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A Study on Methods to Enhance the Stability of Riverbanks in Seaport Areas

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ABSTRACT

In the construction of irrigation structures, especially the wharf structures, it is important to learn about the deformation and damage of river banks in the wharf area. From assessing the stability of the river banks, a solution can be proposed to enhance the stability of the seaports, ensuring their safety during operation. Currently, the number of seaports located in soft soil accounts up to half the total number of seaports in Vietnam. The process of planning, designing, and constructing the foundation of the seaports is not optimized in Vietnam. Some seaports have uncounted problems when they are constructed on the soft soil. This paper aims to develop a method to reinforce the foundation of the riverbank so that the stability of the seaports can be maintained and any undesired deformation, or landslide, can be avoided. This method can help stabilize the structure that protects the seaports. The results obtained for the embankment structure with a stable wharf and a deep sliding with K=2.014 is larger than the allowable coefficient [K] = 1.25 in 22TCN 207-1992.

Keywords : Soft soil, riverbank, slope protection, FEM, Geo-Slope.

I. INTRODUCTION

The waterway system in Ho Chi Minh City has a total length of 975km with an average ratio of length to population of 0.181km/1000 people, and an average ratio of length to area of 0.465km/km2. The city has a waterway density that is equivalent to 73% of the waterway density of the Mekong Delta, which is the region with the highest waterway density in the country. In Ho Chi Minh City, there are currently 92 waterways, spanning a length of 798.7km, and in the whole country, there are 5 national inland waterways with a length of greater than 100km. There are different types of routes, including inter-provincial routes, routes that connect the city to the new seaports, and routes that transport passengers. Regarding inter-provincial routes, there is a large number of routes that run from Ho Chi Minh City to the southwestern provinces and the Southeast region. The local waterways in Ho Chi Minh City, the central waterways, and the multiple ports (ranging from small river ports to large seaports), collectively form an integrated water transport network. This network plays a crucial role in linking the Southern economic region and facilitating international trade and transportation.

Seaports mainly get damaged from the erosion and instability of the river banks. Therefore, the erosion of river banks and canals has been a growing concern in Ho Chi Minh City. Such incident can pose a direct threat to people lives and property. The areas, which have a high risk of landslides, are primarily located in the districts of Can Gio, Nha Be, Hoc Mon, Binh Chanh along with districts 9, 12, Thu Duc, Binh Thanh, and so on. The city currently focuses on solving the erosion issue in these areas.

The number of locations with landslide in the city has increased up to 40. The most recent landslide, which was found in Group 4, Hamlet 3, Hiep Phuoc Commune, Nha Be District, had seriously damaged the property of five households and the surrounding areas. The Nha Be District authorities had requested an immediate relocation for 270 households (975 people) which were situated along the canals with high risk of landslides such as the Kinh Lo river, the Cay Kho canal, and the Lo canal. Along with the relocation plan, the district authorities had also been preparing an anti-sliding project which aimed to prevent the erosion of riverbanks along the Giong Canal, the Kinh Lo river, and many other vulnerable spots. This would help protect the residents and the property in these areas.

In reality, the city has been investing extensively on projects that aim to overcome the problem of landslide and facilitate the relocation of residents in areas with dangerous landslide. Although billions of Vietnamese Dong (VND) have been invested in various projects, experts in urban environments still believe that it is difficult to predict the potential impacts on the city during natural disasters. This is because the urbanization process has led to the over-exploitation of river banks, canals, and reservoirs. There are illegal activities that overload the river banks and narrow down the water flow, leading to the erosion of the river banks. In addition, illegal sand mining activities, which are frequently observed in the Saigon and Dong Nai rivers, disrupt the water flow and eliminates balance between sand and mud, contributing to the erosion of river banks and canals. The inappropriate disposal of wastes into the canals can block the flow, causing water drainage problems. In Nha Be district, the portion of land, which was used for residential purpose, accounted for only 20% of the total area (which is approximately 10000 hectare) about six or seven years ago and now it has increased to 40%. Many farmland areas, which used to contain natural water, were destroyed and replaced with residential houses and factories.

Dominic, E. R. and Harshinie, K. (2022) wrote a book on the introduction to coastal engineering and coastal protection structures [1]. The fundamental principles that are applied to coastal flood protection, coastal protection, and coastal harbor structures are explored.

Nguyen, V. T. and Nguyen T. B. D. (2013) showed that the stability of the tall pier was created from the friction generated among the pile and the soil and the resistance of the soil. For the soft soil, the total stress at the pile tip is negligible and so is assumed to be zero. Thus, it is possible to increase the friction, the driving depth, and the pile circumference [2].

Peter, B. and Ralph, R. C. (2021) determined the level of restoration and protection for different coasts. These coasts were constructed using different methods under different beach management programs. These programs held different evaluations on the impact of storms [3]. The results demonstrated a close relationship between the tidal peaks and the measurement data for structural damage, e.g., coastal erosion.

The calculation of deformation for reinforced CDM columns to prevent landslides on riverside slopes has been conducted by Nguyen, N. T (2023) using the finite element method. The study investigated various factors that could influence the stability of the canals' slopes such as the water level in the canals and the shape of the cement-soil columns [4]. In addition, the effects of cement content on the strength cement-soil columns were considered in this study. The reinforcement of the soft soil using cement-soil columns can increase the load capacity and shear resistance of the soil. In the active region, the axial loading of the columns can contribute to the increase in shear and flexural resistance whereas in the passive region, the columns may crack under tension. Therefore, the columns in the active zone are more stable. In the passive areas or in the areas under shear condition, to enhance stability and prevent failure due to sliding, it is more effective to arrange the columns in the form of a wall, or a block, than to arrange them as single columns in series.

Pitthaya, J. et al. (2015) [5] employed three-dimensional finite element method to investigate the horizontal displacement and the safety factor of slopes reinforced by rows of cement-soil columns. As stability is a primary concern for roadbed slopes and river bank slopes reinforced with cement-soil columns, CDIT (2002) proposes a numerical analysis to check the stability in the structure [6].

Brian, O. O. and Benjamin A. O. (2017) investigated the application of geosynthetic membranes in soil stabilization and coastal defense structures [7]; Wang, J. et al. (2021) used three-dimensional landslide evolution model at the Yangtze River [8]; Amar, D. R. and Nirupama A. (2022) proposed a simple method for landslide risk assessment in Rivière Aux Vases basin, Quebec, Canada [9]; Zhang, H. et al. (2020) included a quantitative evaluation of soil anti-erodibility in the slope of a riverbank that was restored with a nature-based bioengineering soil in Liaohe River, Northeast China [10].

There are many factors that can cause failure in the wharf structures. Most of the damage to the wharf structures typically occurs in the design step, which involves the sketching of the structure and the estimation of the cost. In the design documents, the contractor may not have evaluated all the undesired factors that may reduce the load bearing capacity and the stability of the structure. This study employs the finite element method to enhance the stability of the river banks located along the wharfs in Ho Chi Minh City.

II. MATERIALS AND METHODS

A. Structure information

The project is adjacent to the old port on the downstream of the Dong Nai River, which belongs to the territory of Nhon Trach District, Dong Nai Province (Fig. 1) [7].

Figure 1: Overall design of the wharf structure [7]

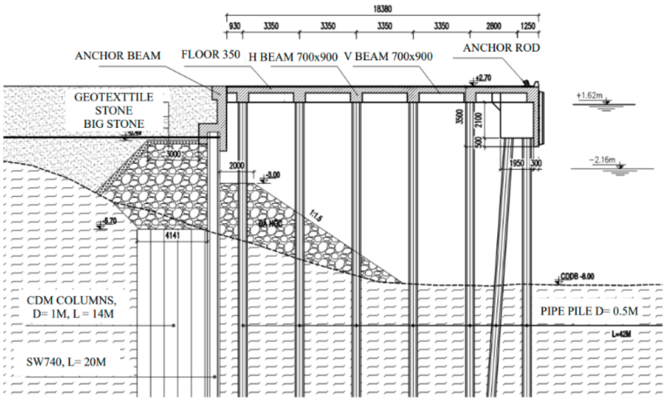

Fig. 2. Cross-section of the coastal protection structure [11]

The embankment behind the wharf is a vertical wall which is made up of pre-stressed concrete sheet piles combined with an anchor system. The piles are prestressed concrete piles SW740, which are linked together using 1x2 stone M300 reinforced concrete head beams. The anchor rods are made of steel with a diameter of D=60mm and a length of 35m perpendicular to the wharf's edge. The anchor bar is anchored to the 1x2 stone M300 reinforced with concrete anchor plate. The outside surface of the anchor plate is reinforced with diameter from 20 to 30 of rocks to bear the counter-pressure block. The part of the wharf behind the pile's wall, which is positioned 4m from the pile's edge, is reinforced with five rows of CDM columns with diameter D=1.0m (Fig. 2).

The piles of the interlocking foundation consist of two reinforced concrete piles which have a cross-sectional area of 0.3m x 0.3m and a length of 11.5 m. The piles are positioned at a distance of 0.5m measured from the existing foundation.

B. **Geological conditions**

Based on the geological conditions of the surrounding area, the stratigraphy of the area includes the following soil layers:

Grade A: Composed of smoothed soil and rock surfaces.

Layer 1: Consists of clay and sand-infused mud, displaying a gray-black color and fluid state.

Layer 2: Comprises clay mixed with dust, exhibiting a range of colors from gray, blue, white, to yellow-brown, and possessing a soft plastic state.

Layer 3: Composed of lightly mixed sand, displaying a mixed color of gray, green, and yellow-brown, and exhibiting a medium to compact texture.

Layer 4: Characterized by a semi-fat clay with a dark gray color and a soft plastic state.

Layer 5: Consists of heavy clay mixed with sand and dust particles, displaying gray-green color hues and a semi-hard to hard state.

Grade 5A: Comprises dust-infused clay, displaying a gray-white color hues and a semi-hard to hard state.

Grade 5B: Composed of lightly mixed sand, exhibiting gray-green color hues and a medium to compact texture.

Grade 6: Characterized by a semi-fat clay, displaying yellow-brown to gray-white color and possessing a semi-hard state.

Physical properties of the soil layers are shown in Table I.

The wharf, which is situated in the Dong Nai River, is mainly influenced by the semi-diurnal tidal regime of the East Sea, with a tidal range fluctuation of about 3.5m. The flow pattern is complex because of the three-way junction that divides the water current, creating an alternating current that comes from the two flow directions: the tide rising from the sea and the flow descending from the upstream.

The water level was selected as follows: The calculated water level that could be considered to be high was +1.62m (P = 1% based on the highest annual water level). The water level which was considered to be low based on the design: -2.49 m (P=98% based on the lowest annual water level).

Flow characteristics include: average flow velocity is 1.00m/s, maximum velocity at high tide is 1.21m/s, average velocity at high tide is 0.48 m/s, maximum velocity at low tide is 2.22 m/s, and average velocity at low tide is 0.74 m/s.

TABLE I.

PHYSICAL PROPERTIES OF THE SOIL LAYERS [11]

Properties	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 5A	Layer 5B	Layer 6
Grain size distribution								
+ Gravel (%)	1.02	0.25	4.17	0.00	1.02	0.00	10.08	0.00
+ Sand (%)	34.79	31.16	84.05	25.52	34.21	29.52	80.91	28.70
+ Silt (%)	19.27	37.06	6.78	29.53	37.90	37.41	4.81	28.25

+ Clay (%)	44.92	31.53	5.00	44.95	26.87	33.07	4.20	43.05
Water content, W (%)	73.15	26.26	20.39	41.49	19.18	18.73	18.39	22.48
Wet unit weight, (γ_w) g/cm^3	1.51	1.93	1.98	1.75	1.99	2.04	1.99	2.00
Dry unit weight (γ_d) g/cm^3	0.87	1.53	1.65	1.24	1.67	1.72	1.68	1.63
Void ratio (e_0)	2.011	0.773	0.617	1.203	0.620	0.578	0.0589	0.679
Liquid limit, LL (%)	62.9	34.8	0.0	51.5	34.9	36.5	0.0	43.8
Plastic limit, PL (%)	31.8	16.0	0.0	25.9	18.2	18.6	0.0	20.5
Friction angle (φ^o)	4°58’	10°32’	26°14’	10°29’	15°7’	15°26’	26°20’	15°10’
Cohesion, c (kg/cm^2)	0.048	0.208	0.051	0.221	0.341	0.377	0.051	0.401
SPT-Value	0-4	6-8	11-17	6-11	6-32	35	26-40	34-36

C. **Overall stability check**

SLOPE/W is a program by GEO-SLOPE, CANADA, specializing in slope stability calculations. The program offers slope calculations in all possible real-world conditions such as: determining pore water pressure, soil anchors, geotextiles, external loads, and retaining walls. Some of the theories used in the Slope/W program include:

1. Ordinary method: Assumes both the normal force and shear force of the sliding mass are zero.
2. Bishop method: Simplifies the problem by focusing only on the normal force and not considering the tangential force between the slices, satisfying only the moment equilibrium equation.
3. Janbu method: Simplifies the problem by only using the normal force and not using the tangential force between slices. This method is based solely on the condition of force equilibrium.
4. Spencer method: Considers both the force equilibrium and moment equilibrium conditions. Shear force is not assumed to be a constant.
5. Morgenstern-Price and Gle methods: Use both the normal and tangential forces between slices. Both the force equilibrium equation and moment equilibrium equation must be satisfied.

The embankment peak level is + 2.70 (Hon Dau altitude system);

The mechanical characteristics are used to calculate the deformation state of the soil with the angle of internal friction and the cohesive force C values for each soil layer at their specific positions in the construction site.

Properties of embankment materials in geo-slope model are illustrated in Table II.

TABLE II. PROPERTIES OF EMBANKMENT MATERIALS IN GEO-SLOPE MODEL

Parameters	Unit weight (□) T/m^3	Cohesion (c) kPa	Friction angle (□) ^{0}
Backfill sand	1.8	0	30
Concrete	2.5	1000	45
Crushed stone	2.4	0	35
Riprap	2.4	0	45
CDM	1.7	89.5	-

Calculations are performed for the case with the lowest target water level. The load on the top of the embankment is q = 0.5 T/m^2 (load is positioned at the edge of the embankment). The load behind the top of the embankment is q = 2.0 T/m² (load positioned 3.0 m from the top of the embankment). The stability of the slope is calculated using the GEO-SLOPE software, and the most dangerous slip surface is identified. Based on the standard 22TCN 207-1992, the overall stability safety factor of the construction is determined Eq (1).

$$n_c.n.m_đ.M_{tr} \leq \frac{m}{K_n}M_g \qquad (1)$$

Where nc is the load combination factor (nc = 1.0 for the basic combination, n_c = 0.9 for the special combination, and n_c = 0.95 for the construction combination), n is the overload factor (n = 1.25), md is the working condition factor (m_d = 1.0), M is the working condition coefficient (m = 1.15), k_n is the reliability factor (k_n = 1.15), and M_{tr} and M_g are, respectively, the total moment of the forces that cause slipping and the forces that correspond to the dangerous slipping center.

$$M_{tr} = R.\sum g_i.sin\alpha_i + \sum W_i.Z_i \qquad (2)$$

$$M_g = R.\left[\sum g_i.cos\alpha_i.tg\varphi + \sum c_i.l_i\right] \qquad (3)$$

where R is the radius of slip circle; g_i is the total weight of the soil layers, the structure, and the load of soil column I; α_i is the inclination angle to the horizontal line of the tangent to the slip circle at the intersection of the slip circle with the line of the force g_i.

$$\alpha_i = \arcsin(r_i/R) \qquad (4)$$

where r_i is the horizontal distance from the center of slip to the line of the force g_i; φ_I and C_i are, respectively, the friction angle and the cohesive force of the soil layer at the base of soil column I; l_i is the length of the slip circle segment at the base of soil column i; Wi is the increased hydrodynamic pressure.

From the above conditions, we have:

$$K = \frac{M_g}{M_t} \geq \frac{n_c.n.m_đ.k_n}{m} = [K] \qquad (5)$$

III. RESUTLS AND DISCUSSION

The calculation results illustrated in Table III.

TABLE III. CALCULATION RESULTS FOR FS_{min}

Calculation Combination	Basic	Construction & Repair	Special
n_c	1	0.95	0.9
k_n	1.15	1.15	1.15
m_d	1.0	1.0	1.0
m	1.15	1.15	1.15
n	1.25	1.25	1.25
FS_{min}	1.25	1.19	1.13

The calculated stability results for deep sliding of the wharf structure show a stability coefficient of 2.014, as illustrated in Fig. 3.

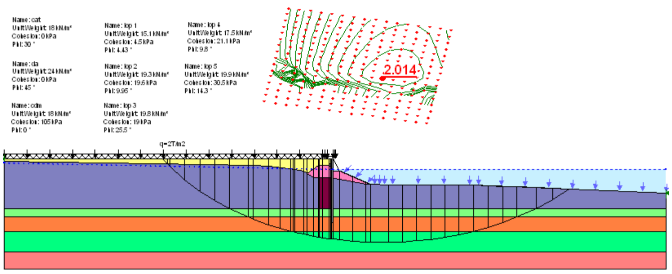

Fig. 3. Stability coefficient of the project

When constructing water ports in areas with complex geology, the calculation of slope stability is a crucial task. The accuracy of the calculation greatly affects the durability and long-term stability of the ports.

To analyze the slope stability, the following two methods are typically used: The first method takes into account the sliding surface in advance and only considers the equilibrium state of the points located on the slip arc. The second method treats the soil as an elastic-plastic environment and applies the finite element method to analyze the stress-deformation of the points in the ground.

IV. CONCLUSIONS

The slope of the embankment behind the wharf is stable with a coefficient K= 2.014 after being reinforced by the proposed method, which involves pre-stressed concrete piles SW740. The pile bars are linked together by a 1x2 steel-reinforced concrete cap beam with a compressive strength of 30MPa. The pile heads, which are anchored with steel bars, have a diameter of 0.06m and a length of 35m. The pile heads extend in the direction that is perpendicular to the wharf edge. The bars are anchored into an anchor plate. On the outside of the anchor plate are stones of diameter from 20mm to 30mm which are used to reinforce the counterweight block. The part of the wharf behind the pile wall, which is positioned 4.0m from the pile edge, is reinforced by 5 rows of cement deep mixing columns with diameter D=1.0 m.

These are preliminary studies that have opened up a new direction for the application of new technology to stabilize the protective structures of water ports. To expand the implementation of the proposed method, it is necessary to continue testing, learning from failures, and re-defining the mix proportions, the pile schemes, and the span lengths to suit the topographical and geological conditions of the areas where the water ports are situated.

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