statistically significant difference was found at the $p < 0.05$ level in the results of the three age groups: $F(2, 34) = 3.59$, $p = 0.04$. The size of the difference between the groups is large, and expressed using eta square, it is 0.17. The results of Tukey's HSD show that the mean of group 1 ($M = 1.86$, $SD = 0.95$) is significantly different from the mean of group 2 ($M = 3.85$, $SD = 2.76$) and not different from the mean of group 3 ($M = 3$, $SD = 1.63$). The mean value of group 2 does not differ significantly from the mean value of group 3.

Injuries according to the injured part of the body are divided into three groups (group 1: head and torso; group 2: extremities; group 3: multiple injuries). A statistically significant difference at the $p < 0.05$ level was found in the results of the three groups: $F(2, 34) = 6.03$, $p = 0.006$. The size of the difference between the groups is large, and expressed using eta square, it is 0.26. The results of Tukey's HSD show that the mean of group 1 ($M = 2$, $SD = 1.09$) is significantly different from the mean of group 2 ($M = 4.57$, $SD = 3.8$) and not different from the mean of group 3 ($M = 1.42$, $SD = 0.9$). The mean value of group 2 differs significantly from the mean value of group 3.

According to the shift in which the injury occurred, the division was made into three groups (group 1: I shift; group 2: II and III shifts; group 3: before/after work). There was a statistically significant difference at the $p < 0.05$ level in the results of the three groups: $F(2, 30) = 4.83$, $p = 0.015$. The size of the difference between the groups is large, and expressed using eta square, it is 0.24. Tukey's HSD results show that the mean of group 1 ($M = 5.25$, $SD = 4.35$) is significantly different from the mean of group 2 (M = 2.1, SD = 1.37) and mean of group 3 (M = 2, SD = 1.34). The mean value of group 2 does not differ significantly from the mean value of group 3.

Considering the established influence of certain groups of etiological factors on the number of occupational injuries in this mining company, the direction of the effect of safety measures at work that is proposed:

- programs for training workers for safe and healthy work, as well as programs for improving the safety culture, should be aimed at categories where workers are most often injured, as well as at the younger categories of workers,
- through organizational measures: schedule of shifts, change in the number of workers in shifts, more short breaks during work, etc., as well as educational measures: the importance of quality use of free time, the importance of preparing for a work shift, etc. react to the impact of shift work on the number of occupational injuries,
- additional educational programs on personal protective equipment.

THE ANALYSIS OF OCCUPATIONAL INJURIES, ACCORDING TO THE INJURED PART OF THE BODY

The analysis of occupational injuries, according to the injured part of the body, is primarily important for determining the health status of the worker, the course of treatment and the possibility of restoring work ability [21]. For the occupational safety system in an organization, this analysis should primarily enable the proper selection of the most adequate means of personal protection in the design of occupational safety measures. The classification of injuries according to this criterion also enables the detection of defects in work procedures or in the construction of machines and devices with which work processes are carried out.

According to the analysis of injuries in 71 surface mines in Turkey, injuries affected three parts of the body (mainbody, head and hand) in 79% of all injuries [17]. The structure of injuries according to the injured part of the body, in the statistical reports of the analysis of accidents and injuries in iron mines in Australia, is that arm injuries are 42%, legs 23% and torso 15% of the total number of injuries for the period 2019-2020, i.e. hands 39%, legs 25% and trunk 22% of the total number of injuries for the period 2020-2021 [22]. Previous investigations of safety and health at work in the mines of the Republic of Srpska for the period 2005-2009. gave the following results: the most frequently injured parts of the body were the extremities: arms and legs 64%, head 16%, eyes 5%, back 3%, and 9% had multiple injuries [7].

The analysis of injuries in the iron ore mine in the RS according to the injured part of the worker's body showed that the extremities of the legs 43% and hands 19% were injured the most, and there was also a significant percentage of multiple injuries 16% (the analysis did not include the three fatal injuries that occurred in the period 2002-2021.). Figure 3 presents an innovative graphic representation (3D model) of injury analysis according to the injured part of the body in an iron ore mine in the RS in relation to the graphic model of the body structure of an average person.

The percentage share of body parts of an average person with body units (H = head, N = neck, A = arms, T = torso and L = legs) [23] is represented by the model (figure 4a), while in figure 4b. graphical model of the percentage share of injuries to certain parts of the workers' bodies according to the analysis in the iron ore mine for the period (from 2002 to 2021). This graphic illustration does not include multiple injuries.

Based on the analysis of injuries according to the injured part of the worker's body, we conclude that when choosing personal protective equipment, special attention should be paid to personal protective equipment for the protection of legs and hands. Also, a significant percentage of multiple injuries points to the need for increased control of the application of work safety procedures and activities to build a work safety culture with an emphasis on measures to develop concentration and attention at work.

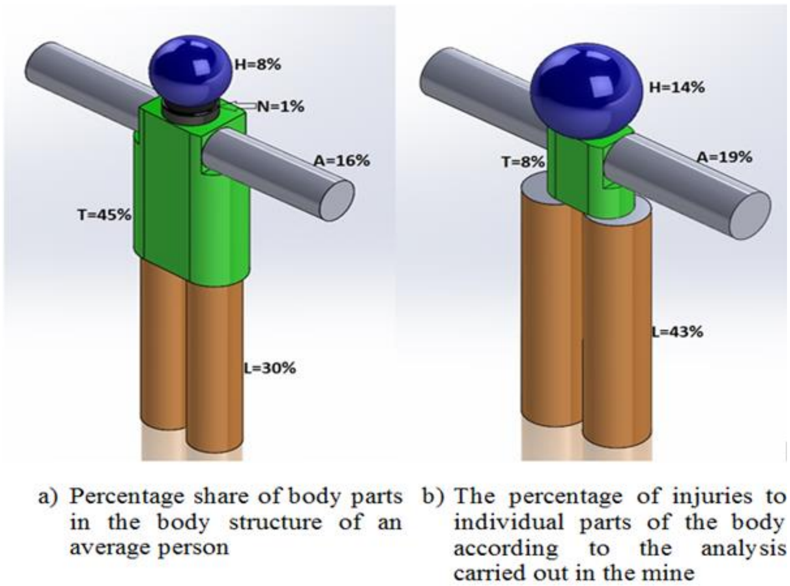

Figure 3. Analysis of occupational injuries according to the injured part of the body in an iron ore mine in the Republic of Srpska for the period 2002-2021

THE ANALYSIS OF SOURCE AND CAUSE OF OCCUPATIONAL INJURIES

Determining the source and cause of occupational injuries, from the group of etiological factors, is the most important for the design of expedient occupational safety measures. The sources of injury are related to material (means of work, materials, tools, etc.), energy and workplace factors that cause injury by their direct effect on the worker's body, the causes of injuries are related to the reasons, i.e. 'roots' of injuries and it is much more difficult to determine them, as well as to divide them into certain groups, due to the often present complex action of several causes [24]. The most common causes of occupational occupational injuries are:

- subjective causes dominated by the so-called human factor,
- objective causes related to the degree and duration of objective danger/harm in the working environment and/or at the workplace or to factors arising in the social environment [25].

According to the study [26], numerous factors that determine the occurrence of occupational injuries in mines are systematized into two categories:

- Personal variables: demographics, negative personality, safe work behaviour, job dissatisfaction, job stress,
- Sociotechnical variables: social support, safety environment, work hazards.

The results of the analysis of sources of injury for the researched mining company are shown in Figure 4. In the analysis, the most common sources of injuries were determined, namely: means, equipment and tools for work in 26% of cases and characteristics of the workplace in 24% of cases. However, in 36% of cases, the actual source of the injury was not determined (the source of the injury was not specified in the report on the occupational injury).

Occupational health and safety measures should be focused both on the improvement of work equipment (modernization of work equipment and tools, planned preventive maintenance of work equipment, etc.), and on the improvement of the ergonomic characteristics of workplaces.

Pareto analysis of the causes of occupational injuries and identification of the main one with the aim of directing the preventive effect of occupational safety measures is shown in Figure 5. Theoretical sources and research data of other authors state that the most common cause of occupational injuries is man, i.e. a certain procedure, action or behavior that a person performs in the working environment, e.g. carelessness, non-observance of safety rules, underestimation of danger, non-use or improper use of personal protective equipment, etc. ’Human factor’ was a crucial factor in around 90% of occupational injuries [27].

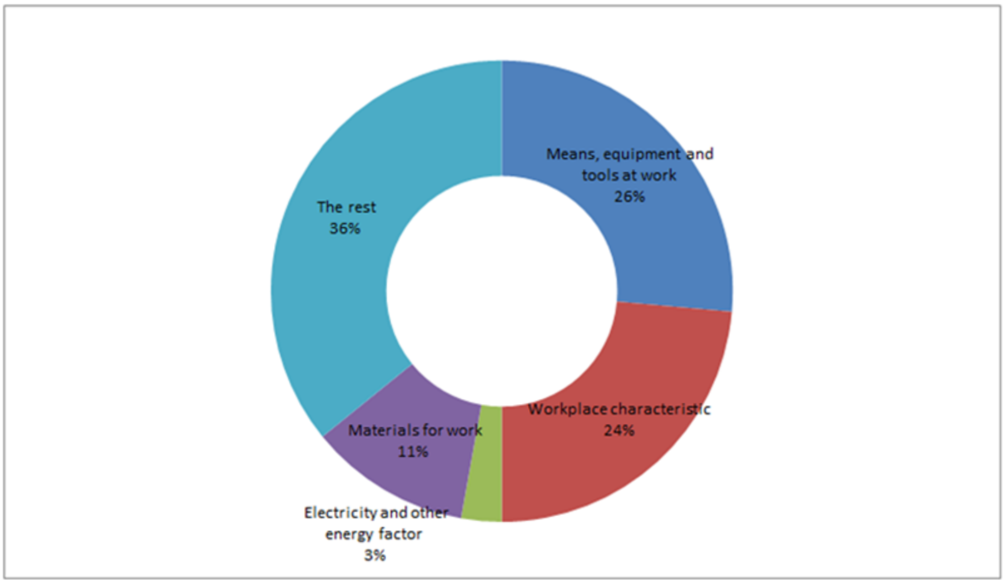

Figure 4. Sources of occupational injuries in an iron ore mine in the Republic of Srpska for the period 2002-2021.

An analysis of 295 mining accidents in Iran showed the importance of human and organizational factors in mining accidents: skill-based errors, routine violations, environmental factors and planned inappropriate execution of work operations were identified as the most common causes [28]. Research results [29] showed that although human error was the cause of most mining accidents, other causal factors such as the existing conditions (physical and technical environment), uncertain leadership climate and organizational factors also had an impact. The cause of worker injuries in mine, which is the subject of research, is the worker's inadequate actions in 44% of cases, so the activities of the occupational safety system should be focused on defining new forms of training and additional education of workers for safe and healthy work with an emphasis on the importance and importance of observing occupational safety measures.

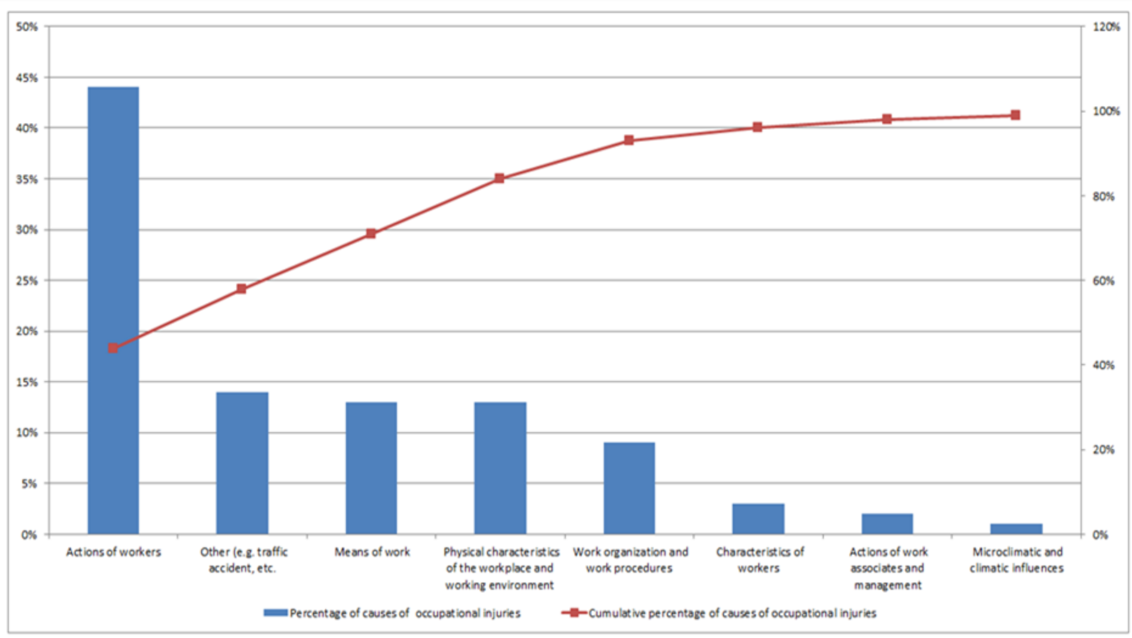

Figure 5. Pareto analysis of the causes of occupational injuries in an iron ore mine in the Republic of Srpska for the period 2002-2021.

The analysis of the causes of fatal injuries that occurred in this company showed that two fatal injuries were the result of a traffic accident (one on the way from home to the place of work, and one on a business trip). The cause of one fatal injury was faulty work equipment (pumping plant for tailings desilting). From the Pareto diagram of causes, it can be seen that other factors, among them traffic accidents, are the second most significant causal factor of all work injuries that occurred in the mine.

RELATIVE INDICATORS OF INJURIES FOR THE PERIOD 2006-2021.

Figure 6 shows the trend of the frequency index and severity index of occupational injuries for the period 2006-2021. in an iron ore mine. The monitoring of these indices in the occupational safety system in the mine has been applied since 2006, and the picture shows that there is a downward trend in these indices. The trend of the injury frequency index (If) is decreasing at an annual rate of 11.12%, and the trend of the injury severity index (Is) is decreasing at an annual rate of 5.7%. The indices were calculated in relation to the effective working hours (table 1.), according to the forms [20,30,31]:

$$I_f = \frac{I}{H} \cdot 10^6 \quad (1)$$

$$I_s = \frac{LW_d}{H} \cdot 10^3 \quad (2)$$

where is:

I the number of injuries

H the number of effective working hours (product of the number of employees and the number realized working hours)

LW_d the number of lost working days due to injuries

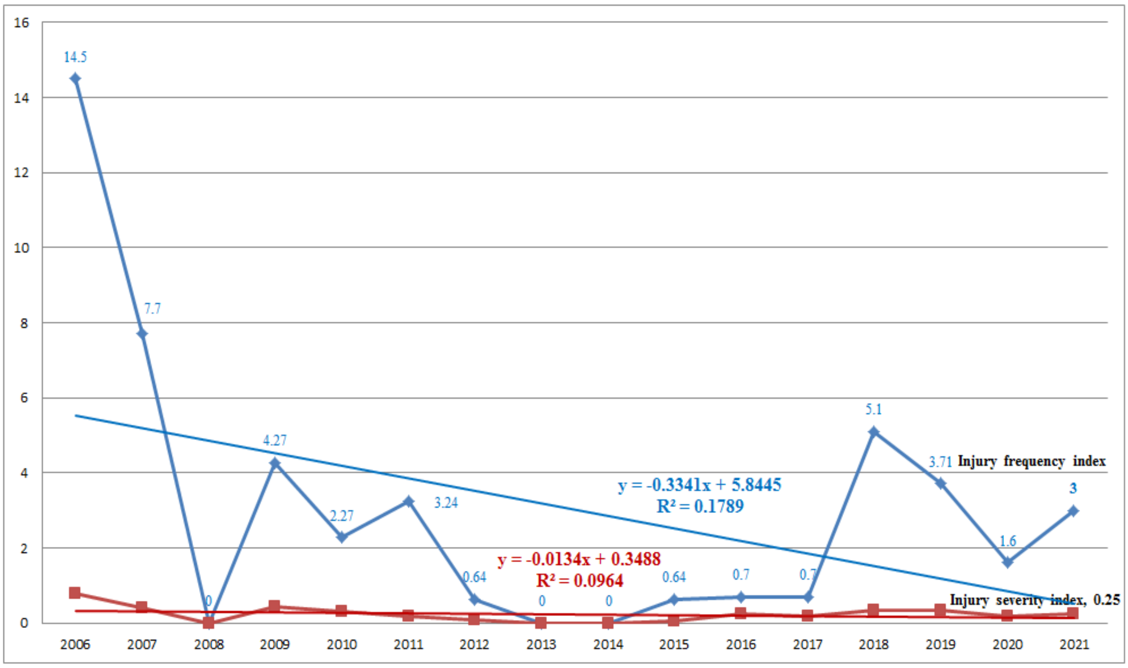

Figure 6. Trend of frequency index and severity index of occupational injuries in an iron ore mine in BiH (Republic of Srpska) for the period 2006-2021.

CONCLUSION

The modern approach to the management of the occupational safety system, within a certain activity, including mining, by analyzing injuries in the previous period and monitoring indicators is a simple and effective model for viewing and analyzing the current state of protection in the production system, with the possibility of tentatively predicting the situation in the future by designing prevention measures and monitoring the trend of indicators. The categories of etiological factors that have an impact on the number of occupational injuries in a certain period (ANOVA analysis), as well as the determination of the dominant causes (Pareto analysis) of injuries, allow for directing the effect of occupational safety measures (organizational, educational, etc.) on those factors, ie. causes and correction of the state of occupational health and safety in the following period. In today's conditions of efforts to ensure socially responsible business, taking care of the safety and health of employees is high on the list of priorities. Employees who feel safe at their place of work and have motivating working conditions realize a greater work ethic, and thus provide their company with a better business result. The implementation of preventive safety measures at work and the implementation of employee training in the company reduce the possibility of injuries, and thus the quality of working conditions is raised to a higher level.

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CONSTRUCTION

Risks in civil engineering projects,
The influence of liquefaction in construction facilities

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IDENTIFICATION AND ANALYSIS OF RISKS IN CIVIL ENGINEERING PROJECTS

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RESUME

The main and primary goal of the research in this paper is to analyze the actual current situation in domestic civil engineering, by determining the causes of missing deadlines and exceeding budgets of civil engineering projects. Since the results of this research also indicate the most likely problems that could arise during the construction of a project, the findings and conclusions could be used in risk assessment and drawing up dynamic plans for actual projects, which would improve the situation in domestic civil engineering and increase productivity of this important industry.

The results obtained in this paper provide a modern approach to the problem of risk in construction, provide a realistic insight into risk factors on civil engineering projects in the Republic of Serbia and the surrounding area, and facilitate the establishment of a better correlation between theory and practice in project management and risk management.

Key words: *risk management, project management, civil engineering projects*

INTRODUCTION

Due to the volume, value and socio-economic importance of building structures, as the final results of the production process, civil engineering represents one of the most important and specific economic activities and an extremely important factor in the development and preservation of the national economy. The more funds, experts, labor, materials, equipment and capital are provided from domestic sources, the higher and more pronounced the level of development of the local economy and its independence in relation to foreign aid and investments is. In addition, the growing tendencies of European integration include an increase in the number of international projects and cooperation with partners from a materially and technically more advanced and economically stable environment, with countries in transition expected to successfully keep pace with more developed partners and respond to business requirements imposed by modern international standards and conditions of international market competition.

Civil engineering projects are always unique, whereby, due to a number of various sources, the occurrence rate of undesirable situations increases [1,2]. They are intrinsically complex and dynamic and they include multiple feedbacks from different processes [3].

According to [4], risk is the cumulative effect of the uncertainty of phenomena that will negatively affect the objectives of the project. Risk and uncertainty characterize situations in which the actual outcome of a particular event or activity is likely to deviate from the estimated one [5]. According to

[6], risk is a measurable part of the uncertainty, for which we are able to assess the probability of occurrence and the magnitude of the damage. Risk is also defined as a deviation from the desired level. It can have a positive or, more often, an adverse effect on civil engineering projects [7,8]. In addition, exceeding costs and missing deadlines does not only affect the civil engineering industry, but also the overall economy and development of countries in general [9].

The previously mentioned concepts suggest that it is necessary to implement advanced project management techniques and methodological approaches that will provide the highest level of service, while achieving maximum profit and minimum unforeseen costs, when designing, planning and performing construction works. Successful management of a construction project, while observing the planned deadlines and budget, depends on the methodology that requires knowledge, experience and the so-called "engineering thinking", i.e. a practical and realistic approach to perceiving and solving problems.

In order to achieve this, it is necessary, among other things, to take a professional approach to risk management, which means anticipating possible causes of downtimes, delays and failures, in order to prevent or eliminate the causes of their occurrence, and develop a strategy to overcome significant negative consequences of their occurrence on the planned budget, deadlines, duration and unimpeded running of the project [10].

It is important to start using risk management from an early stage of the project, in which the main decisions are the way of coordination and the choice of construction methods [11]. On the other hand, the problems that accompany the realization increase with the size of the project, i.e. the uncertainty about the outcome of the project increases with the increase of its size [12,13]. The causes of missing deadlines and exceeding budgets, from the aspect of the project and company size, are discussed in papers [14,15,16,17,18]. The paper [14] presents an analysis of 806 large projects around the world, and it was concluded that the average cost overrun is 35.5%.

Risk analysis and management techniques have been described in detail by many authors [19,20,21,22,23,24]. In general, risk identification is the first and perhaps most important step in the risk management process, as it attempts to identify the source and type of risk [25]. This includes identifying potential conditions for a risk event in the construction project and clarifying risk responsibilities [26]. Risk identification develops the basis for the next steps, i.e. analysis and control of risk management. Identification actually improves and ensures the effectiveness of risk management. According to [27], project risk identification and mitigation are key steps in managing successful projects. In the paper [28], the identification of risks to the achievement of project objectives was investigated, while the identification of risks through the phases of project implementation is presented in the paper [10].

Qualitative and quantitative analyzes represent an important step in the risk management process [25]. Qualitative analysis is considered to be an evaluation process that includes a description of each risk and its impact or a subjective risk labeling (high / medium / low) in terms of the impact of the risk and the likelihood of its occurrence [10]. Quantitative risk analysis attempts to assess the frequency of risks and the magnitude of their consequences, using various methods such as decision tree analysis, cost risk analysis, and Monte Carlo simulations [29].

Uncertainties and risks involved in civil engineering projects cause price overruns, missing construction deadlines and lack of quality in project progress and completion [26,30,31]. A review of the literature reveals a wide range of types and sources of risk in civil engineering projects, as well as that different risk management methods and techniques can be included in construction project management in order to control potential risks. A review of the scientific and professional literature also points to the fact that many authors have dealt with risk assessment and their impact on missing the planned deadlines and budgets [14-16,32-46].

Missing deadlines and overrunning costs in construction projects knows no geographical boundaries and it is a global phenomenon. The research was conducted in various locations around the world [China [10], the Netherlands [14], Saudi Arabia [16,42], Great Britain [28], Nigeria [34,46], Sweden

[41], Turkey [44], Egypt [47], Poland [48,49], Lithuania [50], Korea [51], United Arab Emirates [52], Chile [53], Qatar [54], Australia [55,56], Serbia [57,58], and many others).

Unfortunately, in most civil engineering projects in the Republic of Serbia and the immediate surroundings, the phenomenon of missing the planned deadlines and overrunning the budget is evident and pronounced. This is especially pronounced in the projects where the criterion for the selection of contractors is exclusively the lowest offered price at the public auction, which is the most common practice in the construction of large public structures and when implementing the projects of national importance. As noted in [32], low construction costs are more a rule than an exception, where both the investor and the contractor have significant financial losses due to cost overruns.

The authors [34] indicate that poor risk management is one of the main factors of delay and conclude that the actions and inactions of participants in the construction process can contribute to the overall delay and budget overrun.

The paper [53] states that risk management is either not used or is not implemented effectively enough in many countries where civil engineering projects are implemented. One of the main reasons for this is the lack of experience, skills and knowledge about risk management.

Although advanced risk management methods are highly supported by mathematical tools [15,38,59-64] and contemporary computer tools [51,53,65], in some countries risk management in construction projects is still not efficient. For efficient and effective risk management, it is necessary to have an appropriate and systematic methodology and, more importantly, knowledge and experience from various types of projects that have been previously implemented [46].

Increasing efficiency in terms of time, material and human resources requires careful planning and comprehensive analysis, which includes the analysis of potential risks, as well as a strategy for overcoming them. Identifying the most common and important causes of delays and budget overruns would ensure, if not a complete avoidance of such situations, then at least their timely prediction in order to avoid an adverse surprise factor and to mitigate their consequences.

The research conducted and presented in this paper was aimed at determining as many causes as possible of budget overruns and missing deadlines on actual civil engineering projects, which took place in the period 2000-2016 (in the Republic of Serbia and neighboring countries), with special reference to subjective attitudes from the point of view of investors, contractors and supervisory authorities. The research was conducted through a survey of eighty engineers, and the selection of participants was done so that all target groups in terms of experience, territorial affiliation, types of projects and other aspects that could have a subjective impact on their responses and attitudes expressed in survey are addressed. Sample formation, data collection and their statistical processing were performed in accordance with the rules of scientific research.

Statistical data processing should confirm whether the applied approach was correct and whether the obtained results give a realistic picture of the situation from different points of view, primarily from the point of view of contractors, investors and supervisors.

MATERIAL AND METHODS

In order to gain as objective appraisal of the real situation as possible, the research was conducted in the form of a survey designed and conducted in accordance with the rules and requirements of scientific research, where questions are formulated based on theoretical and practical knowledge of the author, in collaboration with civil engineers who have experience in carrying out construction projects. The questionnaires were distributed to practical engineers, selected to form a sample that demographically accurately reflects the population, i.e. to comprise members of all age/experience groups from Belgrade, Sarajevo, Zagreb and the interior, from different companies and with different experience.

The primary classification differed between construction engineers, supervisors and investor representatives, as these three groups are directly involved in the execution of construction projects, and by the nature of their work they have different and very often opposing opinions. Consideration and comparison of different points of view enabled the acquisition of a realistic insight into the causes for missing deadlines and overrunning budgets of construction projects.

The standard methodology of scientific research was implemented in the research, which includes a quantitative and qualitative approach. The quantitative approach included the formation of a sample, data collection through a survey and their adequate processing, while the qualitative approach included the analysis of the obtained results, the establishment of cause-and-effect relationships and their interpretation and explanation.

The formation of the sample, the choice of the form of the questionnaire, the compilation of questions and the choice of the method for the qualitative assessment of the research results were conducted in consultation with experts from the agency "Source" from Belgrade, which surveys public opinion and markets.

The research was conducted in six carefully prepared and conducted phases:

1. Defining of problems and goals of research and formation of the research plan;
2. Making of survey questionnaires based on the consultation with practicing engineers;
3. Formation of samples, i.e. choice of respondents from all observed categories;
4. Conducting of the survey;
5. Statistical processing of the collected data;
6. Conclusions, recommendations and potential for further research.

Survey questionnaires were formed in accordance with the rules for this type of research, whereby the questionnaires were preliminarily formulated based on experience in project management and construction technology, and the final form was obtained after consultation with five civil engineers, who have experience in construction projects.

For more thorough and accurate data processing, the factors that may affect the missing deadlines and budget overrun are divided into the six following groups:

1. General factors;
2. Contractor responsibility;
3. Supervision responsibility;
4. Investor responsibility;
5. Material;
6. Design and design documents.

Since the subject of the research is the analysis of personal attitudes of the respondents, the ordinal (numbered) Likert scale was adopted as an appropriate measuring device.

It was planned that ninety engineers would participate in the survey, thirty each from the territories of Bosnia and Herzegovina, Serbia and Croatia, selected so that all the above groups and classifications would be better and more evenly covered. Eighty of them returned the completed questionnaires (28 from Bosnia and Herzegovina, 27 from Serbia and 25 from Croatia). The analysis of the survey results showed that there are no differences in opinions that could be attributed to regional (state) affiliation, so that the sample can be viewed collectively, regardless of which country the respondent is from. It was also shown that the gender of the respondents (45 men and 35 women) was not relevant.

Of the eighty survey participants, forty participated in construction projects as contractors (50%), twenty-four as supervisors (30%), and sixteen as investor representatives (20%).

Twenty-four of them had less than three years of experience (30%), twelve between three and five years of experience (15%), twenty between five and ten years of experience (25%), and twenty-four more than ten years of experience (30%).

Of the eighty survey participants, thirty-two were from capitals (40%), twenty from cities with more than 100,000 inhabitants (25%), sixteen from cities with 50–100,000 inhabitants (20%), and twelve from cities with less than 50,000 inhabitants (15%).

Twenty-four (30%) participated in projects with a total annual value of over € 3,000,000, thirty-two (40%) on projects worth € 1–3,000,000, and twenty-four (30%) on projects worth less than € 1,000,000 €.

Of the eighty survey participants, twenty (25%) were from small, twenty-eight (35%) from medium, and thirty-two (40%) from large firms.

The ranking of results by importance was performed based on the value of the importance index (Iv), which is calculated according to the following expression [66]:

$$I_v = \Sigma^5 i=_1 a_i x_i \qquad (1)$$

where:

- Iv - is importance index;
- a_i = 1,2,3,4 or 5 - numerical response value;
- x_i - response frequency a_i, expressed as the respondent percentage that selected this response in relation to the total number of respondents.

Expressing the results through the index of importance is especially convenient and important when the population, i.e. the sample as its representative part, can be divided into subgroups in several different ways. In such cases, depending on the division into subgroups of respondents observed, the ranking of the importance of the impact may differ significantly.

For example, the attitudes of respondents younger than twenty years will differ significantly from the attitudes of respondents older than fifty, but also the attitudes and priorities will differ significantly within each age group separately, i.e. if each age group is divided according to some other criterion, e.g. education, income, education, etc. Full insight into the attitude of the population can be gained only if the problem is viewed from as many angles as possible, i.e. through several different divisions of the population into target groups.

Another advantage of ranking different factors according to the index of importance, lies in the fact that in this way not only the comparison of factors is performed, but also the opinion of a group of respondents on the importance of each factor is obtained. For example, if none of the top ten factors on the ranking list has an importance index greater than 50%, it means that no factor is actually "overly" important. Also, if the first-ranked factor has an importance index in the range of 60-70%, it means that it is the most influential in a given set or for a given group of respondents, but that it does not actually have a primary impact on the subject of research.

On the other hand, if the first five factors from the list have factors of importance greater than 90%, it means that they are all extremely important and that the fifth-placed is no less influential than the first-placed

RESULTS AND DISCUSSION

Factors affecting missing the deadlines

Table 1 shows a consolidated ranking list of respondents' attitudes about the importance of certain factors that affect the missed deadlines in construction projects. Table 1 (figures 1, 2 and 3) gives a ranking list in relation to the attitudes of contractors, investors and supervisors.

The table shows that the attitudes of contractors and investors are similar when it comes to the most influential factors and that the first five places in their rankings are almost identical, both in order and in indices of importance. The attitudes of supervision are somewhat different, but the first three of the first five places on the ranking list are repeated, so it can be said that there is uniformity in attitudes.

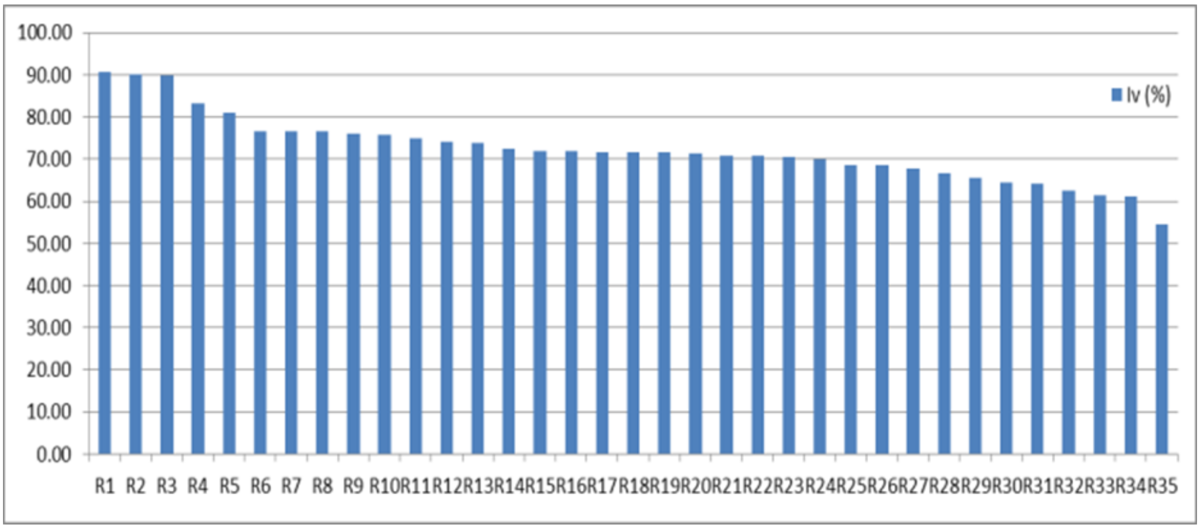

Figure 1. Index of importance of factors affecting missing deadlines - contractor opinion

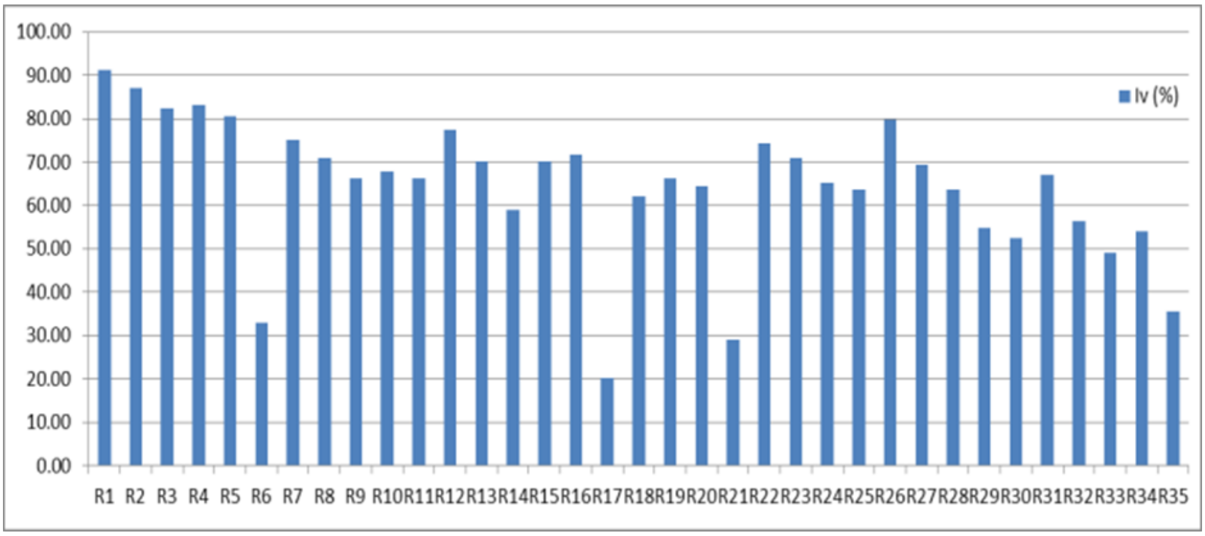

Figure 2. Index of importance of factors affecting missing deadlines - investor opinion

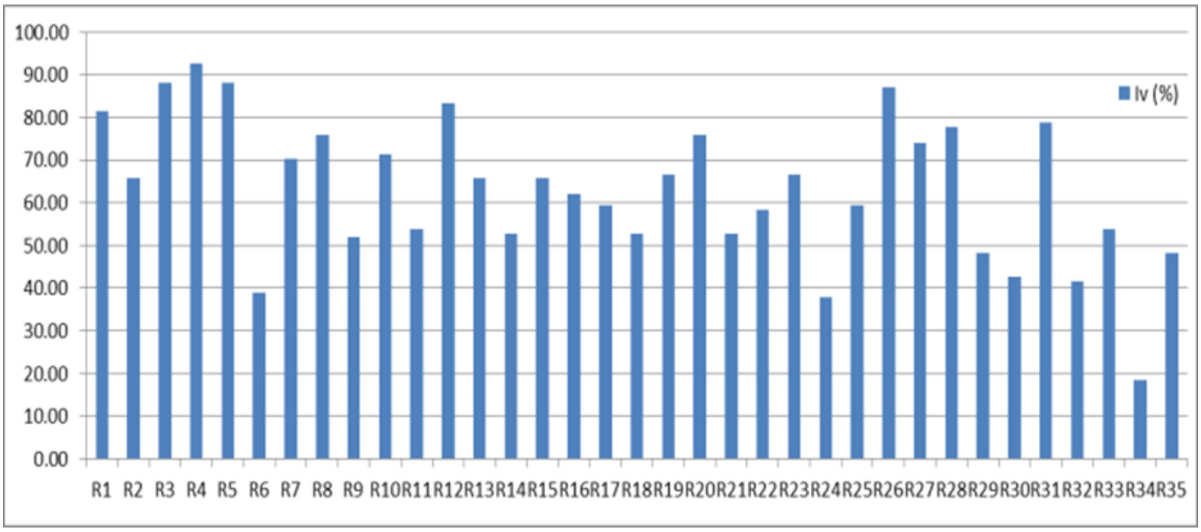

Figure 3. Index of importance of factors affecting missing deadlines - supervision opinion

Table 1. Summed comparative view of the results

No	Factor	Contractors		Investors		Supervision	
		I_v (%)	Rank	I_v (%)	Rank	I_v (%)	Rank
1.	Shortage of material on the market	90.53	1	91.13	1	81.48	6
2.	Shortage of material on the construction site	90.15	2	87.10	2	65.74	16
3.	Delays in material delivery	89.77	3	82.26	4	87.96	2
4.	Financial problems during construction	83.08	4	83.06	3	92.59	1
5.	Poor construction site management	81.06	5	80.65	5	87.96	3
6.	Delay of due payment	76.52	6	33.06	33	38.89	33
7.	Inadequate material quality	76.52	7	75.00	8	70.37	13
8.	Lack or breakdown of machinery and equipment	76.52	8	70.97	12	75.93	10
9.	Sluggishness in issuing permits	76.15	9	66.13	18	51.85	28
10.	Subsequent design alterations	75.76	10	67.74	16	71.30	12
11.	Poor assessment of time and material resources	75.00	11	66.13	19	53.70	24
12.	Self-initiated interruption of works as requested by contractors or investors	73.97	12	77.42	7	83.33	5
13.	Inadequate handling of material on the site	73.86	13	70.16	13	65.74	17
14.	Insufficient training level of workers	72.35	14	58.87	26	52.78	27
15.	Lack of knowledge or lack of implementation of project management	71.97	15	70.16	14	65.74	18
16.	Slowness in solving problems and making decisions at the project level	71.97	16	71.77	10	62.04	19
17.	Unrealistic investor requirements (deadlines)	71.59	17	20.16	35	59.26	20
18.	Incomplete drawings	71.59	18	62.10	25	52.78	26
19.	Poor and/or slow communication between the project participants	71.48	19	66.13	20	66.67	14
20.	Errors in execution of works	71.21	20	64.52	22	75.93	9
21.	Investor interfering in work	70.83	21	29.03	34	52.78	25
22.	Construction site manager inexperience	70.83	22	74.19	9	58.33	22
23.	Centralized decision making system in supervision	70.45	23	70.97	11	66.67	15
24.	Lack of experience and/or of expertise of supervision	70.08	24	65.32	21	37.96	34
25.	Incomplete documents and/or imprecise descriptions and specifications	68.56	25	63.71	24	59.26	21
26.	Insufficient number of workers	68.46	26	79.84	6	87.04	4
27.	Repairs and subsequent works due to poor quality and errors in construction	67.80	27	69.35	15	74.07	11
28.	Self-initiated withdrawal from the project	66.67	28	63.74	23	77.78	8
29.	Designer inexperience	65.53	29	54.84	28	48.15	30
30.	Design errors	64.39	30	52.42	30	42.59	31
31.	Stubbornness in case of a dispute between contractor, investor, and/or supervision.	64.02	31	66.94	17	78.70	7
32.	Insufficient number of supervision personnel	62.50	32	56.45	27	41.67	32
33.	Slowness in correcting errors in the design	61.36	33	49.19	31	53.70	23
34.	Human factor (personal conflicts and dislike, corruption, political games)	61.15	34	54.03	29	18.52	35
35.	Special requirements (high finish quality etc..)	54.55	35	35.48	32	48.15	29

Contractor opinion

Problems related to the material occupy the first three places in the ranking list of contractors' opinions, with a slight difference in the value of the importance index (value range less than 2%). This is followed by financial problems during construction (83.08%) and poor construction site management (81.06%). From sixth place downwards, the list continues with a relatively steady rate of decline in the importance index and ends with special investor requirements (54.55%), as the least important factor. It is interesting to note that the contractors in the evaluation were much less rigid than their counterparts from the other two groups (supervision and investor), as most factors received relatively moderate ratings of importance (55-76%), while only the first three (about 90) %) and the other two (81–83%) differ significantly. In addition, the contractors were more self-critical and objective than their counterparts, as they attached more importance to the factors for which they were responsible. The only significant difference in opinions, compared to the other two groups of respondents, can be seen in the item "Insufficient number of workers", which is only in the twenty-sixth place among contractors (68.16%), while among investors and supervisors it ranks high, in the sixth (79.84%) and fourth (87.04%) places, which testifies to the previously stated position that the problem should be viewed from different angles and that the analysis of only one standpoint would give a one-sided and inaccurate view of the problem.

Investor opinion

It is noticeable that there is a strong subjectivity in the attitudes of engineers who participated in construction projects as representatives of investors, since the factors directly or indirectly related to their responsibility received extremely low grades, i.e. the importance index less than 40%. However, it is noticeable that the representatives of supervision gave relatively low marks to the same factors, but that the position of investors and supervision almost coincides (very low index of importance) when it comes to delays in cash payments by investors, while contractors gave this factor a high sixth place and relatively high importance (76.52%). The most noticeable deviation can be seen in the last place of the list, where there are unrealistic demands of investors in terms of deadlines, which is a factor to which investors do not attach any importance (20.16%), while it is relatively high among contractors and supervisors, whereby contractors attach much greater importance to it (71.59%) than supervision (59.26%). The upper part of this ranking list is almost the same as that of the contractors, and the first five places are in the same range of importance index (80-91%). The difference in relation to the contractors is reflected in somewhat more rigid attitudes, which can be seen by the faster decline of the importance index. While with contractors only one factor has an index of importance of less than 55%, with investors there are as many as eight, four of which have almost no importance (less than 40%).

Supervision opinion

Engineers from this group had the most definite opinions, with the largest range of importance indexes (18.52-92.59%) and with as many as thirteen opinions that they declared little or not important for the project. As with the representatives of investors, there is a noticeable subjectivity and low self-criticism, as the representatives of the supervision assessed all the factors from their domain of responsibility as little important or almost unimportant. As the most important factor in mission the planned deadlines, supervising engineers pointed out the poor handling of finances by the contractor (92.59%), and the next three places are almost equally occupied by delays in material delivery and poor site management, with 87.96% and insufficient number of workers with an importance index of 87.04%, which are all factors in the contractor responsibility domain. It is also noticeable that in the first ten factors there is only one that is from the domain of force majeure, and that is the shortage of materials on the market (81.48%, sixth place), which is in the first place for both contractors and investors.

Factors affecting budget overrun

Table 2 (figures 4, 5 and 6) provides a comparative ranking of respondents' opinions on the factors influencing the budget overrun in construction projects. It can be noticed that in this case the opinions were more uniform both qualitatively and quantitatively, i.e. that the lists of contractors and investors are almost identical, while the list of supervision engineers is somewhat different, but the importance indices are approximately the same order of magnitude.

Table 2. Factors affecting budget overrun

No	Factor	Contractors		Investors		Supervision	
		I_v	Rank	I_v	Rank	I_v	Rank
1.	Works and procurement of materials and equipment delay (contractor responsibility)	83.71	1	83.87	2	95.37	1
2.	Changes in prices of materials and equipment	81.06	2	87.96	1	80.65	4
3.	Inflation (change of value of domestic currency in respect to euro)	78.79	3	77.42	3	88.89	2
4.	Alterations of the project during construction	76.92	4	75.81	4	69.44	8

	(investor responsibility)						
5.	Additional requirements (investor responsibility)	76.52	5	72.58	7	66.67	9
6.	Poor planning and control of financial resources investment dynamics	75.38	6	74.19	6	82.41	3
7.	Poor expenses record	72.73	7	75.00	5	75.93	6
8.	Poor organization structure at the level of project	71.59	8	62.90	8	75.00	7
9.	Incomplete design documents	67.42	9	57.26	10	57.41	10
10.	Errors and flaws in the bill of quantity	64.77	10	61.29	9	77.78	5

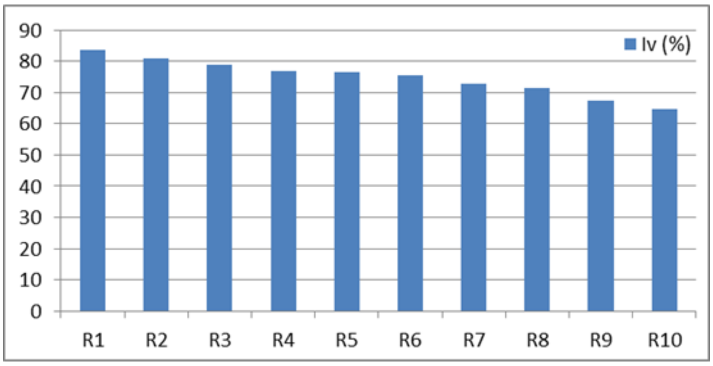

Figure 4. Index of importance of factors affecting budget overrun - contractor opinion

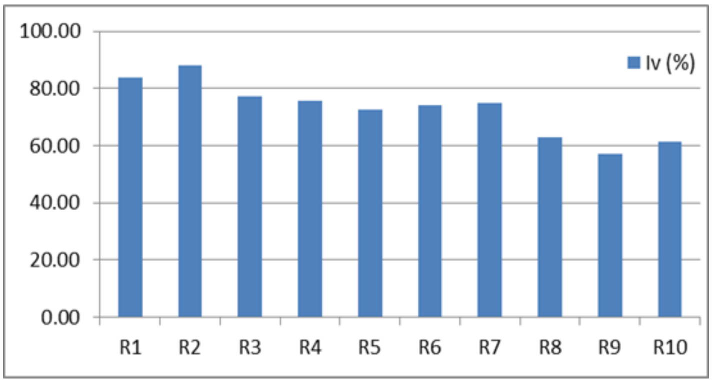

Figure 5. Index of importance of factors affecting budget overrun - investor opinion

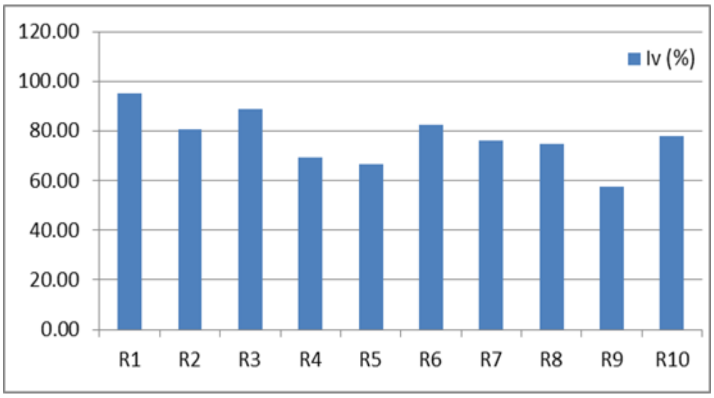

Figure 6. Index of importance of factors affecting budget overrun - supervision opinion

Contractor opinion

The contractors stressed the delay of works and procurement of materials and equipment and the change in the price of materials as the most important factors, with importance indices of 83.71 and 81.06%, followed by inflation (78.79%), project changes (76, 92%) and additional investor requirements (76.52%). As the least important factor, at the bottom of the list there are errors in the bill of quantities with an importance index of 64.77%.

Investor opinion

As already mentioned, the ranking of investors' opinions is almost identical to the ranking of contractors, with clearer divisions between important and unimportant factors, as evidenced by the fact that the last three items have a noticeably lower importance index than others (difference between seventh and eighth place is ten percent).

Supervision opinion

The supervising engineers had a very definite position on the main cause of the budget overrun, which can be seen by the fact that they almost unanimously assigned the extreme importance of 95.37% to the delay of the works. In second place is inflation (88.89%), and in third place, with an importance index of 82.41%, is poor planning and control of dynamics. Of all the categories and groups of factors, in the study presented in this paper, this is the group with the most pronounced differences in the values of the importance index, since the difference between the first and third place is as much as 13%.

The range in importance indices here is even larger than in the other two groups of respondents (almost 40%), which indicates a greater resolution and determination of engineers from this group. It is interesting that on the lists of contractors and investors, flaws and errors in the bill of quantities and estimate are positioned very low, while in the case of supervision engineers they are around the middle of the list, with a noticeably higher importance index.

CONCLUSION

Statistical data processing showed that the applied approach was correct and that the obtained results give an overview of the situation from different standpoints, primarily from the standpoint of contractors, investors and supervisors. Respondents within the same groups had relatively similar views, but the views of the groups differed, leading to the conclusion that this issue should not be generalized, but that factors of budget overruns and missingdeadlines should be viewed and assessed from different aspects.

Although the initial expectation was that the attitudes of contractors and investors would differ significantly, and that the opinions of supervisors would be somewhere between theirs, the results showed that the opinions of contractors and investors were, in most cases, relatively similar while the supervision opinions differed moderately to considerably. In addition, it is noticeable that contractors were more reserved in attitudes, as the range of importance index generally ranges between 65 and 85% and has a predominantly even distribution of values, while the range of values with investors and supervision respondents is broader and with larger peaks, which testifies to more assertive attitudes. Another fact that testifies to the greater moderation of the contractors as a group, is that they were much more self-critical than their colleagues from the other two groups. In contrast, representatives of investors and supervisors gave strikingly low ratings to factors related to their accountability. Such differences in attitudes and mindsets can also be attributed to a relatively small sample (twenty engineers in total), so a significantly larger sample would likely yield more balanced results with a more even distribution of importance indices within the same group of factors.

From the analysis of the causes of missing the deadlines, it is noticeable that the vast majority of respondents ascribed a primary importance to factors directly related to materials and equipment, whether it is force majeure (shortage of materials on the market) or the responsibility of the contractor and poor material handling). It is followed by the factors that are in the domain of responsibility of the contractor (poor management of the construction site, insufficient number of workers, etc.). The general opinion is that the factors related to the project documentation have the least impact on delays during construction. The importance of the shortage of materials on the market and on the construction site was especially pointed out by the respondents from smaller cities, which shows that this problem is more pronounced in less developed environments, where the supply of materials and equipment is lower both in qualitative and quantitative terms. It is also interesting to note that in all groups, younger survey participants, much more than older colleagues, insisted on the importance of knowing project management. Respondents' views on the reasons for overrunning the planed budget were much more uniform. The most important factors of budget overruns are delays in works, changes in material prices and changes in the design during construction. As with deadlines, the least importance is given to shortcomings in the project documentation.

The results of the presented research met the expectations, because they clearly indicate the most likely problems that could arise during the realization of a civil engineering project. The acquired knowledge and conclusions can be used in risk assessment and in drawing up dynamic plans for actual projects, which would improve the situation of civil engineering in the country and the region.

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STATE-OF-THE-ART APPLICATION OF THE LOG-PILING METHOD IN THE ROLE OF SHALLOW GROUND IMPROVEMENT FOR LIQUEFACTION MITIGATION

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SUMMARY

This research paper focuses on evaluating the log piling technique as a sustainable, cost-effective, and environmentally friendly solution for reducing soil liquefaction risks during earthquakes. Although this method has been used extensively in Japan, mainly aiming for complete soil layer penetration, its economic viability is questionable in cases requiring very deep soil improvements. The study highlights that shallow ground improvement can notably enhance the seismic behavior of the soil-improvement-structure system, as evidenced by the reduced total and penetration settlements caused by liquefaction. The paper presents a methodology for determining the optimal dimensions of the modified ground zone using both small and medium-scale 1-g shaking table tests.

The small-scale tests involve a detailed parametric study, examining variables like improvement width, pile spacing, and the depth-to-thickness ratio of the improved layer. Medium-scale tests, on the other hand, are geared towards identifying the minimum effective pile length. This approach provides a practical guideline for engineers to implement log piling for small residential buildings. Additionally, the paper utilizes finite element method (FEM) effective stress analysis, incorporating a PLAXIS 2D-based constitutive model (PM4Sand) calibrated with laboratory undrained cyclic torsional tests. This model accounts for the changes in effective stress during seismic activities. Finally, the study correlates its numerical findings with the results from the 1-g shaking table experiments, offering a well-rounded perspective on the effectiveness of log piling in mitigating liquefaction risks during seismic events

Key words: *soil liquefaction, log-pile, soil improvement, 1-g shaking table test, numerical study, constitutive model, effective stress analysis*

INTRODUCTION

In recent years, the escalating issue of global warming and climate change has risen to the forefront of societal concerns. One promising avenue for long-term carbon dioxide storage, the primary driver of global warming, involves the utilization of wood to establish carbon reservoirs, as depicted in Figure 1. In alignment with this ethos, ground improvement methodologies employing wood, which possesses resistance to biodegradation beneath the water table, have been conceived.

Conversely, the seismic event of the Great Tohoku Earthquake in 2011, particularly in Japan, induced widespread liquefaction occurrences across various regions, resulting in substantial damage to smaller-scale structures, notably single-family residences. Liquefaction mitigation techniques have not been widely applied to residential buildings, underscoring the importance of investigating cost-effective strategies for liquefaction mitigation. Shallow ground improvement approaches, with a specific emphasis on the upper soil layers harboring a relatively thick liquefiable stratum, have been deemed as suitable measures for smaller residential edifices [1].

Presented as an approach that concurrently addresses concerns regarding carbon stock and liquefaction mitigation, a method involving the static installation of timber piles for soil densification and reinforcement has been introduced as shown in Figure 2. This conceptual framework, developed in recent years, seeks to alleviate excess pore water pressure and mitigate structural settlement [2].

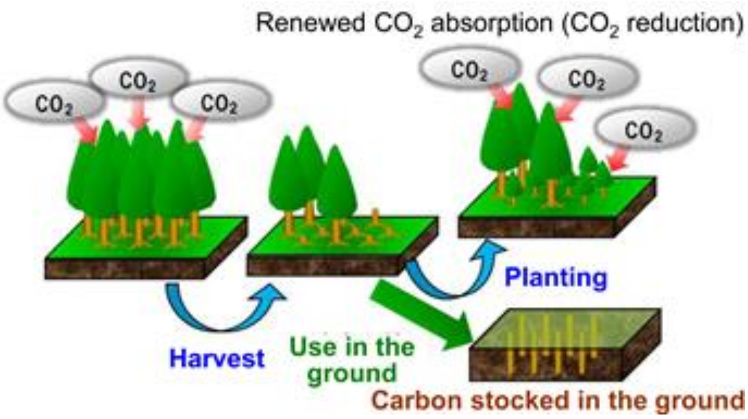


Figure 1. Effect of the log piling method on environment

LOG PILING FOR SHALLOW SOIL IMPROVEMENT IN LIQUEFACTION MITIGATION

Prior investigations into the utilization of log piles for liquefaction mitigation have already demonstrated their efficacy. In a 1-g shaking table experiment conducted by [3], it was emphasized that maintaining a center-to-center distance between log piles of 4 to 5 times their diameter yielded substantial liquefaction prevention effects, comparable to established densification methods. Further studies have also highlighted log piling's utility as a soil improvement technique near existing structures, showcasing its ability to reduce settlement and enhance the seismic resilience of buildings [4,5].

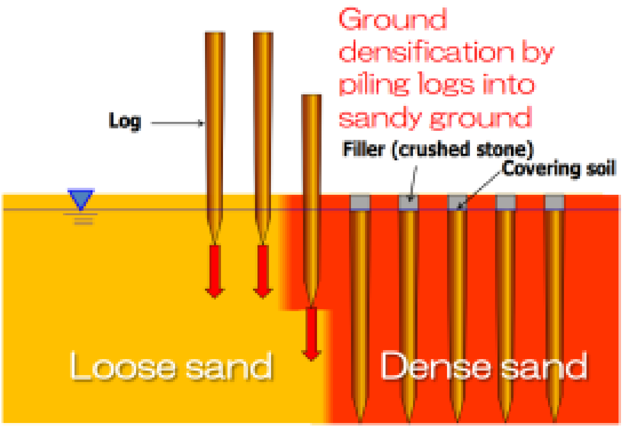


Figure 2. Installation of log piling by static pressing

Nevertheless, it is crucial to note that many previous studies assume the necessity of improving the entire liquefiable soil layer, which poses practical challenges when applying log piling as a liquefaction countermeasure for small residential structures. Additionally, numerous researchers have undertaken parametric studies based on 1-g shaking table model tests, investigating liquefaction mitigation through partial enhancement of soil conditions within the liquefiable layer. For instance, [6], utilized densification soil improvement techniques and explored parameters such as improvement width, structure width, and the ratio of width to height of the structure. Their findings indicate that when the ratio of improved layer thickness to improved width exceeds 0.4, the ratio of liquefiable layer thickness to settlement tends to stabilize at an economically feasible level.

Similarly, [7], conducted a parametric analysis, considering structural dimensions, soil relative density, and liquefiable layer thickness. They observed that when the ratio of liquefiable layer thickness to the width of the structure exceeded 1.0, the ratio of liquefiable layer thickness to settlement remained below 5%. Thus, through 1-g shaking table model tests, the relationship between liquefaction-induced settlement and overall structural dimensions is often elucidated using various geometric ratios within the "liquefiable soil – improved zone – superstructure" system. However, there is currently a paucity of research exploring the concept of log piling as a liquefaction mitigation method at shallow depth. Against this backdrop, this study focuses on shallow soil improvement, offering a pragmatic and cost-effective solution for single-family residences situated atop thick liquefiable layers. The primary objective is to assess the efficacy of liquefaction mitigation through static log pile installation, accomplished through a series of 1-g shaking table model tests featuring variable parameters. Initially, small-scale experiments are conducted, varying improvement width, center-to-center pile distance, and the ratio of improvement depth to liquefiable layer thickness. Subsequently, medium-scale tests build upon these findings, exploring the influence of the absolute length of log piles while maintaining a consistent ratio of improvement depth to liquefiable layer thickness.

The central aim of this study is to establish practical thresholds and offer guidance to practitioners, facilitating the selection of an optimal configuration within the "liquefiable soil – improved zone – superstructure" system. This enables the effective adoption of log piling as a shallow soil improvement technique for liquefaction mitigation in the context of small residential constructions.

SOIL CONDITIONS IN EXPERIMENTS

In the presented study, Silica sand No. 7 has been chosen as the soil type due to its low resistance in terms of liquefaction potential. Key physical properties of this material, including particle density and maximum/minimum void ratio, have been determined through laboratory tests and are as follows: ρs=2.66 g/cm^3, emin=0.705 and emax=1.178.

For the 1-g shaking table model tests, ground conditions with a relative density ranging from 40% to 50% (medium dense soil) have been selected.

SMALL-SCALE 1-G SHAKING TABLE TESTS

A small-scale rigid soil box, measuring 77.5 cm in width, 28 cm in depth, and 40 cm in height, has been employed for the experimental tests. To ensure appropriate scaling, the model has been downsized to 1/20th of its original dimensions. To account for the bending characteristics of the logs, we have implemented the similarity law introduced by [8]. For this study, PVC logs (sticks) were used instead of wooden ones. The house (superstructure) model's dimensions are 15 cm in width, 15 cm in depth, and 7.65 cm in height, with a base stress of 0.75 kPa. This base stress value, equivalent to 15 kPa in the prototype, realistically represents the scaled-down base stress encountered by small residential houses.

The primary objective of these experiments is to investigate the influence of three key parameters on liquefaction mitigation: 1) PVC log length (improved depth). 2) Improvement width. 3) Center-to-center spacing of the log piles. To streamline the testing process, the following approach was adopted: 1) A target relative density of 45% was set for the model soil. 2) Sand was initially introduced into a water-filled tank, and minor vibrations were applied to achieve the desired density up to a height of 30 cm. Subsequently, PVC logs were driven into the soil following a specific sequence for testing purposes. In subsequent trials, sand was not removed after the initial setup. Instead, high-pressure water was injected through valves to create loose ground conditions within the model. The water level was adjusted to match the ground level, and PVC logs were inserted for further experiments.

The experimental setup included various sensors, such as two laser displacement transducers positioned atop the superstructure model and an accelerometer mounted on the bottom surface of the shaking table. In Figure 3, Test 14 is depicted, Figure 4 shows an overview of the complete experimental set-up, while Figure 5 illustrates the initial state of the model and presents the input motion, involving 20 cycles at 5 Hz, with a gradual increase in excitation levels from 50 gal to 650 gal. Precise input accelerations were recorded using accelerometers strategically placed on the shaking table.

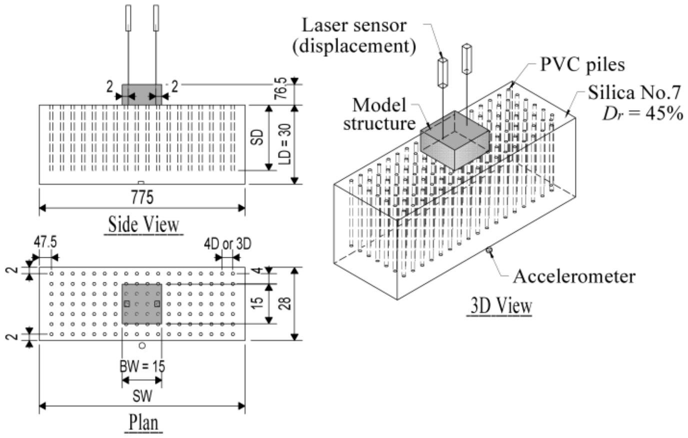


Figure 3. Side view, plan, and 3D view of the model (Test 14)

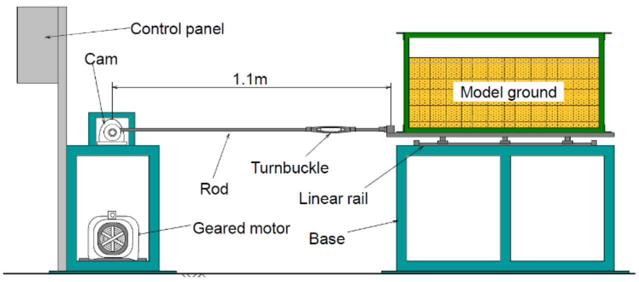


Figure 4. Small-scale 1-g shaking table set-up

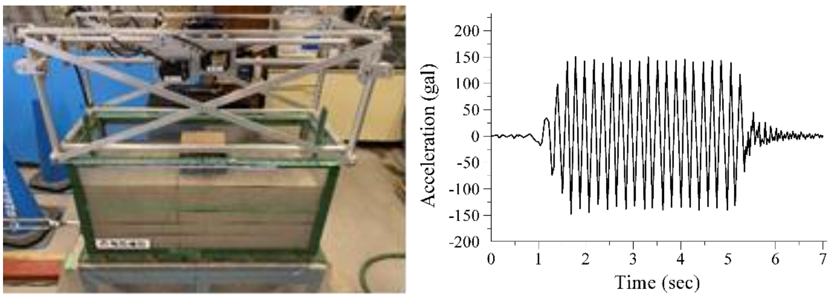


Figure 5. View of the test set-up (left) and input motion (right)

Figure 6 provides a detailed depiction of settlement measurements at each corner of the model following each step of excitation. Total settlement was determined by averaging the settlements recorded by the two laser displacement transducers. Penetration settlement of the model house was defined as the difference between the superstructure's settlement and the soil settlement.

Table 1 provides a comprehensive overview of the experimental setup, encompassing a total of sixteen cases. These cases scrutinize three critical parameters: 1) Improved depth. 2) Improvement width. 3)

Center-to-center spacing of the log piles. The log pile lengths utilized in the experiments are 25 cm, 17.5 cm, and 10 cm, corresponding to a liquefiable soil layer thickness (LD) of 30 cm. The log spacing configurations are either 4D (equivalent to four times the log pile's diameter) or 3D (three times the log pile's diameter). In all cases, the log piles boast a uniform 1 cm diameter (D).

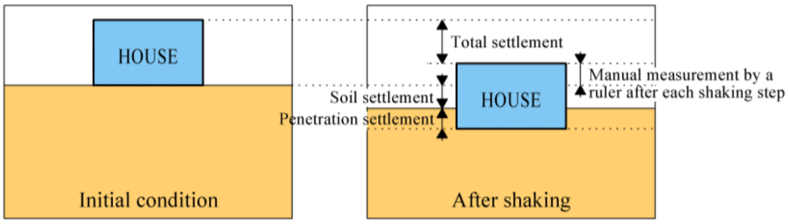


Figure 6. Definition for total settlement and penetration settlement

The chosen improvement widths encompass 13 cm, 37 cm, and 69 cm for scenarios with 4D spacing, and 16 cm and 40 cm for those with 3D spacing. To provide a comprehensive visual representation of the test conditions, both top-view and side-view depictions are presented in Figure 7 and Figure 8, respectively

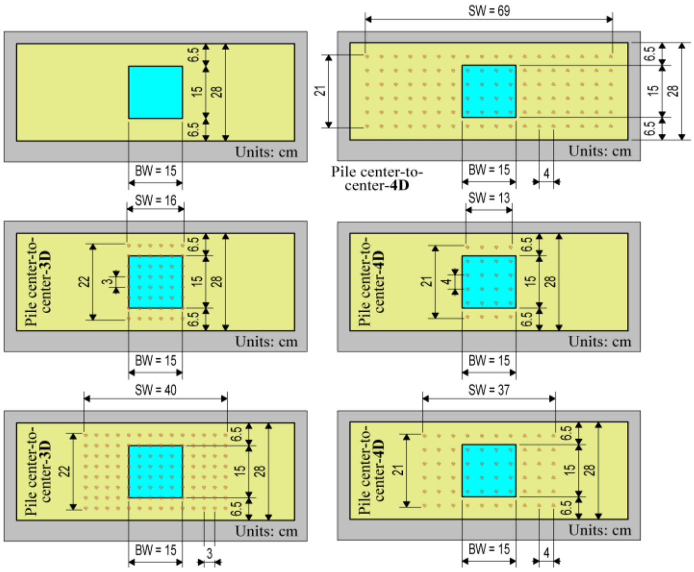


Figure 7. Top view of tests conditions (small-scale 1-g model tests)

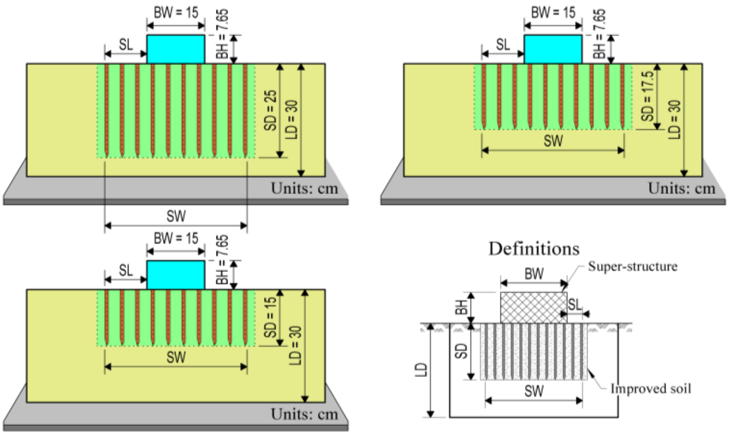


Figure 8. Side view of test conditions (small-scale 1-g model tests) and basic geometrical definitions

The presented research investigation delves deeply into the nuanced effects of varying improvement widths and center-to-center distances among log piles on both the overall and penetration settlements experienced by the superstructure. This comprehensive analysis encompasses diverse log pile lengths, each corresponding to distinct improved depths (SD) relative to the thickness of the liquefiable soil layer (LD) at ratios of 0.83, 0.58, and 0.33.

Table 1. Test conditions for small-scale 1-g shaking table tests

Test ID	Pile center-to-center distance	SW: Improvement width (cm)	SD: Improvement depth (cm)
Test 01	No improvement		
Test 02	4D (four times pile diameter)	13	25
Test 05		37	25
Test 06		13	17.5
Test 09		37	17.5
Test 10		13	10
Test 13		37	10
Test 14		69	25
Test 15		69	17.5
Test 16		69	10
Test 17	3D (three times pile diameter)	16	25
Test 18		40	25
Test 19		16	17.5
Test 20		40	17.5
Test 21		16	10
Test 22		40	10

Figure 9 serves as a visual representation of the intricate interplay between input acceleration and the resulting total settlement, meticulously captured through precision laser displacement sensors. In parallel, it provides a graphical depiction of the relationship between input acceleration and penetration settlement, discerned from averaged measurements obtained from the four corners of the model structure

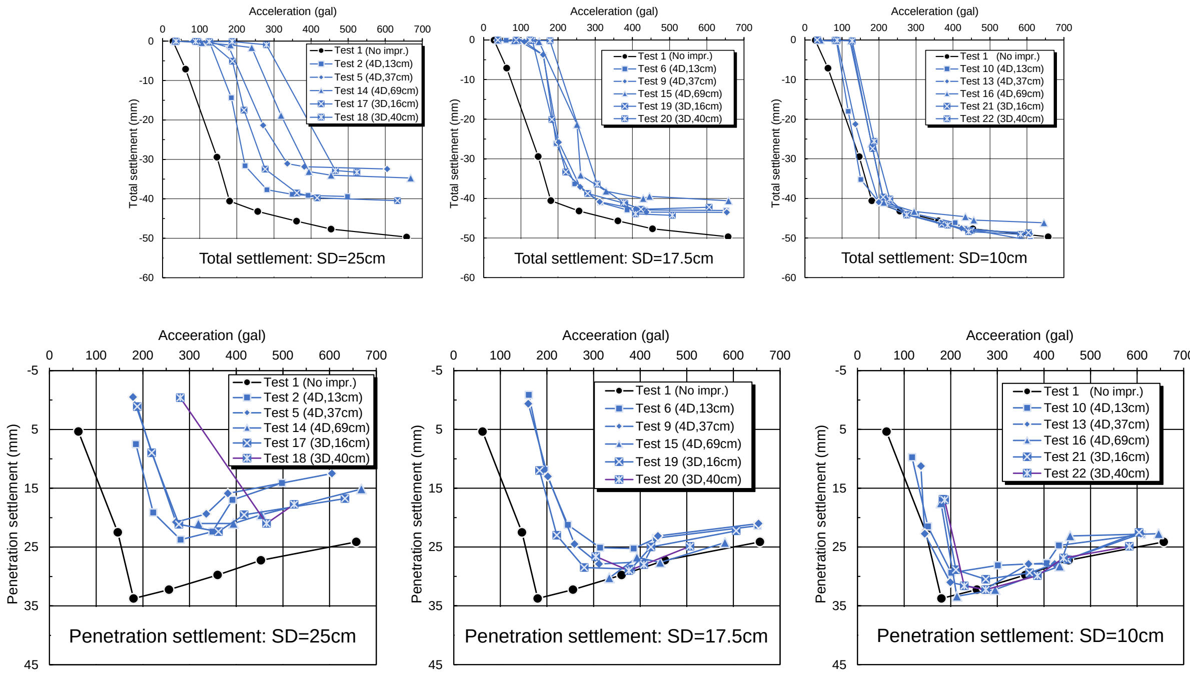


Figure 9. Cumulative settlement with acceleration increase – influence of the improved width and logs center-to-center distance

Specifically, when examining scenarios featuring an improved depth of 25 cm, a meticulous comparison has been undertaken among cases with identical log spacing but varying improvement widths. Notably, in instances characterized by 4D log spacing, a clear trend emerges: wider improvement widths yield substantially more pronounced reductions in settlement, while smaller log spacings accentuate this effect. Subsequent scrutiny has shifted to cases exhibiting similar improvement widths but divergent log spacings. Notably, within the context of 3D log spacing, both Test 2 and Test 17 initiate settlement responses at the same critical acceleration level (190 gal).

However, Test 2 exhibits more considerable settlement. A comparable pattern is observed in the case of Test 5 and Test 18. Broadly speaking, the configuration involving 3D log spacing tends to elicit a more pronounced improvement effect, particularly when coupled with larger improvement widths.

Furthermore, cases characterized by an improvement depth of 17.5 cm exhibit analogous trends concerning improvement width and log spacing. Similarly, instances marked by an improvement layer thickness of 10 cm, featuring 3D log spacing, generally produce marginally superior results. It is important to note that penetration settlement tends to decrease as total settlement escalates. This phenomenon can be attributed to the ongoing ground settlement process and the buoyancy effect exerted by the log piles on the superstructure, highlighting the multifaceted dynamics at play in liquefaction mitigation scenarios.

The study's findings shed light on how variations in log pile lengths (SD) exert influence over both the total and penetration settlements of the model structure, while keeping improvement widths and log spacings constant. To investigate these effects, log pile lengths corresponding to improved depth to liquefiable soil layer thickness (LD) ratios of 0.83, 0.58, and 0.33 have been considered for both 4D and 3D log spacing configurations. Specifically, for 4D log spacing, improvement width (SW) to house width (BW) ratios of 0.87 and 2.47 have been utilized, while for 3D log spacing, the ratios are 1.07 and 2.67. In Figure 10, the intricate relationships between acceleration and cumulative settlement are illustrated for three distinct improvement widths under 4D log spacing conditions. When the improvement width is set at 13 cm, Tests 2 and 6 exhibit similar settlement trends until total settlement stabilizes beyond 300 gal. Specifically, Test 2 reaches a settlement of 31.61 mm at 220 gal, Test 6 records 36.35 mm at 240 gal, and Test 10 achieves 41.03 mm at 200 gal. In contrast, with an improvement width of 37 cm, Test 5 shows 21.35 mm settlement at 270 gal, Test 9 registers 37.1 mm, and Test 13 observes 43.97 mm. It is evident that larger improvement widths result in more substantial reductions in total settlement, particularly when accompanied by greater improvement depth. Nevertheless, for an improvement width of 13 cm, the disparity in settlement between improvement depths of 25 cm and 17.5 cm is minimal. Figure 10 further elucidates the connection between acceleration and cumulative settlement for two different improvement widths under 3D log spacing. With a 16 cm width, Test 17 exhibits 5.11 mm settlement at 180 gal, Test 19 records 20.06 mm, and Test 21 observes 27.42 mm. In contrast, for a 40 cm width, Test 18 shows 0.94 mm settlement at 280 gal, Test 20 documents 23.5 mm, and Test 22 notes 44.18 mm. Larger improvement widths exhibit a more pronounced impact on total settlement, with the role of improvement depth becoming increasingly significant within this context.

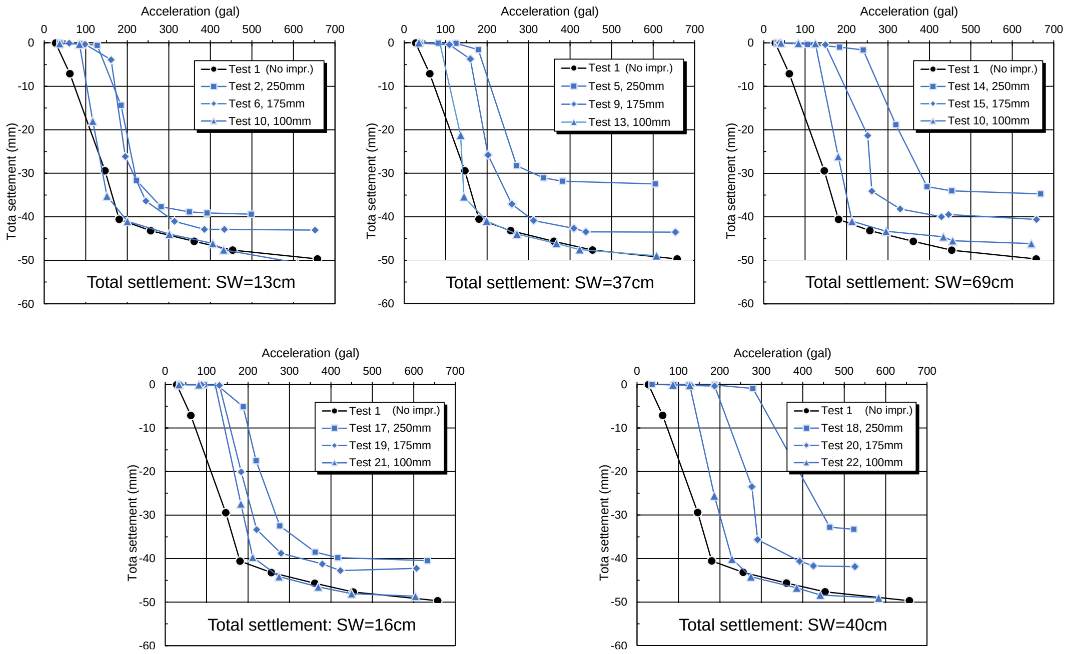


Figure 10. Cumulative settlement with acceleration increase – influence of the improved depth

The presented part of the study conducts a comparative analysis with prior research undertaken by [6], as illustrated in Figure 11. This comparison focuses on assessing how improvement width (SW) and depth (SD) impact the total settlement during seismic events. The study utilizes crucial parameters, including the ratio of improvement width to structure width (SW/BW) and the ratio of total settlement to the thickness of the liquefiable layer

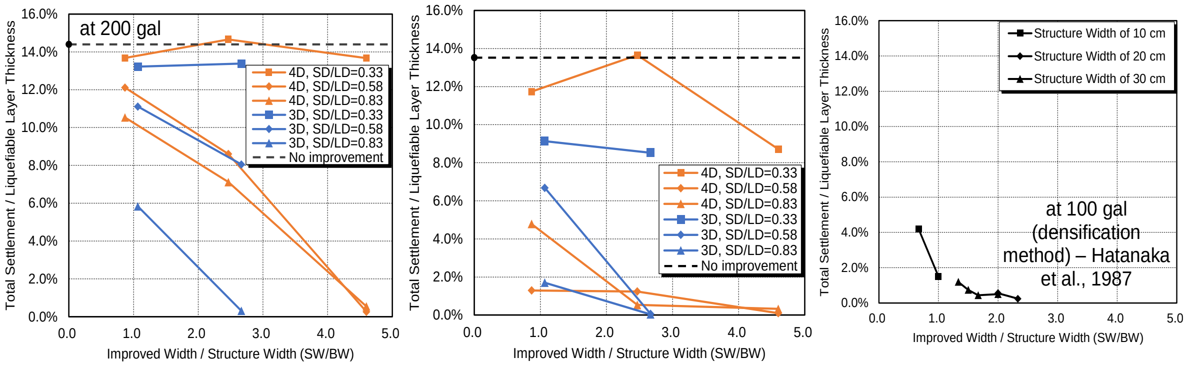


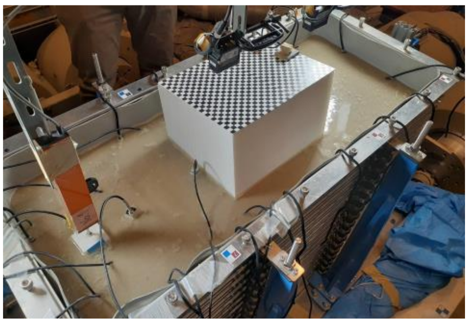
Figure 11. Effect of the log piling method on

The results shed light on the influence of various factors. For instance, when the improvement depth is set at 10 cm, which corresponds to an SD/LD ratio of 0.33, augmenting the improvement width has a relatively minor impact on the total settlement. In contrast, for depths of 17.5 cm and 25 cm (corresponding to SD/LD ratios of 0.58 and 0.83, respectively), wider improvement widths exhibit a notable reduction in settlement. An interesting observation is that at an improvement depth of 17.5 cm (SD/LD ratio of 0.58), the significance of log spacing diminishes when SW/BW exceeds 1.6. Furthermore, the findings suggest a broader trend: regardless of the specific improvement method employed, there is a noticeable stabilization of the total settlement to liquefiable layer thickness (LD) ratio when SW/BW surpasses the 1.6 threshold. This implies that increasing the improvement width beyond this point has a diminishing effect on settlement, particularly for a given level of acceleration.

In summary, for seismic events within the 150~200 gal range, the study identifies key parameters that effectively contribute to settlement reduction. These parameters include a log spacing of approximately 3D, an SW/BW ratio of 1.6, and an SD/LD ratio of 0.58.

MEDIUM-SCALE 1-G SHAKING TABLE TESTS

Silica sand No. 7 was intricately poured into a laminar soil box characterized by internal dimensions of 100 cm in width, 40 cm in depth, and 70 cm in height, as visually represented in Figure 12. The experimental conditions meticulously replicated those of the small-scale 1-g model tests. These conditions included maintaining a model-to-prototype ratio of 1/20, employing PVC piles (sticks) in lieu of wooden piles, adopting a pile diameter (D) of 1 cm, maintaining a structural width to height ratio (BW/BH) of 2.0, and applying a base stress of 0.75 kPa.



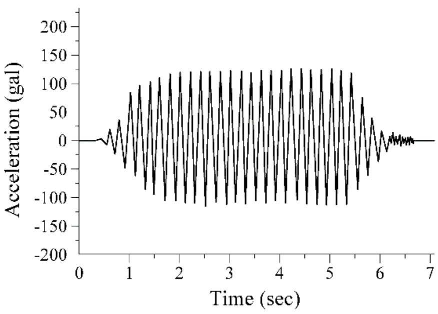


Figure 12. Model view (left) and input motion (right)

Table 2 provides a succinct overview of the five experimental cases, encompassing two cases without any improvement, distinguished by variations in liquefiable layer thickness (LD), and three cases implementing log pile densification improvement. Across all these scenarios, the ratio of improved depth to liquefiable layer thickness (SD/LD) is consistently maintained at 0.58, while preserving an improvement width to structure width ratio of 1.6. Moreover, the center-to-center distance between log piles is uniformly set at 3D for all cases. The primary focus of this series of tests revolves around emphasizing the absolute value of log pile length, or the improvement depth. Figure 13 offers a comprehensive side view of the test conditions across all experiments, clearly illustrating the placement of pore-water pressure transducers and accelerometers utilized in this study.

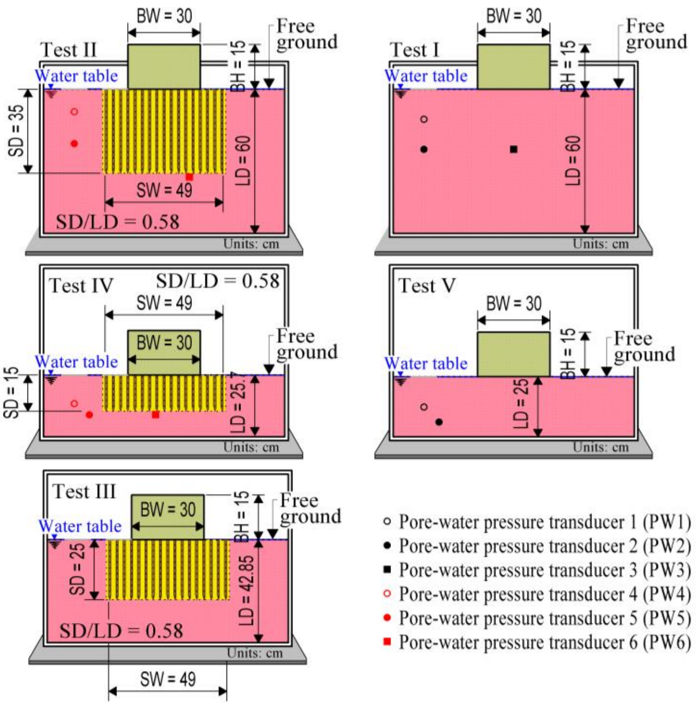


Figure 13. Side view of test conditions (medium-scale 1-g model tests)

Table 2. Test conditions for medium-scale 1-g shaking table tests

Test ID	SD: Improvement depth (cm)	LD: Liquefiable layer thickness (cm)	SD/LD Ratio
Test I	No improvement	60	-
Test II	35	60	0.58
Test III	25	42.85	
Test IV	15	25.7	
Test V	No improvement	25	-

Figure 14 illustrates the correlation between excess pore water pressure (PWP) ratio and cumulative penetration settlement. Comparing Test I and Test II, it is evident that both of them exhibit settlement within a certain PWP ratio range.

Similarly, Test II shows settlement within a similar range at equivalent PWP ratio levels, indicating the effectiveness of measures in Test II in reducing settlement compared to Test I. Such trend is also observed in the comparison of Test V and Test IV. Additionally, when comparing Test II and Test IV, Test II's measures achieve a more significant favorable effect.

Moreover, a comparative analysis of the acceleration responses in Test I and Test II is presented in Figure 15. The horizontal axis represents input acceleration, while the vertical axis displays the values measured by each accelerometer. First, the acceleration response at the top of the structure is examined. In Test I, it can be observed that after the liquefaction of the ground directly beneath the structure, the amplification in response decreases. Conversely, in Test II, although the amplification in response decreases in the free ground following liquefaction, it remains unchanged at the top of the structure. From these observations, it can be inferred that after liquefaction, the top of the structure in Test I experiences a reduction in response amplification, while Test II does not. This suggests that the stiffness of the ground in the improved area may have been maintained even after an increase in excess pore-water pressure. Furthermore, when comparing the free ground in Test I and Test II, it is evident that, for similar input motion, Test II exhibits a higher amplification in response. This suggests that the ground surrounding the improved zone may have undergone densification or reinforcing due to the presence of the piles. The acceleration response trends of the Test V and Test IV are similar, which means that liquefaction countermeasures in Test IV were not effective enough to maintain soil's stiffness during the shaking.

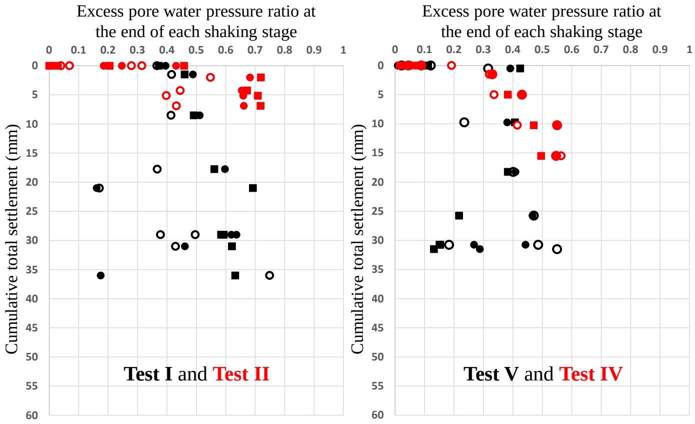


Figure 14. Relationship between excess pore water pressure ratio and cumulative total settlement at the end of each shaking stage

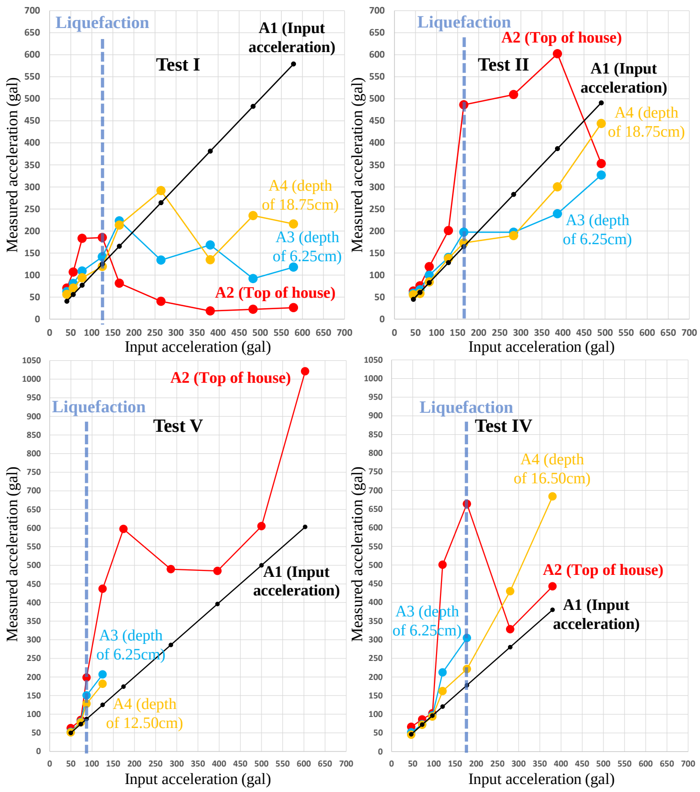


Figure 15. Comparison of acceleration response to input acceleration

The angles of some piles after their excavation are illustrated in Figure 16. The horizontal axis represents the position of the piles, while the vertical axis represents the tilt angles. Since the piles were manually driven, they may not be perfectly vertical, and there should be some degree of inclination before shaking was applied. Therefore, the evaluation focuses on the trend in angle magnitude rather than the exact angle values. Test II's angle data, depicted on the left side of Figures 16, indicates an absence of any significant angle trend across the piles. Test IV's angle data, represented in the right side of Figures 16, exhibits a trend with the left part of the graph showing larger negative angles and the right half of the graph showing larger positive angles. Middle part of Figure 16 schematically illustrates such behaviour.

Results show minimal lateral spreading in Test II, suggesting effective shear deformation restraint by the piles in the improved soil, while Test IV is associated with a failure mechanism (lateral spreading due to liquefaction).

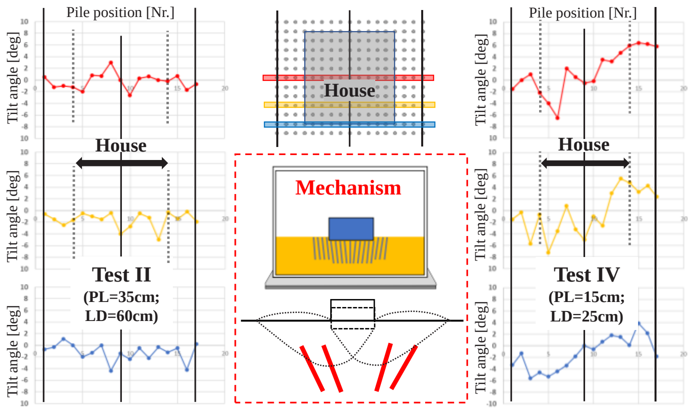


Figure 16. Piles tilt trend and detection of failure mechanism

Figure 17 offers a comprehensive summary of the results obtained from the medium-scale 1-g shaking table tests, outlining key trends concerning both penetration settlement and total settlement of the structure. In scenarios where no mitigation measures were applied, specifically Test I and Test V, it is evident that Test I, characterized by a greater liquefiable layer thickness (LD), experiences a correspondingly larger penetration settlement. However, when these cases are compared to scenarios involving the implementation of piles, such as Test II, Test III, and Test IV, a significant pattern emerges

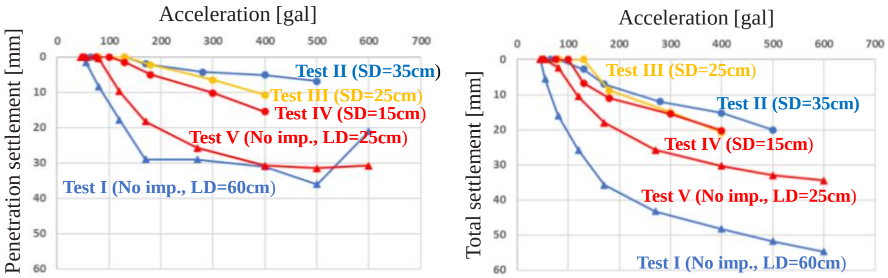


Figure 17. Acceleration versus penetration settlement/total settlement

Test II, featuring the longest pile length and LD, exhibits the smallest magnitude of penetration settlement. Conversely, Test IV, characterized by the smallest SD and LD values, displays the most substantial penetration settlement. Additionally, similar trends are observed when analyzing total settlement.

UNDRAINED CYCLIC TORSIONAL SHEAR TESTS

Data from undrained cyclic torsional tests (Figure 18 – right) played a pivotal role in calibrating the constitutive model (PM4Sand) employed for the numerical study. The tested material aligned with that utilized in the 1-g shaking table tests – Silica sand No. 7 at a relative density of 50%.

The torsional shear apparatus (Figure 18 – left) featuring hollow cylindrical specimens is acknowledged as a valuable tool for accurately assessing the soil response to liquefaction [9]. Notably, it allows the replication of simple shear conditions, which closely resemble the stress state experienced in the field during earthquakes. To prepare the specimens, four medium-sized hollow cylindrical specimens with dimensions of 100 mm in outer diameter, 60 mm in inner diameter, and 200 mm in height were created using the air pluviation method. To achieve specimens with highly uniform density, the falling height was meticulously maintained constant throughout the pluviation process. A high degree of saturation was accomplished (with Skempton's B-values exceeding 0.96) by continuously circulating de-aired water into the specimens. The specimens have been isotropically consolidated by increasing the effective stress state, □ ’c, up to 100 kPa, with a back pressure, ub, of 200 kPa. Subsequently, to emulate seismic conditions, a constant-amplitude undrained cyclic torsional shear stress, τ, was applied at a shear strain rate ranging from 0.25 to 1.0% per minute.

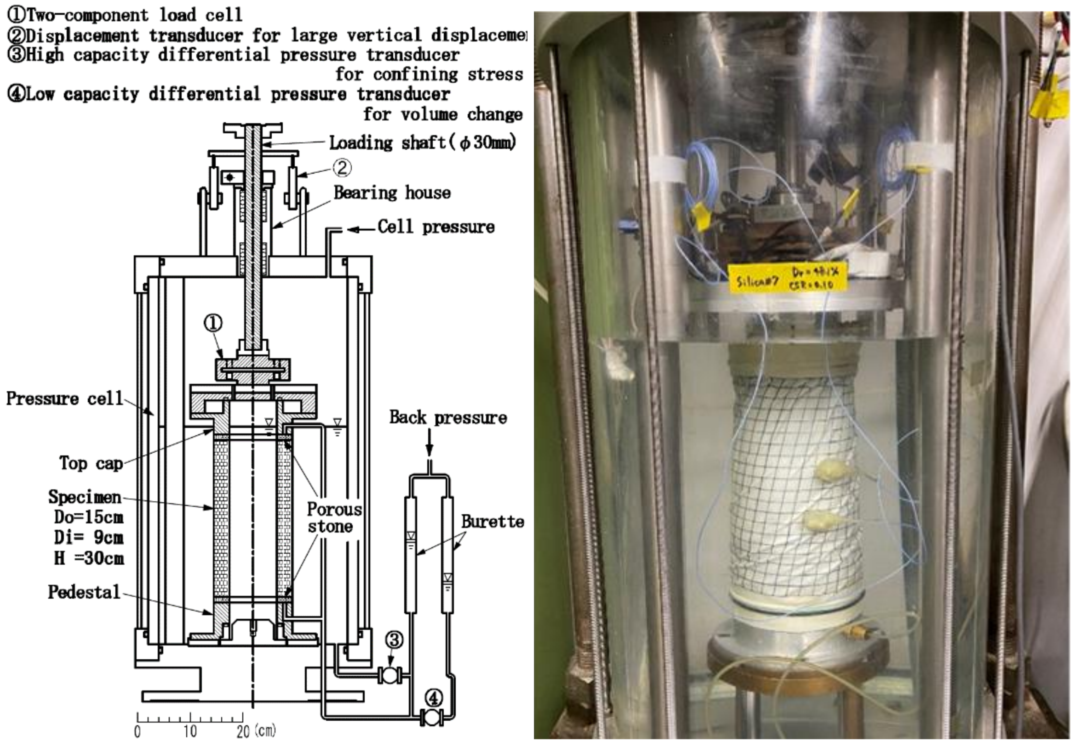


Figure 18. Cyclic torsional tests: torsional shear apparatus scheme [9], left, and testing, right

Experiments were conducted at various cyclic stress ratios, CSR, defined as the cyclic stress, τ, divided by the effective stress, □ ’c, including values of 0.100, 0.150, 0.200, and 0.250. The objective was to establish the CSR-N relationship (where N represents the number of cycles required to generate a 7.5% double amplitude of shear strain) and to derive relationships for "cyclic shear stress – shear strain," pore water pressure buildup, and stress paths. Corresponding to a 7.5 magnitude earthquake, it was determined that 15 cycles were needed to reach liquefaction at a specific cyclic stress ratio. Therefore, the laboratory-derived cyclic resistance ratio, CRR, was determined using the CSR-N relationship for 15 cycles (Figure 20 – left), with CRR set at 0.172.

CALIBRATION OF PM4SAND CONSTITUTIVE MODEL

The numerical investigation in this study employs the PM4Sand constitutive model, renowned for its ability to effectively replicate sand material behavior under dynamic loading conditions. This model encompasses the intricate phenomena of pore pressure generation, liquefaction, and post-liquefaction responses, rendering it particularly appealing for industrial applications due to its parsimonious parameter set requiring calibration.

The PM4Sand model is rooted in the fundamental framework of stress-ratio controlled, critical state-compatible, bounding surface plasticity model for sands, as originally proposed by [10]. Subsequently, it has been adapted and configured in the context of plane stress conditions, as introduced by [11].

This adjustment facilitates its integration into 2D plane strain numerical models, where out-of-plane stress is omitted from the global finite element equations.

Model parameters are categorized into two groups, each serving specific purposes (detailed descriptions can be found in Table 3):

- A primary set comprising 4 parameters (D_{R0}, G_0, h_{p0}, and p_A), which hold paramount significance in the calibration process.
- A secondary set encompassing 9 parameters (e_{max}, e_{min}, n^b, n^d, ϕ_{cv}, ν, Q, R, and PostShake).

In this study, D_{R0}, e_{max}, and e_{min} have been determined based on outcomes from conventional laboratory tests that establish the physical characteristics of the soil. ϕ_{cv} has been established by referencing the critical line derived from stress paths, specifically cyclic torsional tests. G_0 has been derived from shear wave velocity (V_s) measurements employing bender elements conducted prior to the cyclic torsional tests. All other parameters, except for h_{p0}, have been utilized at their default values.

The calibration process for a relative density of 50% (representative of the non-improved zone) involves adjustments to h_{p0}, utilizing experimentally obtained data from cyclic torsional tests. The procedure can be summarized as follows:

- Set relative density, D_r, to 50%.
- Numerically simulate cyclic torsional tests across a range of cyclic stress ratios (*CSR*) corresponding to laboratory tests (0.100, 0.150, 0.200, and 0.250). Fit simulated stress paths, "cyclic shear stress - shear strain" relationships (see Figure 19), and the *CSR-N* relationship to match the experimentally obtained data (laboratory) – refer to Figure 20, left.

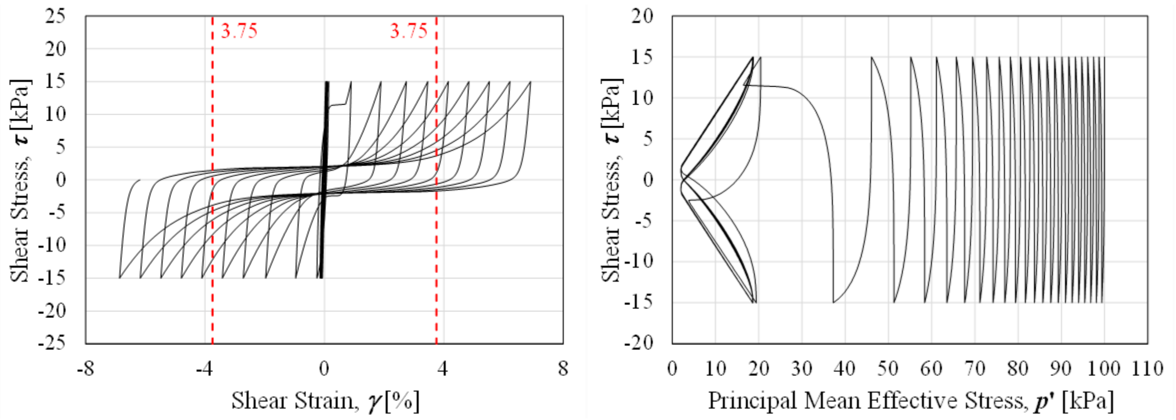


Figure 19. Numerically simulated cyclic torsional test: "cyclic shear stress – shear strain" relation, left, and stress path, right

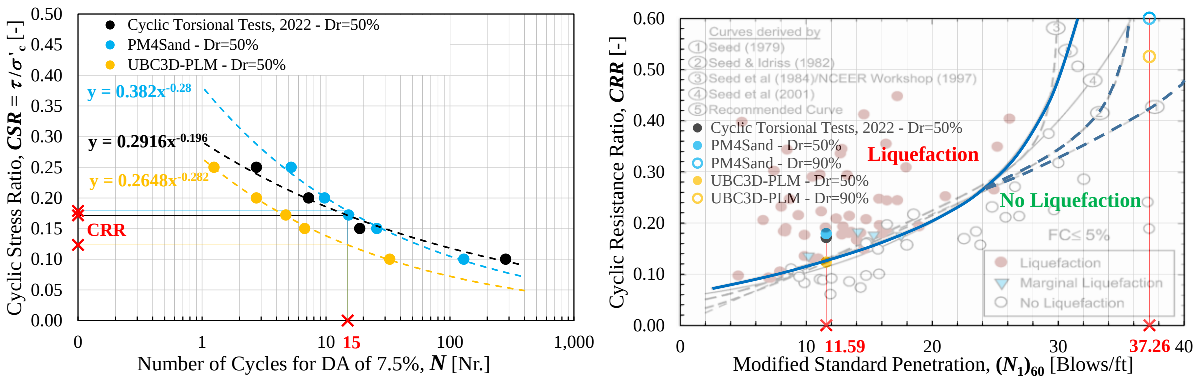


Figure 20. Comparison of experimentally and numerically obtained: CSR-N relationships, left, and CRR – plotted against a commonly used cyclic strength curve from the literature (Seed et al. 1985), right

- Define the cyclic resistance ratio (*CRR*) using the numerically and experimentally obtained *CSR-N* relationships for 15 cycles. Ensure that both *CRR* values closely align with each other and compare them with commonly utilized cyclic strength curves from existing literature (accounting for the conversion of relative density, D_r, to energy-corrected SPT blow count, $(N_1)_{60}$, as shown in Figure 20, right).

- Given the absence of experimental data for a relative density, D_r, of 90% (representing the improved zone), a distinct procedure is adopted, with parameter variation exclusively focused on h_{p0}:
- Set relative density, D_r, to 90%.
- Convert D_r to $(N_1)_{60}$. Establish the *CRR* value corresponding to $(N_1)_{60}$ based on widely accepted cyclic strength curves from the literature.
- Simulate cyclic torsional shear tests and adjust the constitutive model parameters until the adopted *CRR* value aligns with the *CSR-N* relationship for 15 cycles.
- For reference, Table 3 provides the adopted values for all parameters used in the PM4Sand constitutive model formulation, accompanied by a concise overview of the calibration procedure.

Table 3. Summary of PM4Sand parameters – adopted values and calibration procedure

	Symbol	Adopted Value		Default Value	Unit	Description	Calibration Procedure
		D_{R0} = 50%	D_{R0} = 90%				
Primary Parameters	$\boldsymbol{D_{R0}}$	0.50	0.90	-	[-]	Relative density	Lab data or D_{R0} = SQRT[$(N_1)_{60}$/46]
	$\boldsymbol{G_0}$	625	1380	-	[-]	Shear modulus coefficient	$G_0 = [\rho(V_s)^2]/[p_A$ x SQRT(p_{ref}/p_A)] or G_0 = 167 x SQRT[$(N_1)_{60}$ + 2.5]
	$\boldsymbol{h_{p0}}$	0.545	0.425	-	[-]	Contraction rate parameter	Based on CRR-(N1)60 curve for M_w = 7.5 (or 15 uniform loading cycles - element tests) - fitting.
	$\boldsymbol{p_A}$	101.3	101.3	101.3	[kN/m^2]	Atmospheric pres.	-
Secondary Parameters	$\boldsymbol{e_{max}}$	1.225	1.225	0.8	[-]	Maximum void ratio	Basic physical parameters based on lab tests.
	$\boldsymbol{e_{min}}$	0.727	0.727	0.5	[-]	Minimum void ratio	
	$\boldsymbol{n^b}$	0.5	0.5	0.5	[-]	Bounding surface par.	-
	$\boldsymbol{n^d}$	0.1	0.1	0.1	[-]	Dilatancy surface par.	-
	$\boldsymbol{\phi_{cv}}$	44.0	50.0	33	[^{o}]	Crit. state fric. angle	Basic strength parameters based on element tests.
	$\boldsymbol{\nu}$	0.3	0.3	0.3	[-]	Poisson's ratio	
	$\boldsymbol{Q}$	10	10	10	[-]	Crictial state line parameters	-
	$\boldsymbol{R}$	1.5	1.5	1.5	[-]		-
	PostShake	0	0	0	[-]	Post shake switch	PostShake switch (0 or 1) deactivates or activates the reduction of elastic stiffness in order to simulate the post-shaking reconsolidation. PostShake = 1 only after the end of strong shaking (two separate analysis phases).

COMPARISON BETWEEN PM4SAND AND UBC3D-PLM CONSTITUTIVE MODELS

The UBC Sand Constitutive Model (UBC3D-PLM in PLAXIS 2D library) is a widely used tool in geotechnical engineering, particularly for analyzing liquefaction phenomena in sandy soils under dynamic loading conditions, such as earthquakes. Developed at the University of British Columbia, this model is essential for simulating the complex behavior of sands, including pore pressure generation, liquefaction onset, and post-liquefaction responses. It requires calibration based on laboratory test data (procedure is given in Table 4) to accurately predict liquefaction potential, making it a valuable asset for assessing and mitigating liquefaction risks in engineering practice.

Utilizing the proposed calibration procedure for UBC3D-PLM, the selected parameters for this study are presented in Table 4. In order to facilitate a meaningful comparison between the outcomes of an undrained cyclic torsional shear test (specifically, with a CSR of 0.200) and those generated through numerical simulations employing two widely recognized and extensively employed constitutive models, namely PM4Sand and UBC3D-PLM, particularly in the realm of liquefaction assessment, an illustrative representation is given in Figure 21.

Table 4. Summary of UBC3D-PLM parameters – adopted values and calibration procedure

	Symbol	Adopted Value		Default Value	Unit	Description	Calibration Procedure
		D_{R0} = 50%	D_{R0} = 90%				
Stiffness parameters	k^{*e}_{B}	732	977	-	[-]	Elastic bulk modulus fa	$k^{*e}_{B} = 0.7 \times k^{*e}_{G}$
	k^{*e}_{G}	1046	1395	-	[-]	Elastic shear modulus f	$k^{*e}_{G} = 21.7 \times 20 \times (N_1)_{60}^{0.3333}$
	k^{*p}_{G}	717	4723	-	[-]	Plastic shear modulus f	$k^{*p}_{G} = k^{*e}_{G} \times (N_1)_{60}^{2} \times 0.003 + 100$
	me	0.65	0.50	0.50	[-]	Rate of stress-dependency of elastic bulk modulus	Step 1: Based on cyclic element tests - fitting of τ-γ and τ-p' curves.
	ne	0.65	0.50	0.50	[-]	Rate of stress-dependency of elastic shear modulus	
	np	0.55	0.40	0.40	[-]	Stress-dependency of plastic shear modulus	
Strength Parameters	p_{ref}	100	100	100	[kN/m^2]	Reference pressure	Basic strength parameters based on element tests.
	ϕ_{cv}	44.0	50.0	33	[°]	Critical state friction an	
	c	0.0	0.0	0.0	[kN/m^2]	Cohesion	
	σ_t	0.0	0.0	0.0	[kN/m^2]	Tension cut-off and ten	
	ϕ_p	45.0	53.6	34.4	[°]	Peak friction angle	$\varphi_p = \varphi_{cv} + (N_1)_{60}/10 + \max(0; [(N_1)_{60} - 15]/5)$
Advanced Parameters	$(N_1)_{60}$	14.03	33,24	-	[-]	Corrected SPT value	SPT data or $(N_1)_{60} = 46 \times (D_{R0})^2$
	R_f	0.74	0,65	0.90	[-]	Failure ratio	$R_f = 1.1 \times [(N_1)_{60}]^{-0.15}$
	f_{dens}	0.625	1.000	1.00	[-]	Densification factor	Step 2: Based on CRR-$(N_1)_{60}$ curve for M_w = 7.5 (or 15 uniform loading cycles - element tests) - fitting.
	f_{Epost}	0.200	1.000	-	[-]	Post-liquefaction factor	

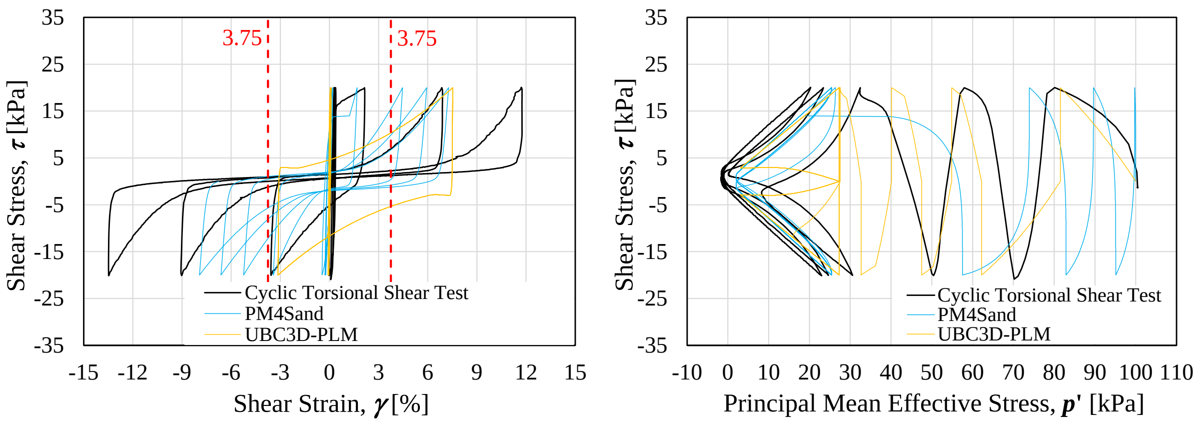


Figure 21. "Cyclic shear stress – shear strain" relation, left, and stress path, right – comparison between a cyclic torsional shear test and numerical simulations (PM4Sand and UBC3D-PLM)

Ultimately, PM4Sand has been chosen as the constitutive model for the presented FEM analysis, a preference commonly observed in engineering practice, particularly for conducting analyses related to liquefaction-induced settlement.

NUMERICAL MODEL

Figure 22 provides an illustrative depiction of the numerical model and the 1-g model setup. The numerical models, founded upon the Finite Element Method (FEM), have been meticulously developed using PLAXIS 2D software [12]. Three distinct models have been crafted to faithfully replicate the conditions observed in the 1-g model tests. The numerical models are inherently designed to mirror the experimental conditions. They encompass separate definitions for the improved zone, characterized by a relative density, D_r, of 90%, and the non-improved zone, featuring a relative density, D_r, of 50%. The log-piles, emulating the physical counterparts, are represented using embedded beam row elements. The structural representation aligns with the house configuration employed in the model tests, factoring in self-weight considerations (equivalent to a base stress of 15 kPa) and adhering to geometrical specifications.

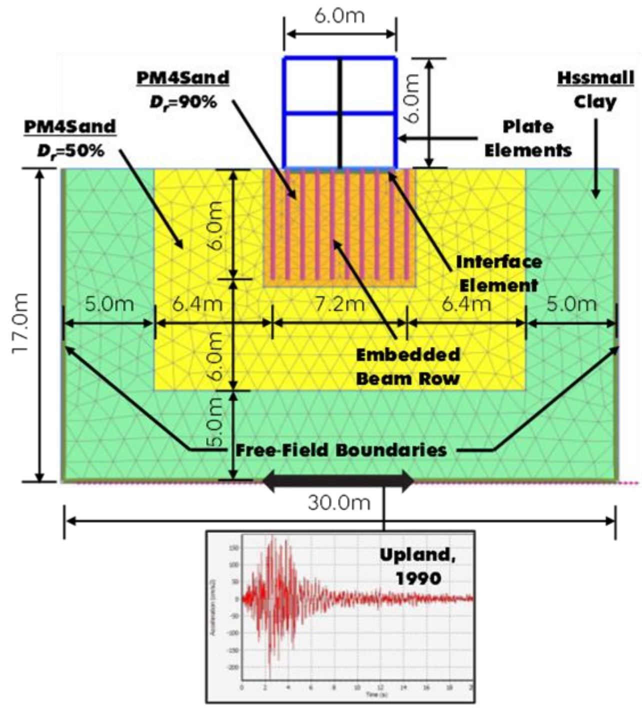


Figure 22. Overview of the numerical model

To account for interface dynamics between the soil and the superstructure, interface elements have been judiciously incorporated within the contact zone. Given the adoption of free-field boundary conditions, it is essential to acknowledge the potential for stress concentration within the region directly interfacing with the liquefiable material. Consequently, a drained zone consisting of non-liquefiable material (HS-Small Model) has been included as a supplementary feature around the liquefiable field. The time-history analysis leverages a representative accelerogram, sourced from the 1990 Upland earthquake records. To systematically scrutinize the system's response across varying acceleration levels, the accelerogram has been meticulously scaled for multiple iterations. It's important to acknowledge that while the physical and numerical models endeavor to capture similar phenomena, direct comparisons are somewhat limited due to inherent differences in scale and input motion between the two approaches.

FEM ANALYSIS RESULTS

Figure 23 and Figure 24 present notable findings from the numerical simulations conducted at an approximate ground acceleration of 140 gal.

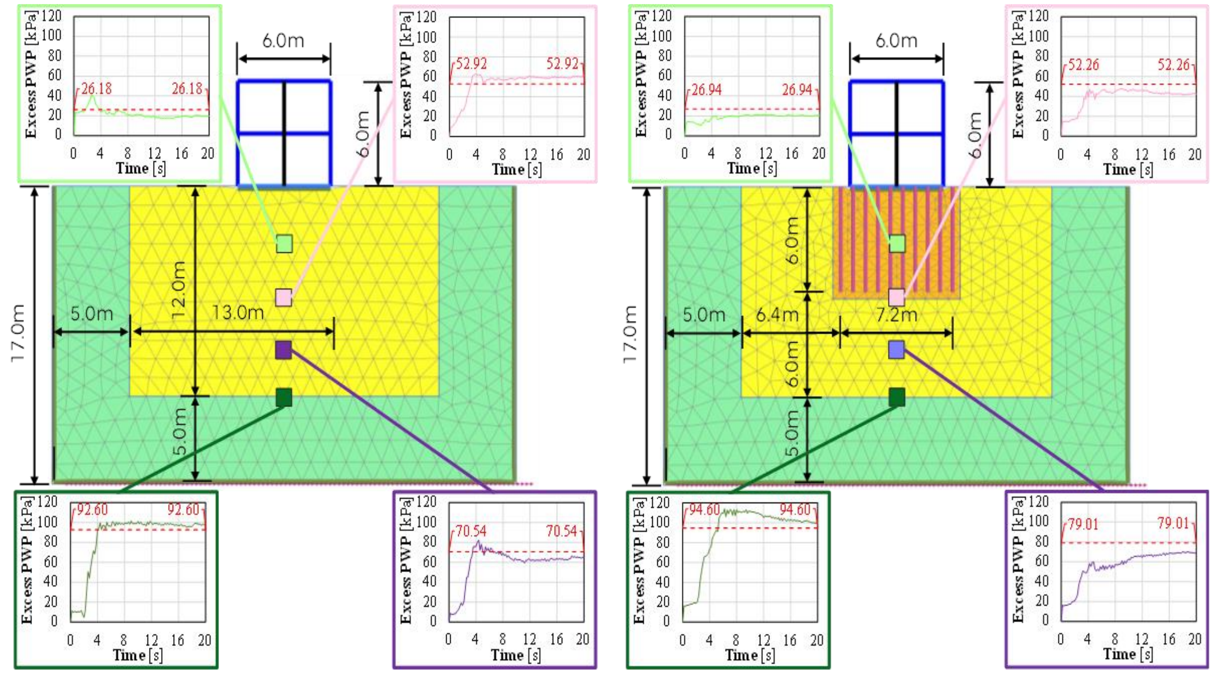


Figure 23. 140 gal: Excess pore-water pressure time history in unimproved case (left) and improved case – piles below and around the structure (right)

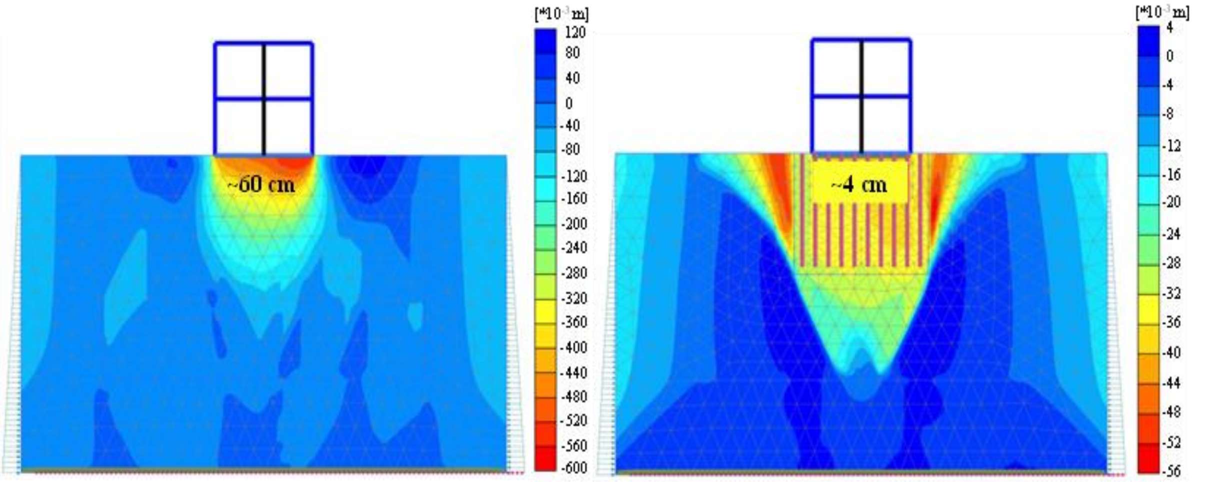


Figure 24. 140 gal: Vertical deformation (liquefaction-induced settlement) of unimproved case (left) and improved case – piles below and around the structure (right)

The analysis of excess pore-water pressure build-up highlights a significant contrast between the non-improved and improved cases. In the former, the entire sand layer experiences liquefaction. Conversely, the improved case demonstrates a more favorable response, with liquefaction observed solely at a depth of 12 meters. In other zones, the ratio of pore-water pressure to effective stress remains below 1.0. This comparison strongly suggests that shallow ground improvement through log-pile installation could positively influence the behavior of the "superstructure – foundation – soil" system, potentially serving as an effective countermeasure against liquefaction. Furthermore, the impact of improvement becomes more evident when assessing the settlement of the superstructure. At an acceleration of approximately 140 gal, the improved case exhibits settlement more than five times smaller than the non-improved case – specifically, 120 mm compared to 618 mm. Liquefaction-induced settlement serves as a pivotal metric for quantifying the effectiveness of mitigation measures.

Figure 25 (left) provides a summarized representation of normalized total settlements derived from the numerical analysis. A comparison with the 1-g shaking table test study by [13], reveals a noteworthy alignment in trends between both approaches. At relatively low accelerations of 80 gal, the non-improved case experiences significant settlement of the superstructure, while the two improved cases show considerable reductions in vertical displacements. This trend becomes even more pronounced with increasing acceleration. The two improved cases, involving improvement beneath the superstructure and improvement encompassing both beneath and around it, exhibit similar responses of the "superstructure – foundation – soil" system.

However, differences in response become apparent after approximately 300 gal, emphasizing the influence of additional improvement width around the house. This reaffirms the recommendations from prior studies [6,14,15] regarding the inclusion of such an additional treatment zone. Figure 25 (right) supplements this by summarizing the collective insights of other authors [16] concerning the significance of the improvement thickness (utilizing various liquefaction mitigation techniques) in relation to the total liquifiable layer thickness ratio.

The graph clearly demonstrates that a ratio exceeding 0.5 significantly reduces liquefaction-induced settlement – a conclusion reinforced by the results of the current study.

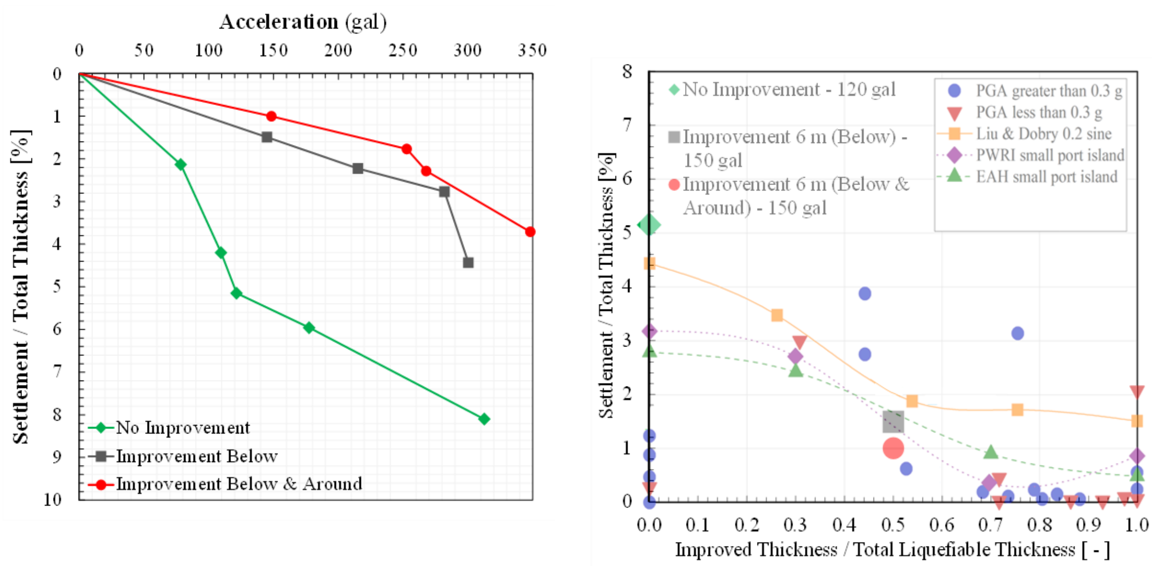


Figure 25. Normalized total settlement against acceleration, left, and countermeasure effect from this study and previous studies (New Zealand Geotechnical Society, 2017), right

CONCLUSIONS

The research presented in this paper highlights the effectiveness of the log piling method as a viable solution for mitigating soil liquefaction, especially in shallow ground improvement scenarios. By increasing soil density and providing reinforcement, log piling plays a crucial role in preventing liquefaction and reducing structural damage. The study initially focused on determining the impact of three critical factors – depth of improvement, width of improvement, and spacing between log piles – on the total and penetration settlements of structures. These evaluations were carried out using 1-g shaking table tests in a small-scale rigid soil box, revealing that wider improvement areas and closer pile spacing significantly influence settlements caused by liquefaction.

Key findings of this research include insights into mitigating liquefaction during seismic activities within the 150-200 gal range. The study identified three vital parameters for minimizing liquefaction effects: 1) The optimal center-to-center spacing of log piles, found to be about three times their diameter. 2) The ratio of improvement width to the structure's width, which proved most effective when exceeding 1.6. 3) The ratio of the thickness of the improved layer to the thickness of the liquefiable layer, with maximum effectiveness around 0.58. Building on these findings, the research progressed to a second phase involving medium-scale 1-g shaking table experiments in a laminar soil box, focusing on the impact of log pile length (improved depth). The results indicated that longer piles are more effective in reducing excess pore water pressure and settlement from liquefaction, maintaining a constant ratio of improved depth to liquefiable layer thickness. Furthermore, the study highlighted the importance of utilizing data from laboratory element tests, like cyclic torsional shear tests, for calibrating constitutive models.

Comparing results from 1-g shaking table tests with numerical finite element method (FEM) simulations showed consistent trends, affirming the positive impact of log piling even when treating a small portion of the liquefiable layer. Overall, the findings provide crucial guidance for optimizing the use of log-based techniques in liquefaction mitigation for engineering applications.

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LIQUEFACTION-INDUCED DAMAGE IN THE CITIES OF ISKENDERUN AND GOLBASI AFTER THE 2023 TURKEY EARTHQUAKE

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SUMMARY

On February 6th 2023 two major earthquakes struck southeast Turkey, M7.7 Kahramanmaras and M7.6 Elbistan, respectively. Unfortunately, due to the impact of these catastrophic events more than 50 000 casualties and 35 000 collapsed buildings have been reported since then. The aim of the study is to demonstrate preliminary site response analysis and assessment of re-liquefaction potential of sites which have been affected by the earthquakes – especially the cities of Iskenderun and Golbasi. Both site-specific areas have clear evidences of liquefaction and lateral spreading events which imply the focus of the presented paper. A series of geophysical MASW and microtremor tests have been performed in order to determine shear wave velocities up to depth of 30 m as well as the fundamental natural frequency of the soil deposits.

Moreover, samples have been collected from sand and silt ejecta in order to evaluate some basic physical properties – grain-size curves, specific gravity and plasticity parameters. On the basis of the obtained data seismic classification of the investigated sites according to current design codes has been made and in-depth distance to relatively stiff layer has been assumed. For the sake of evaluating risk of re-liquefaction the widely-used simplified stress-based approach to triggering assessment has been adopted considering some rules of the thumb (e.g., sieve analysis and plasticity properties evaluation). Lastly, post-liquefaction reconsolidation settlement and lateral displacement have been determined in terms of future earthquakes.

Key words: *lateral spreading, liquefaction assessment, shear wave velocity, microtremor, microseismic characterization, Iskenderun, Golbasi, 2023 Turkey-Syria Earthquake*

INTRODUCTION

Following the significant seismic events that transpired in Turkey and Syria on February 6th, 2023, a joint reconnaissance team was swiftly assembled. This multidisciplinary team brought together researchers and engineers hailing from Japan, Turkey, and Bulgaria, representing prominent organizations in the field of earthquake engineering. These organizations included the Japan Association for Earthquake Engineering, the Architectural Institute of Japan, the Japan Society of Civil Engineers, the Japanese Geotechnical Society, Bogazici University, MEF University, and the

University of Architecture, Civil Engineering and Geodesy (UACEG). Their collaborative efforts culminated in an extensive on-site investigation conducted between March 28th and April 2nd, 2023, which transpired approximately two months after the catastrophic earthquakes.

During the survey of the Islahiye region, several noteworthy findings came to light. Among them, a landslide event had transpired, leading to the formation of a landslide dam. Furthermore, another landslide occurrence unfolded in Tepehan, impacting a relatively gentle slope predominantly composed of limestone. This event sparked inquiries regarding potential links to fault movements.

The city of Iskenderun encountered a multifaceted array of challenges in the aftermath of the seismic events. In addition to documented building collapses occurring on soft ground, instances of building tilting and ground subsidence were attributed to the liquefaction of reclaimed coastal soil. These complexities further compounded the post-earthquake scenario.

In Golbasi, the survey revealed significant structural damage in buildings with shallow foundations that were situated on soft ground. This damage was primarily attributed to liquefaction-induced effects, including tilting and settlement. However, a more detailed investigation is imperative to precisely delineate the extent of liquefied soil layers in the area.

Antakya and Kahramanmaras emerged as regions where building damage correlated with surface ground vibrations. Despite experiencing severe building collapses, Antakya exhibited relatively stable ground conditions characterized by an average shear wave velocity (V_s) exceeding 400 m/s. This suggests the possibility of wave amplification linked to underlying geological factors. Conversely, Kahramanmaras witnessed notable building damage concentrated in alluvial fan formations.

This collaborative reconnaissance mission has yielded invaluable insights into the diverse impacts of the earthquakes on different regions – survey route is presented on Figure 1. It underscores the critical importance of conducting further research and comprehensive investigations to gain a complete understanding of the underlying geological and geotechnical factors contributing to these effects. Such knowledge will be instrumental in enhancing earthquake preparedness and mitigation measures in these areas.

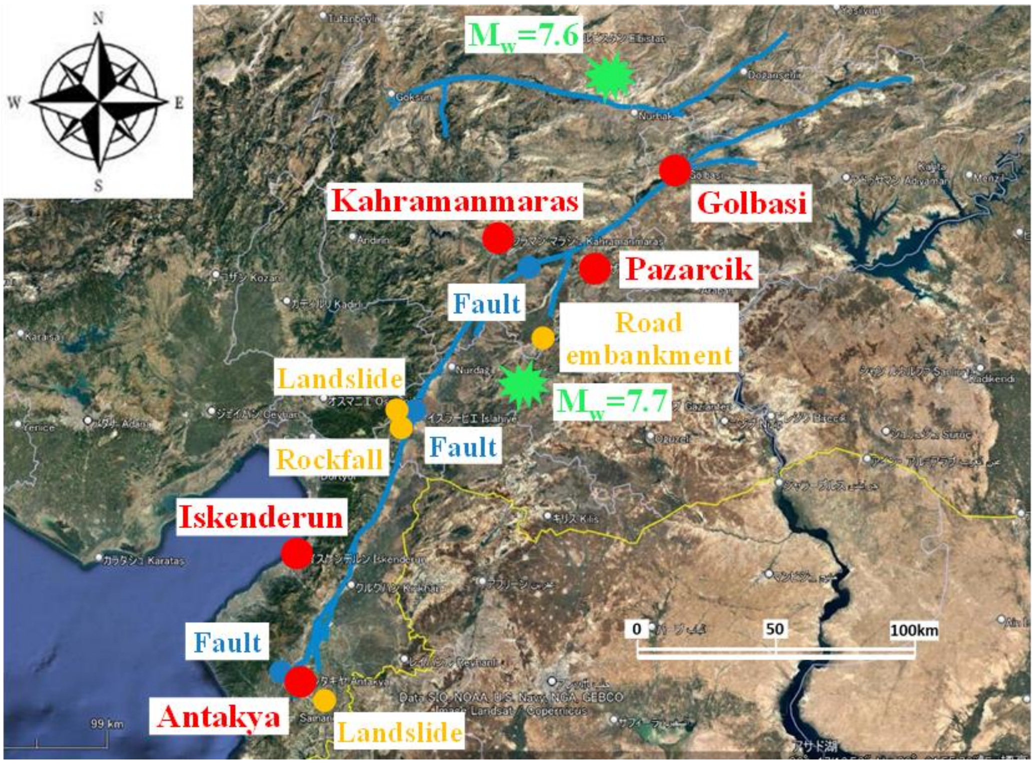

Figure 1. Map of the affected area (including fault and epicenter) – survey route

EARTHQUAKE CHARACTERISTICS

On February 6th, 2023, at 4:17 local time (1:17 GMT), a significant earthquake, measuring 7.7 on the moment magnitude (M_w) scale, with a focal depth of 8.6 km, struck in the vicinity of Kahramanmaras (epicenter: 37.228N, 37.043E) along the East Anatolian Fault, which traverses the border between Turkey and Syria. Roughly nine hours later, at 13:24 local time (10:24 GMT), another major earthquake, with a moment magnitude (M_w) of 7.6 and a focal depth of 7.0 km, occurred in Elbistan (epicenter: 38.089N, 37.239E) in the same region. Subsequently, on February 20th, the region experienced a significant aftershock, registering a moment magnitude (M_w) of 6.4 with a focal depth of 22 km, centered in Hatay (epicenter: 36.037N, 36.021E). These seismic events resulted in devastating consequences, including a significant loss of life, widespread destruction of more than half a million structures, including critical infrastructure such as communication and energy facilities, and substantial economic losses [1].

These earthquakes induced left-lateral fault ruptures along the East Anatolian Fault, spanning from Malatya to Hatay and covering a distance of approximately 320 km. While the severe damage to buildings and pipelines along this rupture has been discussed elsewhere in the context of lifeline impacts, this study primarily focuses on the geotechnical damage arising from the intense ground shaking. The most intense recorded ground motion was observed at the Pazarcik station [2]. The earthquake's duration was approximately 60 seconds, during which peak horizontal ground accelerations exceeded 2g (where g represents the acceleration due to gravity, approximately 9.81 m/s²). Notably, this earthquake generated numerous strong-motion records collected by seismographs located in close proximity to the fault, which spans over 300 km in length [2]. These records not only help address gaps in the existing attenuation equation for short distances from the hypocenter but also provide valuable insights into the generation mechanism of ground motion associated with fault movement. The maximum acceleration recorded near the epicenter significantly surpassed the predictions of previous attenuation equations.

In response to this catastrophic event, a collaborative reconnaissance team was assembled, comprising researchers and engineers from Japan, Turkey, and Bulgaria. This team was organized by prominent earthquake engineering organizations, including the Japan Association for Earthquake Engineering (JAEE), the Architectural Institute of Japan (AIJ), the Japan Society of Civil Engineers (JSCE), the Japanese Geotechnical Society (JGS), Bogazici University, MEF University, and the University of Architecture, Civil Engineering and Geodesy (UACEG). Led by Professor Kusunoki of the Institute of Industrial Science, University of Tokyo, this team conducted an extensive on-site investigation from March 28th to April 2nd, 2023, approximately two months after the earthquakes. The team consisted of 27 Japanese members, 22 Turkish professors and practitioners, 1 Bulgarian researcher and 2 Mexican geophysicists and with their base of operations in Gaziantep. They conducted daily visits to different disaster-affected areas for assessments.

As mentioned earlier, these earthquakes led to widespread destruction of buildings and a significant loss of life. One of the main objectives of the geotechnical field investigation was to determine whether the extent of such damage depended not only on the structural characteristics of buildings but also on the seismic properties of the soil. Due to time constraints, the decision was made to focus the survey on sites and towns with pre-existing information available. In addition to ground-based assessments, aerial surveys using drones were conducted to evaluate damage over a broader area. Various geotechnical investigation methods were utilized, including the portable dynamic cone penetration test (PDCPT), the multichannel analysis of surface wave (MASW), and microtremor-based horizontal-to-vertical spectral ratio (H/V) measurements, to characterize ground properties. Furthermore, interviews were conducted with local residents.

For the sake of converting portable dynamic cone penetration test (PDCPT) blow counts (N_d) to NSPT (standard penetration test, SPT, values), an equation proposed by [3], was employed, assuming sandy soil characteristics:

$$N_{\mathrm{SPT}} = 0.66_{\mathrm{x}} N_d \text{ (when } N_d \leq 4\text{) and } N_{\mathrm{SPT}} = 1.1 + 0.30_{\mathrm{x}} N_d \text{ (when } N_d > 4\text{)} \quad (1)$$

The study area is situated in the southeastern part of Turkey, extending from Kahramanmaras to Antakya. This region is surrounded by elevated mountain ranges on all sides, including the Amanos Mountains to the west (comprising pre-Cambrian to Eocene rock units), the Southeast Anatolian Mountain Range to the east (comprising Cretaceous to Miocene rock units), and the Baer-Bassit Range to the south (predominantly composed of Cretaceous ophiolite and Miocene rock units). The region features diverse topography, including multiple mountain ranges, river valleys, and plateaus. It is located at the convergence of the Arabian and African tectonic plates with the Anatolian block, making it seismically active and contributing to its varied topography. Key fault zones in this region include the East Anatolian Fault Zone (EAFZ), Dead Sea Fault Zone (DSFZ), and Karasu Fault System (KFS). Several residential areas are located within Quaternary geological units in the graben, primarily consisting of alluvial fans, alluvium, and basaltic compound volcanic rocks.

ISKENDERUN CITY

The city of Iskenderun, located on the Mediterranean coast with a population of approximately 500 000 residents, is situated approximately 100 kilometers away from the earthquake's epicenter and approximately 15 kilometers west of the earthquake fault. Iskenderun experienced relatively minimal structural damage compared to Kahramanmaras and Antakya. However, it did suffer significant damage due to geotechnical issues, primarily in areas with land reclamation. Unfortunately, there are no available strong motion records for Iskenderun.

Figure 2 displays a map of Iskenderun, where a survey was conducted on March 31st. The ground in Iskenderun predominantly consists of Quaternary alluvium containing silt and sandy layers, as observed in the Konarli area of Iskenderun Bay. This region has loose soil and a high water table [4]. Iskenderun, once a small town surrounded by marshland, underwent substantial development between the mid-19th century during the Ottoman Empire and the French Mandate period, lasting until the first half of the 20th century. Figure 2 illustrates the location of the lighthouse, which was originally at sea, and the coastal area beyond this point has been transformed into modern reclaimed land. After the earthquake on February 6th [5], it was reported that streets near the coast were inundated by seawater. During the authors' survey on March 31st, inundation was confirmed due to rainfall the previous day. Figure 2 also displays the inundated area during the survey, primarily in the reclaimed land area, with the coastal shopping street submerged by about 30 centimeters. Moreover, in the eastern part of the survey area, the inundation extended further inland. Local residents indicated that this area was once marshland, possibly resulting in more pronounced subsidence on the elevated ground in the eastern part. Numerous signs of sand ejecta were also observed in and around the inundated region.

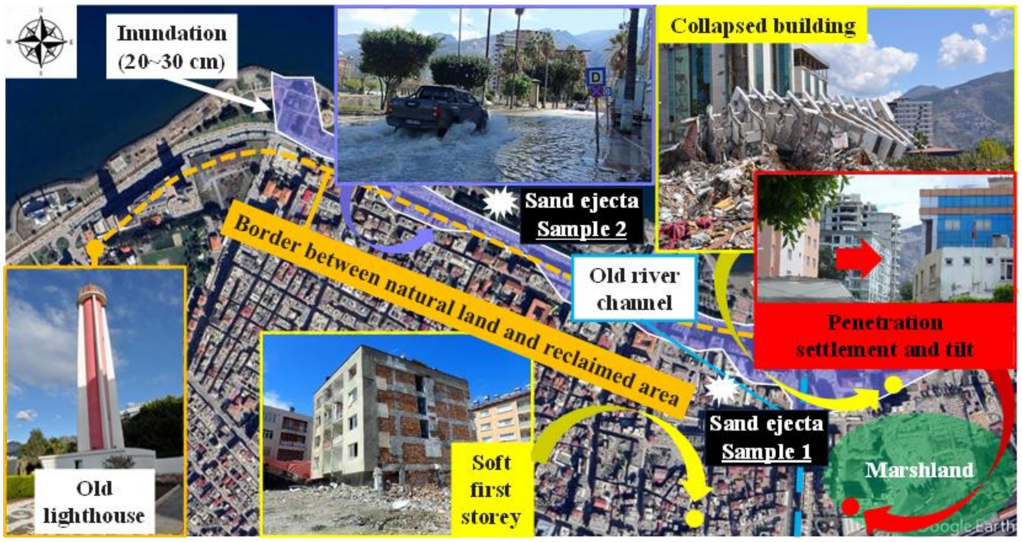

Figure 2. Reclaimed land boundary and inundation area in Iskenderun City observed in the authors' survey on 31st March, 2023

Figure 2 additionally illustrates damage caused by liquefaction, including slight building subsidence and significant sidewalk uplift. This street was previously a water channel [6], potentially making the reclaimed soil in the channel susceptible to liquefaction during the earthquake. Figure 3 and Figure 4 depict typical structural damage in Iskenderun. Figure 3 shows a mid-rise building in reclaimed land, where the entire structure has tilted significantly.

Figure 3. Evidence of various kinds of geotechnical problems observed in Iskenderun

Figure 4. Collapsed building located in the former marshland – near Hotel Ramada

Similar damage, characterized by tilting and settlement of buildings, is evident along coastal streets, albeit to varying degrees (Fig. 5 and Fig. 6). This damage is likely attributed to liquefaction of the reclaimed ground and loss of bearing capacity. In contrast, the structure shown in Figure 4, located in an area formerly marshland, collapsed in a twisting manner. Information on the distribution of structural damage patterns in Iskenderun City is currently limited, but it is observed that there was relatively more structural damage due to seismic motion inland from the reclamation boundary. Both geotechnical issues and structural damage occurred in the marshland area. Different surface ground characteristics and the presence or absence of liquefaction likely contributed to distinct patterns of structural damage.

Figure 5. Tilted middle-rise buildings near the sea-coast (at the beginning of the reclaimed land)

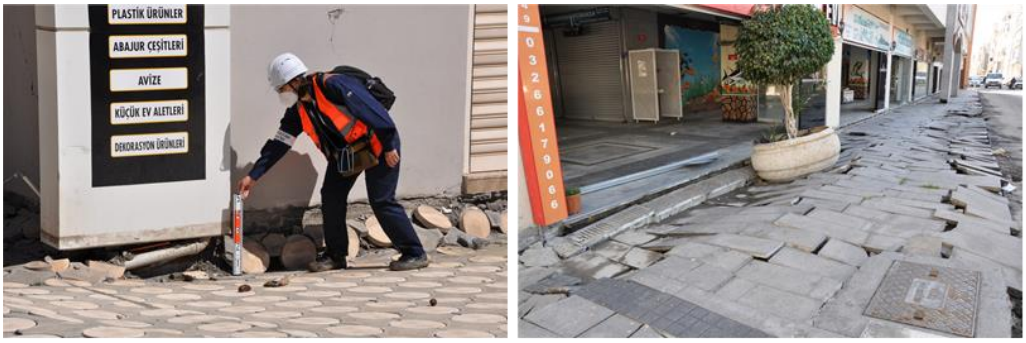

Figure 6. Settlement of buildings due to soil liquefaction

Figure 8 illustrates the reclaimed area where the promenade along the seashore subsided significantly, by approximately 50 centimeters, nearly aligning with the sea level. This subsidence likely led to the inundation of coastal streets by high waves. Observations also noted water flowing backward and emerging from sewage manholes on the inundated coastal street. The blockage of sewage pipelines due to liquefied soil deformation likely caused extensive inundation damage.

Figure 7. Location of performed tests in the city of Iskenderun (36°35'33.99"N, 36°10'6.98"E)

Figure 7 presents the exact location and Figure 11 results of the PDCPT at DP-3 and the multichannel analysis of surface wave (MASW) at SW-1. The PDCPT was conducted down to a depth of ground level -4.0 meters, revealing that the groundwater table was almost at the surface. The N_{SPT} values at the top surface were consistently low, less than 5, and slightly increased, ranging from 5 to 10, from ground level -1.0 meter to -2.0 meters. Below this depth, the soil layer exhibited loose characteristics, with N_{SPT} values averaging around 5. As shown in Figure 8, several cracks, each several tens of centimeters wide, were identified in the reclaimed land due to lateral spreading, and numerous sand ejecta were observed, indicating that coastal area land subsidence was induced by liquefaction. Calculating the distance of cracks from the seashore to be 60 meters inland, the lateral displacement was estimated to be as much as 1.5 meters toward the sea.

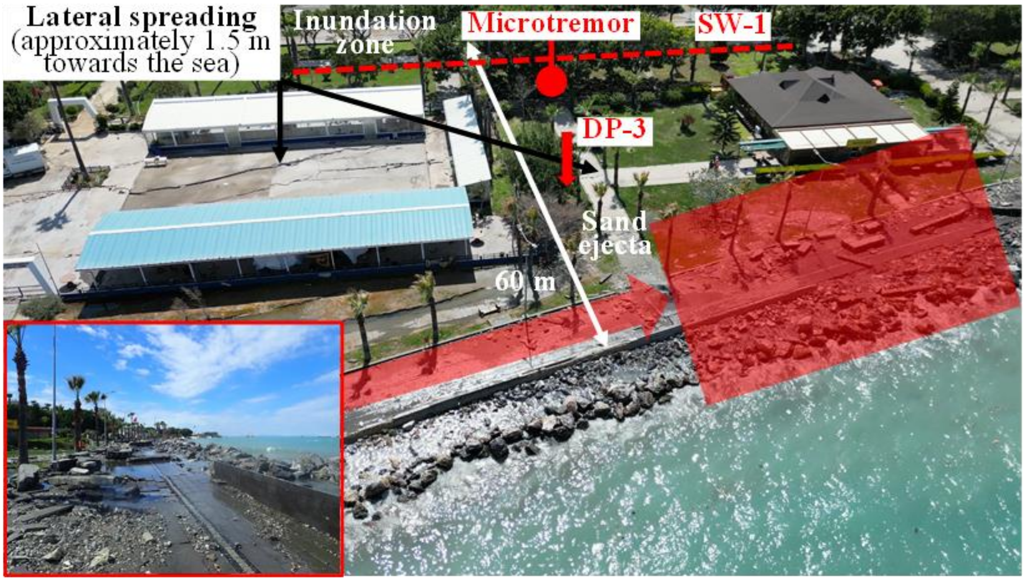

Figure 8. Liquefaction-induced damages in the reclaimed land (36°35'33.99"N, 36°10'6.98"E)

The shear wave velocity value of the surface layer from the MASW was consistently less than 150 m/s but gradually increased up to approximately ground level -10 meters, reaching about 190 m/s (Fig. 10). At greater depths, the shear wave velocity value remained relatively stable until about ground level - 25 meters. Given these findings, it can be reasonably concluded that the observed 50 cm subsidence along the shoreline was a result of liquefaction in the reclaimed ground up to approximately ground level -10 meters, especially considering the occurrence of lateral spreading. However, it should be noted that the inundation damage in Iskenderun City was more severe immediately after the earthquake. Further investigations are necessary to explore the causes of land subsidence in the city, which may not solely be attributed to liquefaction but also to crustal deformation effects.

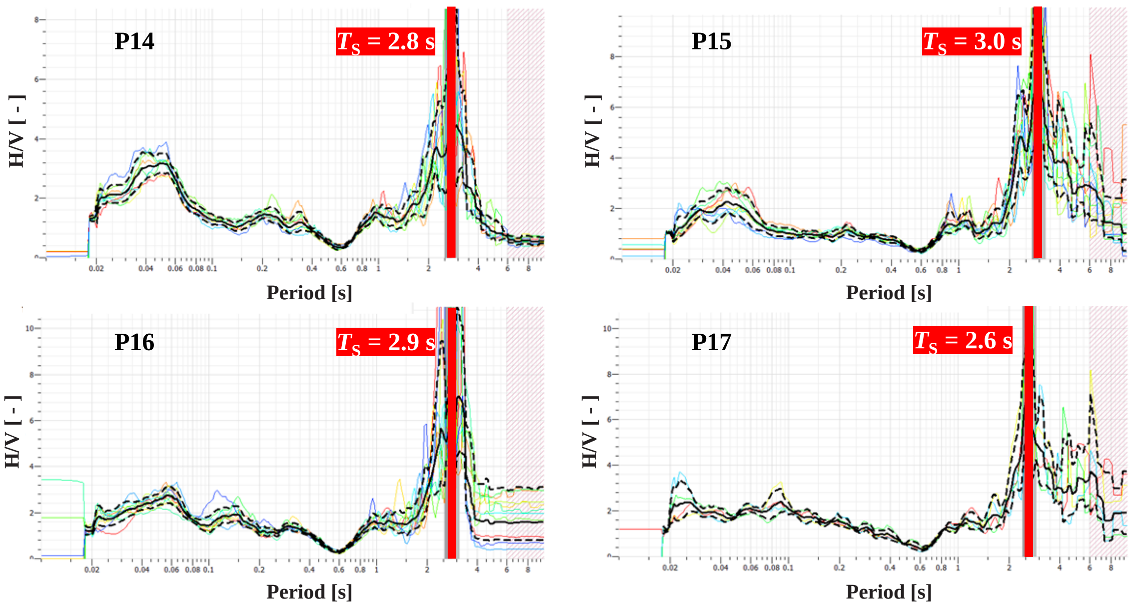

Figure 9. Results from microtremor tests – evaluation of predominant period of the soil profile at P14, P15, P16 and P17

During this survey, two distinct soil samples, referred to as Sample 1 and Sample 2, were collected, with their specific locations illustrated in Figure 2. Both samples, identified as sand ejecta, exhibit a specific gravity of 2.68. Notably, the particle distribution curves for these samples are remarkably similar, characterizing them as poorly graded and "highly liquefiable" soils. Moreover, a thorough evaluation of site-specific soil characteristics to advance our understanding of seismic site response was conducted. Cutting-edge geophysical techniques for direct measurements of key parameters were utilized. The MASW method was employed to determine the shear wave velocity (V_{s30}), representing the average shear wave velocity in the upper 30 meters of the soil medium at the study site. Additionally, the microtremor-based horizontal-to-vertical spectral ratio (H/V) method was employed in order to derive the predominant period of the soil profile (T_s), resulting in an average T_s value of 2.8 seconds (Fig. 9). These measurements provided critical input parameters for further seismic hazard analysis. Based on this data, the average depth (H) to a stiff layer with a shear-wave velocity greater than 800 m/s within the soil profile was estimated using the classic formula $H = (4 \times V_{s30}) / T_s$ [7], resulting in an estimated H value of approximately 130 meters. These findings offer valuable insights into site-specific seismic response characteristics, crucial for seismic hazard assessment and infrastructure design in the studied region. According to Eurocode 8, the classification of a soil profile based on its shear wave velocity in the upper 30 meters (V_{s30}) is a fundamental step in assessing seismic hazard. When V_{s30} falls into the Type D category, it indicates that the soil profile consists of relatively soft and potentially liquefiable soils. This classification holds significant implications for seismic risk assessment and structural design since Type D soil profiles are associated with increased ground motion amplification and heightened seismic vulnerability.

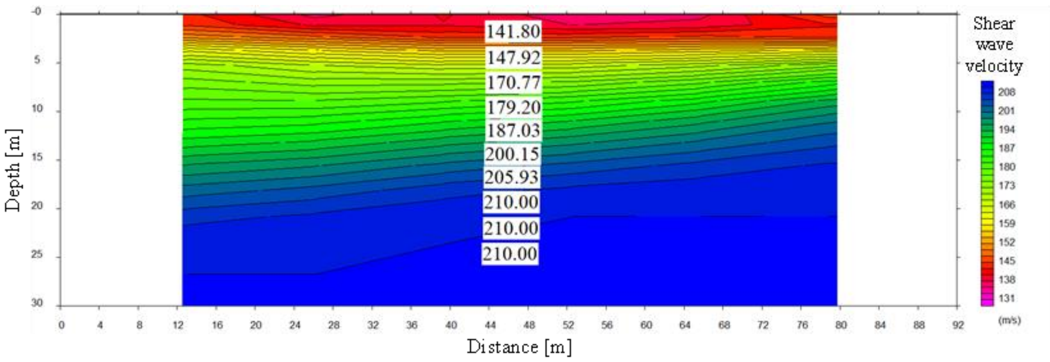

Figure 10. 2D shear wave velocity profile based on MASW measurements at SW-1

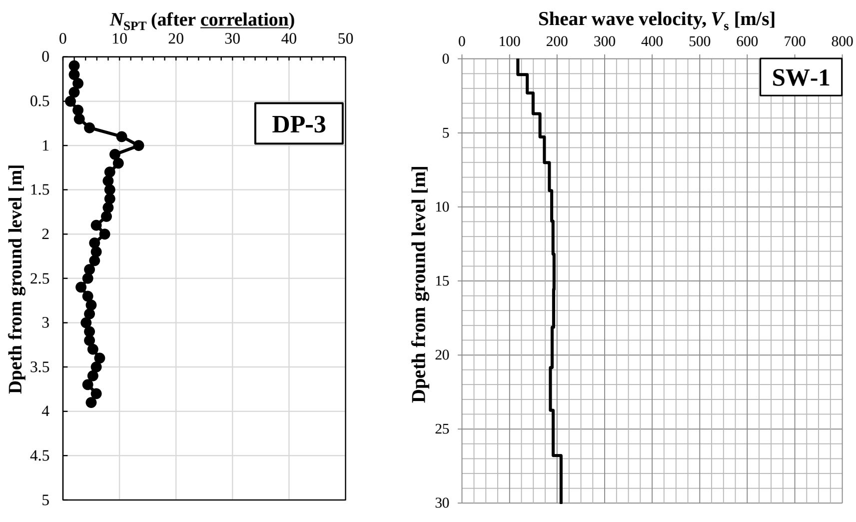

Figure 11. Results of PDCPT at DP-3 and MASW at SW-1

Liquefaction assessment within this study adhered to a robust deterministic framework, combining crucial data from two primary sources: 1) results obtained through PDCPT, converted to N_{SPT} [8,9, 10], and 2) shear wave velocity (V_s) profile [11,12]). The calculated factor of safety (FS) consistently revealed values below unity, indicating the susceptibility of the soil profile to liquefaction-induced phenomena. This susceptibility necessitated a meticulous examination of potential consequences, encompassing liquefaction-induced reconsolidation settlement and lateral spreading phenomena [13,14,15,16,17,18,19]. The results, as presented in Figure 13, portray a conservative outlook on the liquefaction potential. However, it is imperative to exercise discernment in interpreting these findings. Some layers within the soil profile may exhibit relatively low V_s values, potentially raising concerns regarding their liquefaction susceptibility. Nonetheless, a profound understanding of the physical properties (especially particle size distribution curve and plasticity parameters) of these specific layers might mitigate these concerns. Their intrinsic characteristics, such as high confining pressures, significant fines content, or cementation, effectively preclude them from active participation in the liquefaction response. Thus, the comprehensive liquefaction assessment underscores the importance of considering not only the quantitative data but also the qualitative aspects of soil behavior, ensuring a holistic understanding of site-specific seismic risk. Such knowledge is invaluable for seismic hazard assessment and resilient infrastructure design in the region.

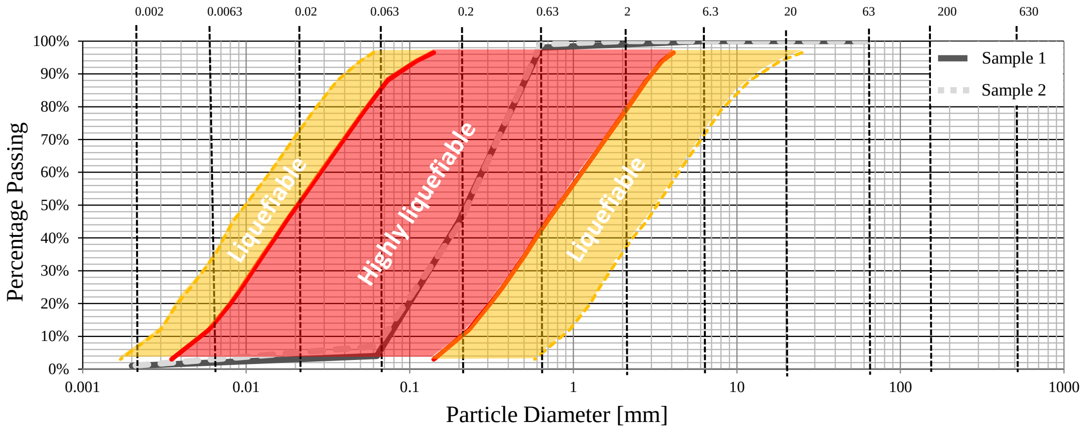

Figure 13. Particle distribution curves of sand ejecta (Sample 1 and Sample 2) collected from the city of Iskenderun – comparison with critical range associated with liquefaction

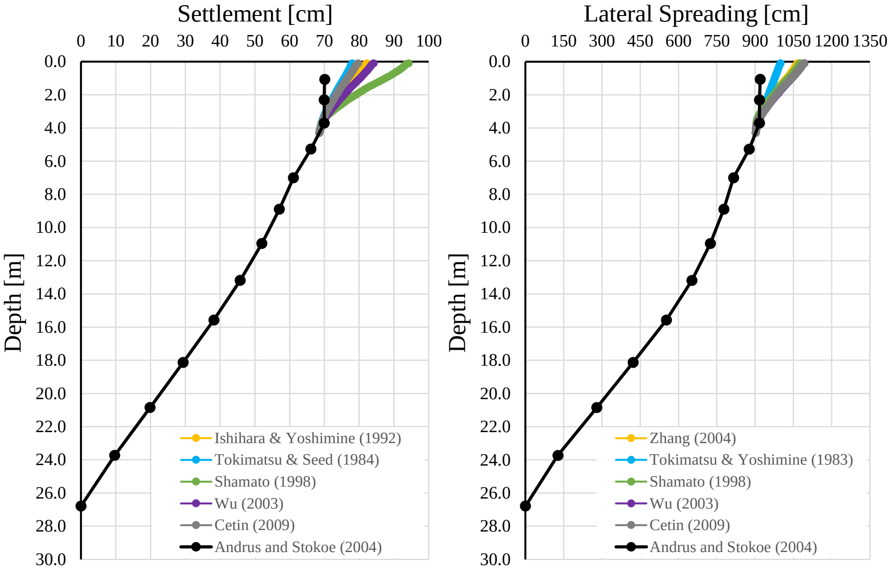

Figure 13. Evaluation of settlement and lateral spreading due to possible re-liquefaction in the city of Iskenderun by simplified procedures

GOLBASI TOWN

The Golbasi Basin is characterized as a pull-apart basin with a left-lateral strike-slip fault along the Golbasi-Turkoglu segment, situated on the East Anatolian Fault Zone (EAFZ). Golbasi Lake in this region resembles a sag pond. A study conducted by [20] revealed that the lithological units in the examined area primarily consist of Quaternary alluvial deposits (Qal), including clay, silt, sand, and gravel, along with swamp sediments (Qb). In terms of groundwater levels, they typically range from -0.65 meters to -3.5 meters, except in three boreholes where no water was detected. The dominant ground vibration period varies from 0.23 to 0.67 seconds, averaging at 0.48 seconds. As a result of this investigation, the region has been designated as a priority area for preventive measures. It has been categorized into two zones: OA1, where Qal formation prevails, primarily addressing earthquake hazards, and OA2, where Qb formation is dominant, focusing on mass movement hazards and high slopes.

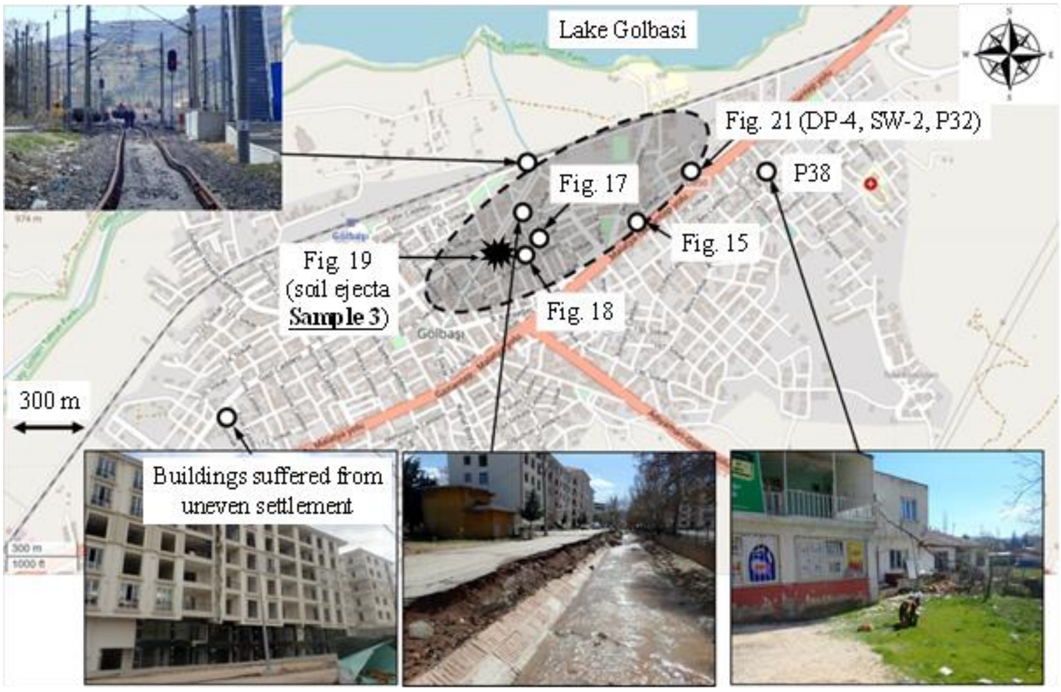

Figure 14. Extent of the liquefaction-induced damages in the town of Golbasi

Figures 16, 17, 18 and 19 depict typical building damages. Figure 16 shows the most severely tilted building identified during the authors' survey, leaning against an adjacent structure. The groundwater level at the foundation of this building, which exhibited pronounced tilting, was approximately at ground level -1.0 meter.

Figure 15. Tilted foundation of demolished building [5]

Golbasi, a town with around 35 000 residents situated approximately 85 kilometers northeast of the earthquake epicenter, experienced damage due to liquefaction. This led to settlement and tilting issues in many mid-rise buildings. The damage was particularly severe in the southern part of Golbasi, adjacent to Lake Golbasi. This substantial damage was attributed to loose lake sediments and the prevalence of shallow foundations in most buildings. For instance, Figure 15 illustrates the foundation of a five-story building, which initially had a 3-meter-deep foundation but was completely overturned by the earthquake by the time of the authors' survey on April 1st. The remaining mat foundation measured approximately 0.8 meters in thickness. It is likely that many mid-rise buildings in Golbasi had similar shallow foundation types. Numerous structures in this area suffered from settlement and tilting damage due to insufficient bearing capacity of the soil, especially along the demarcated area near the lake on the main road (D360), as depicted in Figure 14. This region also witnessed damage to the water channel's retaining wall and subsidence of the railway track.

Aerial imagery in Figure 16, captured from a northwest perspective, reveals significant subsidence and tilting damage affecting nearly all mid-rise buildings within the frame. Figure 17 features a building that experienced extensive settlement, subsiding by approximately 1.5 meters, causing cars to become wedged into the parking space's ceiling. Additionally, Figure 18 showcases notable settlement of mid-rise buildings on both sides of a narrow road, accompanied by substantial uplift of the road surface. These damages correspond to significant ground deformations, with minimal structural damage to walls and columns. However, Figure 19 illustrates a severely damaged building where the ground floor's walls and columns failed due to seismic motion, subsequently sinking into the soil. According to interviews with local residents, one building (green and white) reputedly had a pile foundation and incurred substantial settlement and tilting damage, as illustrated in Figure 19. Specifics regarding this building's pile foundation remain unclear at present.

Figure 20 and Figure 21 display the locations and Figure 23 demonstrates the results of PDCPT (PD-4) and MASW (SW-2) assessments conducted near a heavily subsided and tilted building. The N_{SPT} values were exceptionally low, registering less than 5 up to ground level -3.0 meters and gradually increasing beyond that depth. Conversely, the shear wave velocity values exhibited minimal values of 130-140 m/s up to ground level - 7.0 meters, followed by a slight increase with depth. Loose soils extended continuously down to ground level - 20 to - 25 meters. These findings suggest that the shallow surface soil layer, characterized by liquefaction and loss of bearing capacity, led to considerable settlement and tilting in structures. Borehole data from the city of Golbasi has been analyzed [20], revealing that the lake-side region adjacent to the main road (D360) predominantly comprised soft clay. Nevertheless, same study also identifies pockets of medium-dense sand and gravelly soils, which align relatively closely with the area demarcated by the broken line in Figure 14. This implies that significant liquefaction may have occurred in these medium-dense sand or gravelly soil sections.

Figure 16. Heavily tilted building and surrounding buildings with similar damage

Figure 17. Building suffered from subsidence of about 1.5 m – before and after the earthquake

Figure 18. Road uplift and settlement / tilting damage to buildings on either side of the road

19. Structural damage (soft first storey) – blue building, and settlement /
tilting damage of a building (green and white) that reportedly had pile foundations

Furthermore, the ejected soil collected near the green and white building in Figure 19 primarily consisted of reddish-colored silt and clay, characterized by a substantial plasticity index (*PI*) of 20. According to liquefaction assessment standards established by JRA and AIJ (REF), soils with a plasticity index exceeding 15 are deemed unlikely to liquefy and are thus exempt from liquefaction analysis. This suggests that liquefaction may have taken place at the source of the ejected clayey soil, or alternatively, the surface clayey soil could have been entrained in the ejected water resulting from liquefaction in the underlying sand or gravelly soil layers, subsequently surfacing.

Figure 20. Location of performed tests in the town of Golbasi (37°47'26.57"N, 37°39'7.88"E)

While this survey included PDCPT and MASW evaluations conducted immediately adjacent to the main road, confirming the thickness of the soft surface layer, it is conceivable that the thickness of the soft layer may increase closer to the lake. Additionally, subsidence and tilting damage to buildings resulting from ground deformation were observed on the western boundary of Golbasi, as indicated on the map in Figure 14. Consequently, further investigations are imperative to ascertain the liquefied soil layer and its distribution in the future. Figure 24 illustrates particle size distributions for soils typically associated with liquefaction, and the envelope of soil considered susceptible to liquefaction [21]. Samples from soil ejecta were collected right next to green and white building on Figure 19, and particle size tests were conducted in the laboratory. The resulting particle size distribution curve was then compared to the particle size distributions for soils typically associated with liquefaction. It is evident that a significant portion of the curve falls within the “highly liquefiable” region. Moreover, the evaluation of liquefaction susceptibility incorporates three established procedures: those proposed by [22,23] and the criteria outlined in the Japanese Road Bridge Design Standards. These methods are based on analyzing fundamental physical soil properties, including the Plastic Index (*PI*), Liquid Limit (LL), and Fines Content (*FC*). According to these simplified assessment techniques, the soil in the Golbasi region is classified as “moderately susceptible to liquefaction” – Figure. 25 and Figure. 26. This categorization is critical, emphasizing the need for vigilance, especially given the severe damages in Golbasi associated with such soil classifications.

Figure 21. Locations of PDCPT, MASW and microtremor measurement conducted near buildings with subsidence / tilting damage (37°47'26.57"N, 37°39'7.88"E)

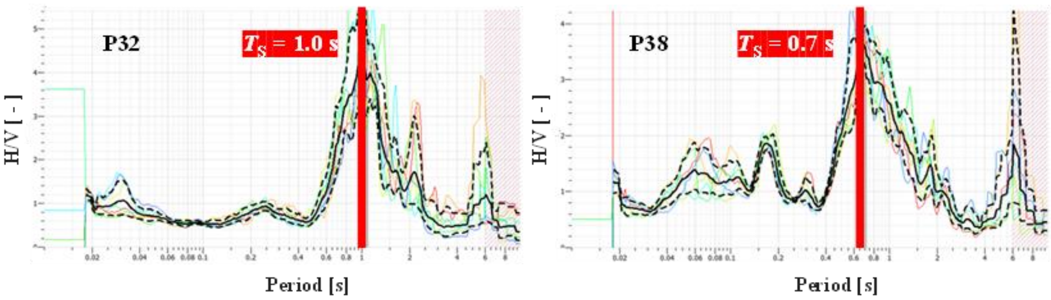

Figure 22. Results from microtremor tests – evaluation of predominant period of the soil profile at P32 and P38

In parallel measurements conducted in the city of Iskenderun, a similar suite of tests to assess site-specific soil characteristics was employed. The shear wave velocity (V_{s30}) in Golbasi was determined to be approximately 170 m/s, and the predominant period of the soil profile (T_s) was measured at 1.0 second, with a specific focus on point P32, a location associated with numerous liquefaction occurrences (Figure 22). The estimated depth (H) to a stiff layer with a shear-wave velocity exceeding 800 m/s within the soil profile was found to be around 43 meters. The outcome of this evaluation led to the categorization of the soil as Type D in the context of Eurocode 8, indicating the existence of relatively soft and potentially liquefiable soils. This classification holds notable consequences for seismic risk assessment and structural design, primarily due to the elevated levels of ground motion amplification and heightened seismic vulnerability typically associated with Type D soil profiles.

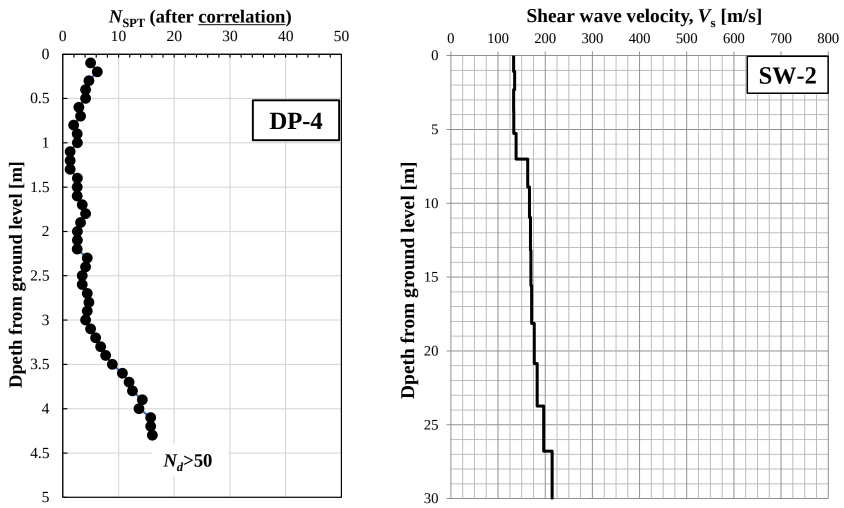

Figure 23. Results of PDCPT at DP-4 and MASW at SW-2

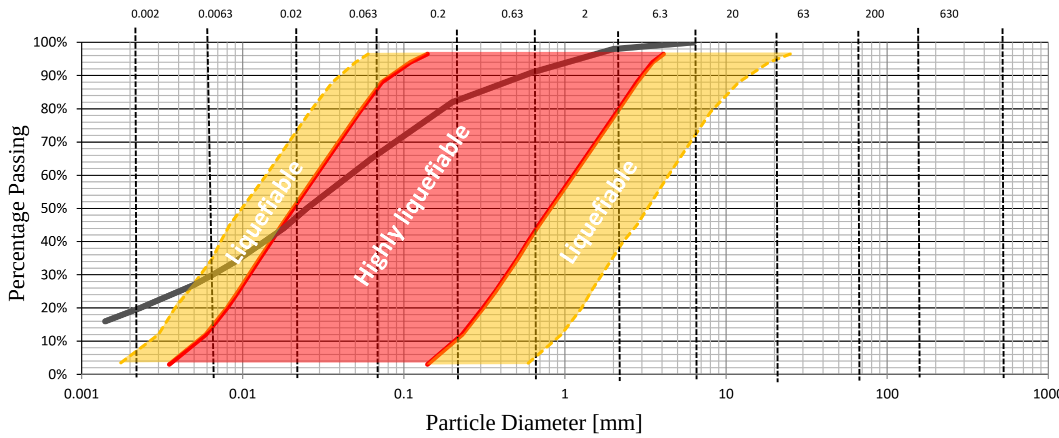

Figure 24. Particle distribution curve of soil ejecta (Sample 3) collected from the town of Golbasi – comparison with critical range associated with liquefaction

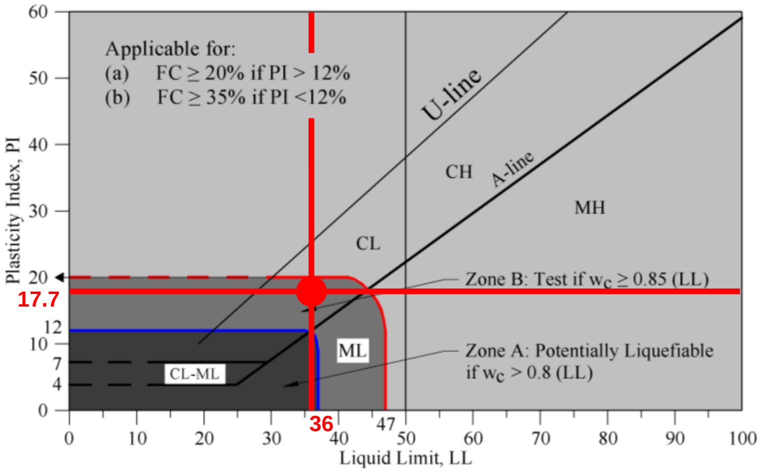

Figure 25. Sample 3 (Golbasi soil ejecta): Liquefaction susceptibility evaluation by adopting the procedure of (Seed et al., 2003)

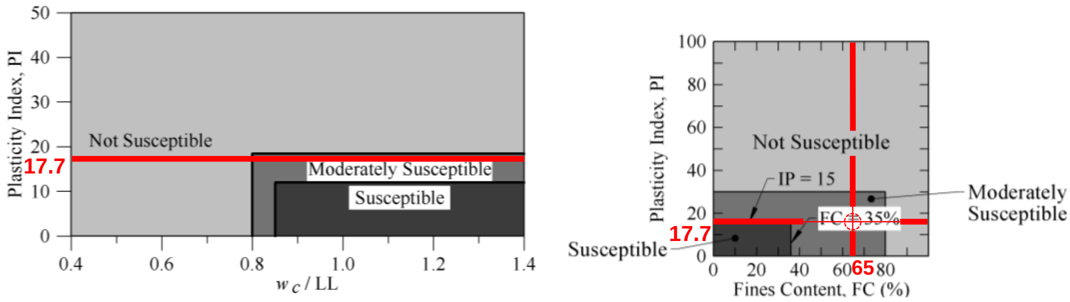

Figure 26. Sample 3 (Golbasi soil ejecta): Liquefaction susceptibility evaluation by adopting the procedure of (Bray & Sancio, 2008), left, and Japanese Road Bridge Design Standards, right

The calculations for the Golbasi site mirrored those for Iskenderun, reaffirming the re-liquefiable nature of the site with a factor of safety (*FS*) consistently below 1.0, following the deterministic approach. This comprehensive assessment encompassed the evaluation of potential liquefaction-induced reconsolidation settlement and lateral spreading (Figure. 27). It is worth emphasizing that the obtained values, while relatively substantial, stem from the limited knowledge of the soil's physical properties. Nonetheless, these cautious estimations emphasize the imperative for more in-depth investigations.

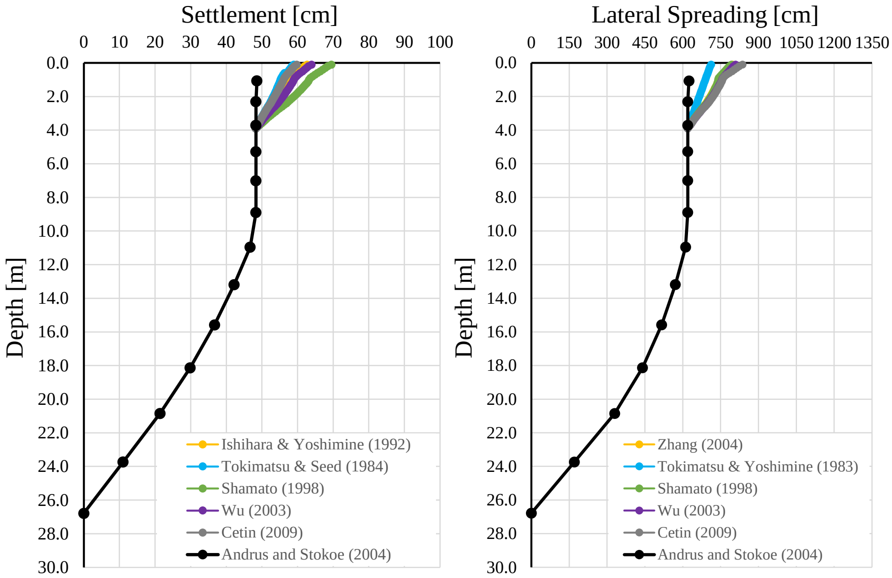

Figure 27. Evaluation of settlement and lateral spreading due to possible re-liquefaction in the town of Golbasi by simplified procedures

The collection of soil samples and the subsequent determination of their physical properties represent a significant step towards a more comprehensive understanding of the geotechnical characteristics in the affected regions. Notably, the determination of a plasticity index (*PI*) within the range of 15 to 20 provides valuable insights into the soil's behavior under seismic loading conditions. Moreover, with direct measurements of average shear wave velocity (V_{s30}) using the multichannel analysis of surface waves (MASW) method and the assumed bulk density (ρ) in the range of 1800 to 2000 kg/m³, it becomes possible to make preliminary estimations of critical parameters. One such parameter is G_{max}, the maximum shear modulus of the soil ejecta, which equals the product of bulk density and shear wave velocity squared, resulting in G_{max} values ranging from 50 to 55 MPa in this context ($G_{max} = \rho \times V_s^2$). Drawing from the work of [24], stiffness degradation curves and damping ratios can be roughly assumed, further enhancing our capacity to model the soil response during seismic events (Figure. 28).

These additional details derived from the physical properties of the soil samples contribute to a more accurate and refined assessment of liquefaction potential and seismic hazard, facilitating the development of robust mitigation strategies for the impacted regions.

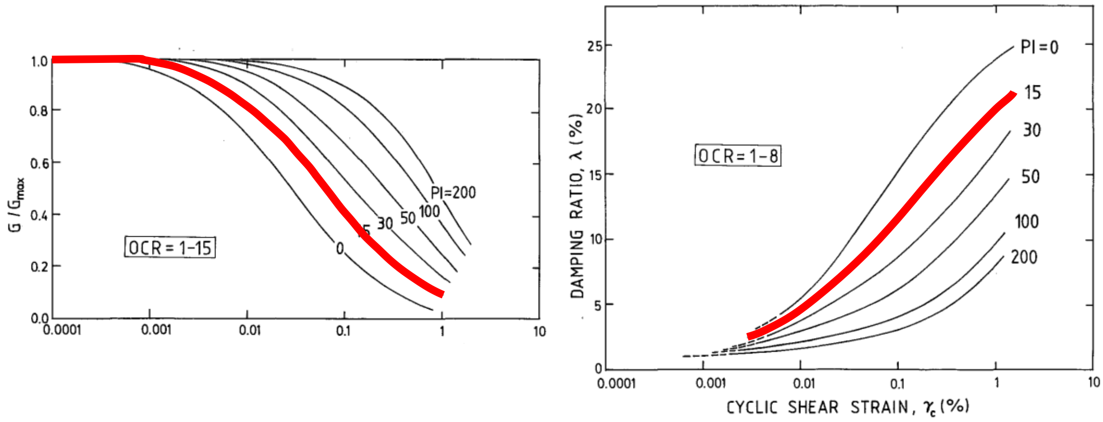

Figure 28. Assumed stiffness degradation and damping ratio in relation with shear strain – tested soil ejecta – after [24]

The stiffness degradation curve and damping ratio play a crucial role in equivalent linear seismic analysis, providing a simplified yet effective approach to model soil behavior during seismic events. By incorporating these parameters into the analysis, it could be understood how the soil responds to varying levels of ground shaking. This, in turn, allows for the estimation of ground motion amplification and the evaluation of structural performance. Equivalent linear analysis is essential because it strikes a balance between accuracy and computational efficiency, making it a valuable tool for seismic hazard assessment and the design of resilient structures. By considering the stiffness degradation curve and damping ratio, engineers and researchers can make informed decisions regarding earthquake-resistant design and mitigate the potential risks associated with liquefaction-induced ground motion.

CONCLUSIONS

In response to the significant seismic events that occurred in Turkey and Syria on February 6th, 2023, a collaborative reconnaissance team was established. This team comprised researchers and engineers from Japan, Turkey, and Bulgaria, representing organizations such as the Japan Association for Earthquake Engineering, the Architectural Institute of Japan, the Japan Society of Civil Engineers, and the Japanese Geotechnical Society, Bogazici University, MEF University, and the University of Architecture, Civil Engineering and Geodesy (UACEG). Their joint efforts led to an extensive on-site investigation conducted from March 28th to April 2nd, 2023, approximately two months after the devastating earthquakes.

During this investigation, several noteworthy findings emerged from various affected areas:

- Iskenderun faced not only building collapses on soft ground but also instances of building tilting and ground subsidence attributed to the liquefaction of reclaimed coastal soil.
- Golbasi witnessed significant damage caused by liquefaction to structures with shallow foundations on soft ground, leading to tilting and settling. However, a more comprehensive investigation is deemed necessary to accurately delineate the extent of liquefied soil layers.

These findings collectively represent the outcome of the collaborative efforts of the reconnaissance team, shedding light on the diverse effects of the earthquakes and the geotechnical challenges faced in the impacted regions.

The initial outcome from the investigations following the seismic events in Turkey and Syria in 2023 highlight the urgent need for further studies in the affected regions. The identification of liquefaction-induced damage in areas like Iskenderun and Golbasi underscores the significance of assessing the re-liquefaction potential. To better comprehend and address the geotechnical challenges posed by these earthquakes, it is crucial to conduct more comprehensive investigations that delve into the specific physical properties of the soil. By obtaining a detailed understanding of the soil's characteristics, it becomes possible to differentiate and potentially exclude certain soil layers from the liquefaction susceptibility analysis. This, in turn, can inform more accurate risk assessments and contribute to the development of resilient engineering solutions for the affected regions. Hence, additional studies hold the key to mitigating future seismic risks and ensuring the safety and stability of infrastructure in earthquake-prone areas.

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ENVIRONMENT

The impact of landfills on underground and surface water

Editors

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EFFECTS OF LANDFILL LEACHES ON GROUND AND SURFACE WATERS: A CASE STUDY OF A WILD LANDFILL IN EASTERN BOSNIA AND HERZEGOVINA

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ABSTRACT

Landfill leachate, due to its high total pollution, and above all due to their high organic pollution, represents a significant environmental problem. This study investigated the impacts of the Tilić ada landfill on ground and surface waters. The location of the landfill Tilić ada is extremely sensitive due to the fact that it is located right next to the Drina river bed on the border with the Republic of Serbia, and especially because the source of drinking water Tilić ada is located at a distance of approx. 500 meters.

Therefore, analyses of impact of leachate from the landfill were carried out, which indicated that the groundwater is at risk even 5 years after the landfill was closed. At the same time, water analyses from the Drina River were also carried out, which showed that the water quality was not impaired in relation to the defined water class.

Keywords**:** *landfill, leachate, groundwater, surface water*

INTRODUCTION

The Tilić ada landfill was used for the disposal of collected mixed municipal and non-hazardous waste from the wider and narrower urban areas of the City of Zvornik in the ten-year period between 2007 and 2017 at the location of the old gravel pit. Its estimated area is approximately 2.5 ha. Given that it is a wild, unorganised, unsanitary landfill, no technical documentation based on the principles of sustainable waste management has been prepared, that is to say necessary measures were not implemented to reduce the hazardous environmental impact of the waste during its disposal at the location. No stormwater drainage system was built, and leachate went and continues to go underground in an uncontrolled manner.

The location of the Tilić ada landfill is extremely sensitive due to the fact that it is located right next to the Drina River bed on the border with the Republic of Serbia, and especially because Tilić ada potable water spring is located upstream of the landfill at a distance of approx. 500 metres. [1]. This is the reason why the landfill's impact on underground and surface water was analysed 5 years after its closure.

Landfill leachate, due to its high total pollution, represents a significant environmental problem [2,3]. Previously, a large number of studies were conducted that pointed to the leachate of waste disposal sites on the environment [4,5,6,7].

The chemical characteristics of leachate are affected by biological decomposition of biodegradable organic substances, chemical oxidation processes and dissolution of organic and inorganic substances in waste [8,9]. Landfill leachate is produced by percolation of rainwater through the landfill body, during which soluble, colloidal and suspended substances are extracted. In other words, landfill leachate is a polluted liquid, which seeped through the layers of deposited waste and thereby absorbed large amounts of pollutants, including the products of chemical and biochemical reactions that take place in the landfill body. Leachate consists of liquids that enter the landfill body from the outside, that is to say from precipitation, infiltrated groundwater, as well as water contained in the waste itself [10].

Total quantity of the landfill leachate consists of the quantity of external water that entered the landfill body and the quantity of internal water in the landfill. External waters that can reach the landfill are: rainwater from the catchment area, surface catchment waters, precipitation and ground water.

Part of the water also enters the landfill body directly with the waste (internal water), through the moisture of the waste being deposited. In untreated municipal waste, the moisture ranges between 20 and 60 wt %. However, as a rule, the moisture of untreated municipal waste is below the saturation point, so that on average the deposited waste can accept about 12 vol % of additional moisture.

The main source of landfill leachate is precipitation that comes to the landfill surface and seeps through the landfill body. Part of this water runs off as rainwater from the landfill, part returns to the atmosphere by evaporation from the upper landfill surface or vegetation (evapotranspiration), while the rest is leachate that occurs after the waste reaches full moisture saturation. The climate has a significant effect on the rate of leachate formation, as the quantity of this water is much higher in the high rainfall zone than in the low rainfall zone. Soil topography affects the direction of the water flow, as well as the quantity of water entering and leaving the landfill zone. The permeability of soil interlayers located in the landfill will affect the rate of downward movement of water. Leachate quantity decreases with increased surface water runoff, more intense evaporation of water from the landfill surface and a decrease in moisture in the overlying soil layers.

Given the identification of an unorganized landfill with different quantities of waste, type of waste, disposal period, composition and slope of the terrain, etc., where no monitoring is carried out, the quantity of leachate can be calculated in accordance with the following formula:

$$Qf = \frac{k\ x\ P}{30} \tag{1}$$

where:

Qf - daily amount of filtrate (leachate), m^3/day;

k - infiltration coefficient = 0.7 (for landfills on flat terrain) and 0.5 (for landfills on steep terrain);

P - total monthly amount of atmospheric precipitation on the given landfill area, m^3.

The infiltration coefficient is high due to the lack of data on evapotranspiration, surface runoff, the amount of moisture in the waste, the clear composition of the waste, and represents a certainty in the calculation.

A municipal solid waste landfill can be viewed as a biochemical reactor, with waste and water as inputs, and biogas and leachate as the main output components. The composition of leachate varies during the landfill exploitation and the most important factors that influence variations in the composition of leachate are [11]:

- composition of waste and its variability, which determine the rate of decomposition.;
- temperature in the landfill body oscillates according to the season and affects the growth of microorganisms and the rate of chemical reactions.;
- thickness of the deposited waste layer: thick layers of waste need more water to saturate, so the decomposition process takes longer.

The composition of leachate is particularly affected by landfill age. As the landfill ages, the concentration of organic substances decreases more than the concentration of inorganic substances, because they are decomposed and washed away, while inorganic substances are only washed away. Moisture significantly affects the degree of waste decomposition, considering that it helps with the exchange of substrates, nutrients, dilution of inhibitors and the growth of microorganisms. The most important influence on the moisture content in the landfill is the landfill construction method, the method of waste disposal and the climate. Leachate from municipal waste landfills generally contain the following impurities [12]:

- nitrogen compounds in organically bound form and in the form of ammonia: represents the highest percentage of soluble nitrogen in leachate landfill waters and is formed during the biodegradation of organic substances present;
- phosphorus compounds: they are involved in many physical, chemical and microbiological transformations. Phosphorous species are most often used in microbiological processes, complexation and dissolution.;
- heavy metals: certain concentrations of the following heavy metals occur in most filtrates from municipal landfills: Al, As, Cu, Ba, Fe, Zn, Cd, Co, Ag, Pb and Hg;
- cations: the most common cations that occur in leachate are: Na^+, K^+, Mg^{2+}, Ca^{2+}. They react with each other and with cations in waste complexes, creating complexes;
- anions: Cl^-, SO_4^{2-}, S^{2-} and HCO^{3-} are only partially transformed.;
- organic pollution: expressed through non-specific parameters BOD_5, COD and TOC;
- chlorinated hydrocarbons and pesticides;
- specific organic compounds: aromatic hydrocarbons, phenols, chlorinated aliphatic compounds, which are usually found in traces.

The filtrate is characterised by the following properties [13]:

- colour dark brown to black, unpleasant smell;
- pH in young landfills is acidic, and in old landfills basic (pH = 5.3 - 9.1);
- BOD_5 and COD very high during the acid fermentation phase, and significantly lower during the methane fermentation;
- the content of heavy metals in the acid fermentation phase is relatively high, and during methane fermentation it is almost negligible;
- relatively high chloride content in the acid fermentation phase;
- high ammonia content;
- very low phosphorus content.

The water contained in solid waste, as well as the water that infiltrates the landfill, form a medium in which all soluble substances are dissolved and which causes the movement of unreacted material downwards, towards the bottom of the landfill [14,15]. These waters are known as leachate. Landfill leachate is a medium whose composition and quantity change significantly during the landfill lifetime [16,17]. Filtrates from waste disposal sites are among the most problematic types of waste water from the aspect of toxicity. Research has shown that each landfill represents a separate system and that in this sense, the composition and amount of leachate depends exclusively on the characteristics of the landfill itself ([18,19,20].

In order to more reliably assess the expected composition of leachate from this wild and unregulated landfill, the results of leachate impact on groundwater and surface water were analysed.

MATERIAL AND METHODS

Sampling and physical and chemical analysis of leachate at the Tilić ada landfill location was carried out on November 2022. Sampling was carried out at the following locations (Figure 1.):

- three deep piezometers with a depth of 15 metres installed directly next to the landfill body (marked PB-1, PB-2 and PB-3),
- two deep piezometers with a depth of 10 metres installed in the landfill body (marked PB-4 and PB-5)
- two shallow piezometers with a depth of 6 metres installed in the landfill body (marked PB-6 and PB-7).

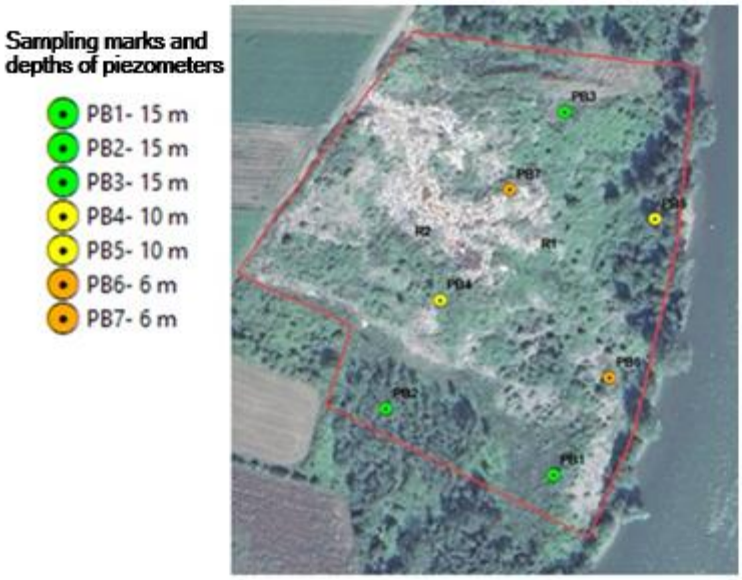

Figure 1. Position of piezometers from which groundwater sampling was performed

Sampling and physical and chemical analysis of the Drina River water quality was carried out at three locations on November 2022. Sampling was carried out at the following locations (Figure 2.):

- upstream of the landfill location (marked D1),
- directly next to the landfill location (marked D2)
- downstream of the landfill location (marked D3).

Figure 2. Sampling sites of the Drina River water

The analysis of water quality from piezometers and the Drina River included the following parameters: pH, electrical conductivity (EC), total dissolved solids (TDS), total suspended solids (TSS), biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total nitrogen, total phosphorus, sulphates, chlorides, lead (Al), iron (Fe), manganese (Mn) and aluminium (Al). Four metals were chosen because of their availability in landfill leachates All parameters were analysed in accordance with the current water regulation [21,22]. The classification of surface waters according to the current water regulation is shown in the Table 1.

Table 1. The classification of surface waters with maximum permissible concentration (MPC)

Parameter	**Unit of measure**	**MPC for surface waters**				
		Class I	**Class II**	**Class III**	**Class IV**	**Class V**
pH	pH units	6.80-8.50	6.80-8.80	6.5-9	6,5-9,5	<6,5;>9,5
EC at 20°C	μS/cm	<400	400-600	600-800	800-1500	>1500
TDS	g/m^3	<300	300-350	350-450	450-600	>600
TSS	g/m^3	<2	2-5	5-10	10-15	>15
BOD_5	$g\ O_2/m^3$	<2	2-4	4-7	7,0-15	>15
COD	$g\ O_2/m^3$	<12	12-24	22-40	40-50	>50
Total nitrogen	$g\ N/m^3$	<1	1-6	6-12	12-30	>30
Total phosphorus	$g\ P/m^3$	<0.01	0.01-0.03	0.03-0.05	0,05-0,100	>0,100
Sulphates	g/m^3	<50	50-75	75-100	100-150	>150
Chlorides	g/m^3	<20	20-40	40-100	100-200	>200
Pb	mg/m^3	<0.1	0.1-0.5	0.5-2.0	2,0-5,0	>5,0
Fe	mg/m^3	<100	100-200	200-500	500-1000	>1000
Mn	mg/m^3	<50	50-100	100-200	200-400	>400
Al	mg/m^3	<20	20-50	50-200	200-500	>500

All leachate samples were collected in 5l polypropylene carboys, transported to the laboratory, stored at 4°C and analyzed within 3 days. The samples were collected, preserved, and analyzed according to the Standard Methods for the Examination of Water and Wastewater:

- Water quality - Sampling - Part 1: Guidance for designing sampling programs and techniques sampling BAS EN ISO 5667-1:2008*, BAS EN ISO 56671/Cor1:2008
- Water quality - Sampling - Part 3: Preservation I handling of water samples BAS EN ISO 5667-3:2019
- Water quality - Sampling - Part 6: Guidelines for water sampling from rivers and streams BAS ISO 5667-6:2017
- Water quality – Sampling – Part 11: Guidelines for groundwater sampling BAS ISO 5667-11:2010.

On-site measurements of pH and electrical conductivity (EC) were conducted during the sampling process using a digital pH meter and a digital EC meter. Specifically, for heavy metal analyses, samples were individually gathered in pre-washed polypropylene containers with a capacity of 100 ml. To prevent metal precipitation, the samples were acidified on-site. Physical-chemical parameters such as total dissolved solids (TDS), major anion such as chlorides (Cl-) of leachate and groundwater samples were analysed by titrimetric methods. Chloride is included in water quality assessment because it measures the extent of dispersion of leachate into the groundwater body [23]. Sulfates in groundwater samples were analyzed by nephelometric turbidity method.

Nitrates and determination of total organic carbon (TOC) in groundwater samples was performed using a spectrometer. The assessment of chemical oxygen demand (COD) was done by closed reflux titrimetry, while biochemical oxygen demand (BOD) was calculated by determining oxygen by Winkler titration for a preserved leachate sample. Heavy metals such as Mn Pb concentrations in leachate and groundwater samples were analyzed using an atomic absorption spectrophotometer.

RESULTS AND DISCUSSION

Discussion of physicochemical parameters of groundwater

Physicochemical parameters of groundwater quality in the investigated area were determined on water samples from 7 piezometers located as mentioned earlier. The results of testing water quality from piezometers PB-1, PB-2 PB-3, PB-4, PB-5, PB-6 and PB-7 are presented in Table 2. The obtained measurement results were compared with the wastewater discharge limits.

According to literature data, the pH of the leachate varies depending on age of the landfill [5]. As a rule, leachates from younger landfills show lower values (less or more acidic), while with age, the pH value moves towards basic values. As we can see from Table 2., pH values of the tested samples range from 7.35 to 7.60, indicating that leachate are in intermediate or semi-matured stages.

Table 2. Comparative presentation of test results and limit values prescribed by the Rulebook [21]

Parameter	Unit of measure	Result							LV–wastewater discharge limits
		PB-1	PB-2	PB-3	PB-4	PB-5	PB-6	PB-7	
pH	pH units	7.41	7.35	7.48	7.45	7.40	7.40	7.60	6.5-9.0
EC at 20°C	µS/cm	1326	469	2704	4610	1326	1201	5320	-
TDS	g/m^3	0.96	1.84	1.21	3.75	0.73	13.5	26.2	-
TSS	g/m^3	0.62	1.43	0.11	3.14	0.62	0.30	0.41	35
BOD_5	$g\ O_2/m^3$	1	4	7	6	1	2	50	25
COD	$g\ O_2/m^3$	8	12	16	12	6	6	72	125
Total nitrogen	$g\ N/m^3$	1.171	1.171	1.171	1.2	12.88	1.64	1.15	15
Total phosphorus	$g\ P/m^3$	0.027	0.1	0.089	6.80	0.022	0.14	1.36	3
Sulphates	g/m^3	0.0915	0.042	0.14	0.27	0.16	0.15	0.25	200
Chlorides	g/m^3	0.071	213	2.284	1668.5	1.07	142	2982	250
Pb	mg/m^3	n.d*	n.d	n.d	n.d	n.d	n.d	n.d	10
Fe	mg/m^3	n.d	40	821	1800	710	130	3350	2000
Mn	mg/m^3	10	380	1270	750	920	320	400	500
Al	mg/m^3	n.d.	10	40	n.d	80	80	250	1000

*n.d.-not detected

EC and TDS values are attributable to the presence of dissolved inorganic materials. Higher values of EC, especially on PB-7, agree with the highest values of dissolved inorganic materials in that sample (chlorides, Fe).

In the current study, BOD ranged between 1 and 50 g/m^3 and COD values ranged between 6 and 72 g/m^3. The BOD_5/COD ratios (0.13-0.69) confirmed the leachate semi-matured stages. Usually, landfill leachate with BOD_5/COD ratio less than 0.1 is considered to be toxic and are typical for younger landfills.

The chloride concentration in the leachate varied from 1000 to 8000 g/m^3 for young leachate samples and 1000–6,00 g/m^3 for stabilized samples. [23].

Heavy metals have significant impact on groundwater as well as surface water quality even if it found in traces amount. Pb was not detected in any sample examined, and the concentration of Al is below the limit value. The metal's concentration in the tested samples are mostly below the safe limit except for Fe (3350 mg/m^3 in PB7) and Mn (1270 mg/m^3 in PB3, 920 mg/m^3 in PB5 and 750 mg/m^3 in PB4). It can be noted that, especially in the case of Fe, the higher concentration values are in the piezometers located in the body of the landfill. Figure 3 shows Fe and Mn concentrations by sampling location.

Factors that affect groundwater quality are either natural or anthropogenic. The content of Fe and Mn in piezometer waters can be of natural origin and originate from the dissolution of rocks. Unfortunately, we did not have any data on the quality of groundwater in this area before the start of exploitation of this landfill. This is the reason why we cannot say with certainty that, especially in the case of enormously high concentrations of Fe and Mn, it is not possible to say with certainty that it is a purely anthropogenic factor.

However, bearing in mind that the highest concentrations of Fe were found in the samples from piezometers in the body of the landfill, we must not ignore the anthropogenic factor.

Moreover, the samples from shallow piezometer installed in the landfill body (PB-7) have a high value for the most parameters (EC, TDS, BOD_5, COD, chlorides, Fe, Al).

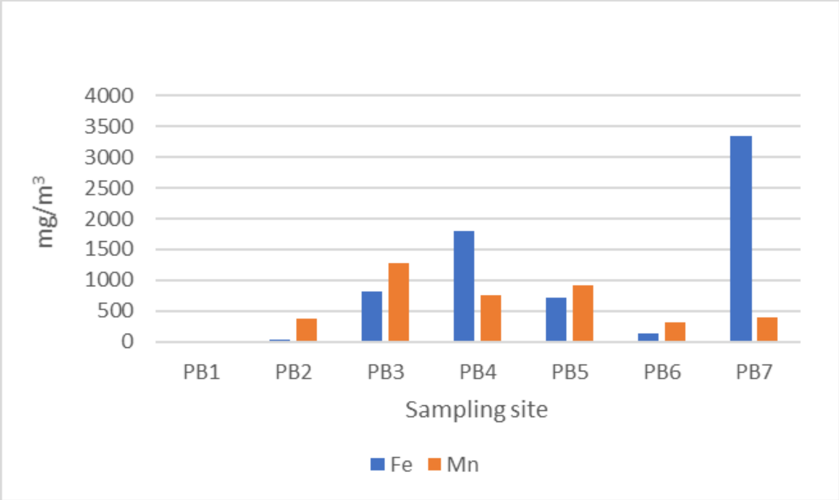

Figure 3. The concentrations of Fe and Mn by sampling location

Discussion of physicochemical parameter of surface water

Testing of the river Drina covered the basic groups of physical-chemical parameters of the water sample. The obtained measurement results were compared with the limit values, which are defined by the Regulation [22]. Results of the Drina River surface water analysis are shown in Table 3.

Table 3. Comparative presentation of the Drina River test results and the limit values prescribed by the Regulation [22]

Parameter	Unit of measure	Result/class of surface waters (Table 1.)		
		D1	D2	D3
pH	pH units	8.54/II	8.34/I	7.43/I
EC at 20°C	µS/cm	297/I	302.15/I	237/I
TDS	g/m^3	0.25/I	0.22/I	0.22/I
TSS	g/m^3	0.13/I	0.13/I	0.21/I
BOD_5	$g\ O_2/m^3$	1/I	2/II	2/II
COD	$g\ O_2/m^3$	4/I	5/I	5/I
Total nitrogen	$g\ N/m^3$	1.155/I	1.1569/II	1.1533/II
Total phosphorus	$g\ P/m^3$	0.01/II	0.012/II	0.013/II
Sulphates	g/m^3	0.39/I	0.20/I	0.14/I
Chlorides	g/m^3	0.11/I	0.14/I	0.11/I
Pb	mg/m^3	n.d./I	n.d./I	n.d./I
Fe	mg/m^3	20/I	20/I	30/I
Mn	mg/m^3	n.d./I	n.d./I	n.d./I
Al	mg/m^3	n.d./I	n.d./I	n.d./I

*n.d.-not detected

Based on the analyses performed and the results obtained, it can be observed that the Drina River has a good quality of the quality class II. The Tilić ada landfill is located in an area that does not meet the prescribed criteria for a location where waste can be deposited, considering the engineering-geological characteristics of the terrain and the fact that it is located in the very Drina River bed, and that it is in a narrow zone of sanitary protection of Tilić ada spring. Waste is disposed of without operational guidelines regulating this field.

The location is also not fenced off, and during the period of exploitation, the types and quantities of disposed waste were not recorded, nor the compaction and covering of waste with inert material. A special problem is the complete absence of emissions control and prevention of leachate filtering through soil layers, as well as degassing through the biothorn system.

In addition to negative impacts on human health and the environment, generally speaking, all uncontrolled landfills are characterised by the problem of unpleasant odours and a negative aesthetic effect, because they are often located in close proximity to roads and populated areas [24,25]. The problem with this landfill is the occurrence of torrential floods, which consequently lead to the lifting of and carrying away large amounts of waste during the withdrawal of water from the landfill site, to which border municipalities in the Republic of Serbia located downstream from the landfill site are exposed.

Based on the above, it can be concluded that the unsanitary way of waste management does not entail great costs during the period of use of the site for waste disposal, but on the other hand, apart from being contrary to the legal regulations, it entails complex environmental impact and significant costs during rehabilitation and recultivation. In order to minimise harmful effects, landfills as sources of pollution require closure and rehabilitation, but certainly one of the biggest obstacles in developing countries is the lack of financial resources.

CONCLUSION

This study investigated the impacts of the Tilić ada landfill in eastern Bosnia and Herzegovina on ground and surface waters, as typical example of wild and unorganized waste disposal sites. The landfill was active for a ten-year period, and the research presented in this study were conducted five years after the landfill was closed. Physico-chemical parameters of water quality were analysed on samples of underground water from the body of the landfill and immediately next to it, as well as on samples of surface water of the Drina River.

The results presented in the paper showed high concentrations of Fe and Mn in groundwater taken from piezometers located in the body of the former landfill. Although the origin of these metals in groundwater cannot be defined with certainty due to the lack of data from earlier times, anthropogenic influence cannot be excluded.

The results obtained from the analysis of water samples of the Drina River are classified into appropriate classes of water to which a certain value belongs, according to the Regulation on water classification and watercourse categorisation. Based on the analyses performed and the results obtained, it can be observed that the Drina River has a good quality of the quality class II.

Wild and unregulated landfills are one of the biggest potential polluters of underground and surface waters. The results presented in this article show that the impact of landfill leachates does not stop even after their closure, especially if no remedial measures have been taken.

Considering the performed analysis and categorisation of the landfill in question, based on the risk to the environment and human health, it is necessary to take steps towards its rehabilitation and recultivation, and prioritise planning and finding funds for these needs.In addition, hydrochemical monitoring is a necessary measure to control groundwater pollution in the research area.

The findings of this study can be used as initial data for the next phase of the research, which will investigate the possible further impact of the landfill's leachates, as well as ensuring permanent hydrogeological monitoring.

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