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Mechanism of Cement Deep Mixing and Design Method Improving Soft Ground in Mekong Delta

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ABSTRACT

Soil improvement technique for embankment fill is often needed to rapidly improve the strength of the soft ground. In recent years, the cement deep mixing (CDM) method has been commonly used as an alternative to improve the soft soil foundation of embankment. This method is a reasonable solution used for several applications as to treat the soft soil behind the bridge abutment, to minimize settlement and enhance the bearing capacity of soil foundation for a storage tank, to support segmental retaining walls, and to prevent differential settlement between a new embankment over soft soil and existing embankment where the settlement has ceased. The stability of cement column stabilized embankments can be analyzed either by a numerical calculation method such as the finite element method (FEM) or by a limit equilibrium method. The numerical calculation methods require reliable material parameters as determined by field and laboratory tests or in-situ on excavated columns and on the unstabilized soil. Another method considering simplified method is limit equilibrium method. In analysis of this method, the stability of cement column is analyzed by assuming that failure of the cement columns and the soil.

Keywords: Soil Improvement, Cement Deep Mixing, Soft Soil, Embankment, Simplified Method

I. INTRODUCTION

Most of cities and provinces of Vietnam are located in the Mekong River Delta and the Red River Delta. The Mekong Delta is located in the southern side of Vietnam. As a delta area, most of the soil layers are categorized as the soft soil. The properties of the soil are high water content, low stiffness and less frictional angle and less cohesion as well. During recently years, stabilization of soft ground by CDM method has become an increasingly and strong developed in this area.

Anucha Wonglert et al. (2018) studied the bearing capacity and failure behaviors of floating stiffened deep cement mixing columns under axial load [1]. This research aims to clarify and gain an insight into the impact of the length of the stiffened core and the strength of the CDM socket on the behaviors of floating stiffened deep cement mixing columns. The observed behaviors include the axial ultimate bearing capacity, settlement and failure mode. The study begins by conducting a series of physical model tests as a preliminary investigation. The results reveal that the strength of the CDM socket can be reduced to a certain value by inserting a sufficiently long reinforced core to achieve the highest possible load-carrying capacity, indicating an optimum length of the stiffened core for a specific CDM socket strength.

Nguyen Ngoc Thang, Nguyen Anh Tuan (2018), Hong-Son Nguyen et al. (2020), Nguyen Anh Tuan, Nguyen Thanh Dat (2020) and Dario Pedutoa et al. (2020) studied the nonlinear FEM analysis of cement column configuration in the foundation improved by deep mixing method. In this study, the stress distribution and the deformation in the foundation improved by DMM are analyzed using nonlinear FEM in which the stress-strain relation is elastoplastic to apply to more detailed specification of the configuration of CDM columns [2, 3, 4, 5].

Anand et al. (2019) has a study on design and construction of lightweight EPS geofoam embedded geomaterial embankment system for control of settlements [6]. This research describes initial design and construction details of the rehabilitation works performed on the embankment system along with a focus on the early performance details. Field monitoring studies were conducted for almost three years to study the bump/settlements under the EPS geofoam embankment system. Short term measured settlement data was analyzed with hyperbolic model to predict the long-term settlements. Numerical finite element studies attempted in this study showed that settlements could be reasonably predicted by modeling these geofoam embankments. Based on the monitoring and modeling studies, the effectiveness of utilizing EPS geofoam as an embankment fill material was addressed to mitigate the differential settlements under a bridge approach slab.

Tuan Anh Nguyen, Thang Ngoc Nguyen (2020) studied the stress distribution in soft ground consolidated with deep cement mixing columns under road embankment [7]. In this study, the finite element method (FEM) using PLAXIS software is adopted to analyze the stress distribution on columns and ground base of the CDM columns system combined with geotextiles in the consolidation of soft ground under road embankment in Tien Giang Province. With this method, the behavior of the CDM columns in soft ground treatment is examined by the distribution of stress and settlement of the CDM columns and soft soil layers.

II. SELECTION OF MATERIAL PROPERTIES FOR DESIGN METHOD

The engineer usually specifies the strength required and provides provisions to verify the continuity and homogeneity of the CDM columns. The contractor specializing in soil cement mixing usually determines the mix design. In the mix design the type of the soil, type of the equipment used for installation procedure, required quality and economics of the project are taken into account.

Laboratory tests performed on soils stabilized by deep mixing are unconfined compression, triaxial compression, direct shear, and oedometer (for one-dimensional compression). Laboratory tests are performed for samples made in the laboratory by mixing cement grout or dry cement reagent with the soil from the site and for core samples drilled from installed deep mixing elements.

The purposes of mix tests are: To check the suitability of soil cement mixing to decrease compressibility of the soft clay under the road embankment; To determine an economical and suitable cement content which is defined as the percentage of the weight of the cement to the wet weight of the soil to be improved; To provide the strength parameters of the soil-cement for design.

1. Unit weight, γ

The unit weight of the cement slurry or cement powder is not much larger than that of the original soil. Therefore, the change in unit weight of the soil-cement is small. Even when the cement content reaches 25%, the unit weight of the soil-cement increases only 3% as compared to that of the original soil.

2. Undrained shear strength $c_{u,col}$

The shear strength of the stabilized soil in the CDM columns is not uniform even when the mixing of the binders with the clay is done very carefully. Because of the aggregation, the measured shear strength varies with the testing method and the size of the tested samples. According to the SGF Report 4:95E (1997), the maximum value of the undrained strength cu,col of the CDM column should not exceed 150 kPa irrespective of the results of laboratory and field tests [8].

3. Unconfined Compressive Strength $q_{u,col}$

Generally, the unconfined compressive strength of soil-cement is between 50 kPa to 4000 kPa, which is 50 to 100 times greater than that of pre-improved soil but still much lower than that of concrete.

For a wide variety of soil types and binder mixes, the 28-day unconfined compressive strengths for soils treated by the wet method may range from 140 to 27,600 kPa. The 28-day unconfined compressive strengths for soils treated by the dry method may range from 14 to 2,760 kPa.

The focus of the material design is that quality of the product must be achieved to satisfy the minimum strength and other design requirements. Although the soil-cement mixing contractor often determines the mix design, it is important for the design engineer to understand the factors contributing to the strength and permeability of the soil cement.

The strength of soil columns is affected by flocculation and by cementation caused by chemical reactions of the cement. However, the shear strength is also affected by the in-situ state of stress, the drainage conditions, time, temperature, loading rates and the soil composition etc. There are, generally, many factors, which affect the strength of treated soil.

4. Permeability, k

The permeability of the stabilized soil is often determined in the laboratory by constant and falling head permeability tests and by triaxial and oedometer tests. The laboratory tests are carried out on relatively small samples from actual cement columns or with compacted laboratory samples. The permeability determined in the laboratory in general is too low compared with the permeability measured in-situ in the field.

The permeability of CDM columns is low. In some cases, the permeability is even lower than the permeability of the unstabilized soil. Due to the low permeability, CDM columns do not work as drains. The coefficient of permeability of a soil treated with cement decreases with increasing cement content.

5. Modulus of Elasticity, E_{col}

The modulus as determined by unconfined compression tests on samples prepared in the laboratory is higher than the modulus for cores from CDM columns in the field. This difference could possibly be caused by cracks and fissures in the investigated cores caused by coring and the handling of the core. Ekstrom J. C. et al. (1994) recommends that E_{col} should not be determined from laboratory samples [9]. The ratio $E_{col}/q_{u,col}$ can be assumed to about 350 to 1000 for CDM columns.

6. Poisson's Ratio of CDM column, ν

Hirade et al. (1995) reported that the static Poisson's ratio is approximately 0.5 if the soil cement is loaded under undrained conditions and ranges between 0.3 to 0.45 under other loading condition [10]. Michell (1970) suggested that the Poisson's ratio of cement-stabilized soil is 0.1 to 0.2 for granular soils and 0.15 to 0.35 for fine-grained soils [11].

III. DESIGN METHOD AND PROBLEM IN THE DESIGN OF CDM COLUMN

CDM column is a type of columns where cement is mixed in-situ with soil to stabilize soft soil such as clay or organic soil. Hence, the design method of CDM column is similar to other columns such as stone columns, sand compaction plies, geotextile encased sand/gravel column. There were many researches on design of CDM column, which supports embankment. Bengt B. Broms (1999), Kitazume M. et al. (2003) and EuroSoilStab (2002) are there of them. In their researches, the uniform lengths of the CDM columns were used to stabilize the soft soil under embankment [12, 13, 14]. The method of design is:

a. Collect data of construction load, surcharge load.

b. Check bearing capacity and settlement of natural soil, when a structure is built (using the standard approach for shallow foundation).

c. Determine the load, which will apply on the CDM columns.

d. Determine the bearing capacity of the CDM columns based on surrounding soil conditions (failure soil).

e. Determine the strength of the CDM columns (field test and laboratory test) based on material CDM column (failure column): Laboratory tests are performed on samples made in the laboratory by mixing cement with soil from the site, and on sample drilled from trial CDM column.

f. Determine the best of set: length of the CDM column (L), diameter of the CDM column (d) and spacing of the CDM columns (s).

g. Determine settlement of improvement soil.

h. Check stabilities of construction

i. Determine bearing capacity of the soil below improvement soil.

j. Check bearing capacity, stability and settlement again by FEM.

In the numerical analysis, CDM columns, soil layers and embankment are modeled as nonlinearly elasto-plastic materials with Mohr-Coulomb failure criteria. The two-dimensional (2D) numerical model is used for this study. Due to the symmetry of the problem, only half of the section is modeled to save the computing time.

A finite element mesh is constructed to simulate the sequential construction procedure of the embankment and to calculate the resulting consolidation settlements in the soft soil layer. The mesh includes half of the geometry because of symmetry. The mesh consists of the part the soft soil layer, silty clay layer, silty sand layer and the embankment layers. The elements are pore water/ stress four node quadrilateral element (with bilinear displacement and bilinear pore pressure) and appropriate for finite strain analysis.

The CDM column in FEM is assumed continuum element, which elements can be used to model the widest variety of components. Conceptually, continuum elements simply model small blocks of material in a component. Since they may be connected to other elements on any of their faces, continuum elements, like bricks in a building or tiles in a mosaic, can be used to build models of nearly any shape, subjected to nearly any loading.

A coupled pore water diffusion/stress analysis is carried out with fully saturated water elements, and pore pressure elements. Pore pressure elements are provided for modeling water flow through a deforming porous medium in a coupled pore water diffusion/stress analysis. These elements have pore pressure degree of freedom in addition to displacement degrees of freedom 1–3.

1. Plane strain condition:

The plane strain assumption is frequently used in geotechnical analysis of soil structure that is very long in one dimension while having a uniform cross section with finite dimensions. The soil, embankment and CDM column are long in the z direction while having a uniform cross section with finite dimensions in the x-y plane. In this embankment model, the strains along the z-axis are assumed to be nil (i.e., $\varepsilon_{33} = \varepsilon_{13} = \varepsilon_{23}$). The seemingly three-dimensional embankment problem reduces to two-dimensional plan problem in which the cross section of the embankment, in the x-y plane.

2. Total and excess pore water pressure:

The coupled pore water diffusion/stress analysis can provide solutions either in terms of total or excess pore water pressure. The excess pore water pressure at a point is the pore water pressure in excess of the hydrostatic pressure required to support the weight of pore water above the elevation of the material point. The difference between total and excess pore pressure is relevant only for cases in which gravitational loading is important. Total pore pressure solutions are provided when the gravity-distributed load is used to define the gravity load on the model. Excess pore pressure solutions are provided in all other cases.

3. Steady-state analysis:

In the first calculation step, the embankment is removed from the finite element mesh. Then the embankment layers are added, layer by layer, in subsequent calculation steps. When the embankment layer is added, it is situated on the deformed layer that was added earlier. The new layer is assumed to be strain-free at the time of construction.

Boundary condition: In embankment model, two types of boundary conditions must be specified: Displacement boundary conditions and hydraulic boundary conditions. The top surface layer is made pervious, thereafter, a perfect drainage is assumed so that the excess pore pressure is always zero on this surface.

4. Static equilibrium:

Most geotechnical problems begin from a geostatic state, which is a steady-state equilibrium configuration of the undisturbed soil or rock body under geostatic loading. The equilibrium state usually includes both horizontal and vertical stress components. It is important to establish these initial conditions correctly so that the problem begins from an equilibrium state. Since such problems often involve fully or partially saturated flow, the initial void ratio of the porous medium, e_0, the initial pore pressure, u_0, and the initial effective stress must all be defined.

The geostatic procedure is normally used as the first step of a geotechnical analysis; in such cases gravity loads are applied during this step. Ideally, the loads and initial stresses should exactly equilibrate and produce zero deformations. However, in complex problems it may be difficult to specify initial stresses and loads that equilibrate exactly. This stress state, which is a modification of the stress field defined by the initial conditions, will then be used as the initial stress field in a subsequent coupled pore water diffusion/stress or static analysis.

IV. CONSOLIDATION PROBLEM

In coupled finite element analysis, the effective stress principle is applied. Each point of the saturated soil mass is subject to a total stress, σ, which is the sum of the effective stress, σ', carried by the soil skeleton, and the pore pressure, u. The pore pressure will increase by the addition of a load to the soil. Consequently, a hydraulic gradient of pore pressure will develop between two points within the mass. The hydraulic gradient between the two points will cause the water to flow. The flow velocity, ν, is assumed to be proportional to the hydraulic gradient, i, according to Darcy's law, $\nu = ki$, where k is the soil permeability. As the external load is applied to the soil mass, the pore pressure rises initially. Then as the soil skeleton absorbs the extra stress, the pore pressure decreases and the soil consolidates.

To understand the consolidation analysis, Terzaghi's one-dimensional consolidation theory is reviewed.

Several assumptions are used in the derivation of Terzaghi's one dimensional consolidation equation:

The soft soil if fully saturated and homogeneous.

Water compressibility is negligible.

The compressibility of soil grain is also negligible, but soil grains can be rearranged during consolidation.

The flow of water obeys Darcy's law.

The total stress (Δσ) applied to the element is assumed to remain constant.

The coefficient of volume compressibility, m_v, is assumed to be constant.

The coefficient of permeability, k, for vertical flow is assumed to be constant.

Using these assumptions and considering that the rate of volume change of the cubic element is equal to the difference between the rate of outflow and the rate of outflow and the rate of inflow of water $\partial V / \partial t = q_{out} - q_{in}$, one can derive the basic equation for one dimensional consolidation.

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad (1)$$

Where cv is the coefficient of consolidation, given by Eq. (2)

$$c_v = \frac{k}{m_v k_v} \qquad (2)$$

For uniform initial excess pore pressure distribution with depth, and using a Fourier series, the exact solution of Eq. (1) is

$$u = \sum_{m=0}^{m=\infty} \left(\frac{2u_0}{M} \sin \frac{M_z}{H_{dr}} \right) \exp\left(-M^2 T_v\right) \qquad (3)$$

In which, m= 0, 1, 2, 3,…, ∞; u_0 is the initial excess pore pressure; and T_v is a nondimensional number call the time factor, defined as Eq. (4)

$$T_v = \frac{c_v t}{H_{dr}^2} \qquad (4)$$

Define the degree of consolidation at depth z and time t as:

$$U_z = \frac{u_0 - u}{u_0} = 1 - \frac{u}{u_0} = 1 - \sum_{m=0}^{m=\infty} \left(\frac{2}{M} \sin \frac{M_z}{H_{dr}} \right) \exp\left(-M^2 T_v\right) \qquad (5)$$

The degree of consolidation at a point given by Eq. (5) is the ratio of the dissipated excess pore pressure (= u_0-u) to the initial pore pressure at the same point (u_0). For example, at t=0, when stress is applied, the excess pore pressure (u) is equal to the initial excess pore pressure (u_0); Therefore, U_z= 0 and no consolidation has occurred. However, when t→∞, u→0 and U_z→1 (or 100%); that is, the consolidation is 100% complete.

Or more interest is the overall degree of consolidation of a soft soil layer rather than the degree of consolidation at a point within the soft soil layer. The average of consolidation for the entire thickness of the soft soil at time t is defined as Eq. (6).

$$U = 1 - \frac{\frac{1}{2} H_{dr} \int_0^{2H_{dr}} u dz}{u_0} = 1 - \sum_{m=0}^{m=\infty} \left(\frac{2}{M^2} \right) \exp\left(-M^2 T_v \right) \tag{6}$$

It is to be noted that in this equation the initial pore pressure distribution is assumed uniform throughout the thickness of the soft soil layer. Also note that at t=0, when stress is applied, the excess pore pressure is equal to the initial has occurred. However, when $t \to \infty$, $u \to 0$ and $U_z \to 1$ (or 100%); that is, the consolidation is 100% complete.

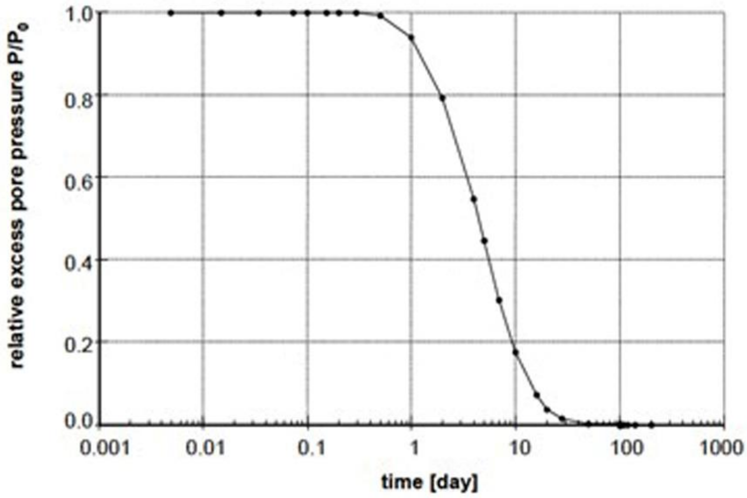

Figure 1. Development of excess pore pressure as a function of time

For the excess pore pressure to dissipate during consolidation, is shown in Fig 1, water must travel to the top boundary of the soft soil layer and sometimes to the bottom boundary as well. Where a soil layer is considerably more permeable than the soft soil layer itself. Logically, the rate of consolidation depends on the length of the longest path traveled by drop of water. This length is calling the drainage path length, H_{dr}. There are two possible drainage types:

Two-way drainage with a permeable layer both above and below the soft soil layer, as indicated in Fig 2.a. In this case the longest path traveled by a drop of water located anywhere within the soft soil layer is H_{dr} = h/2, where H is the thickness of soft soil layer.

One-way drainage with a permeable layer above the soft soil layer. In this case H_{dr} = H, as indicated in Fig 2.b.

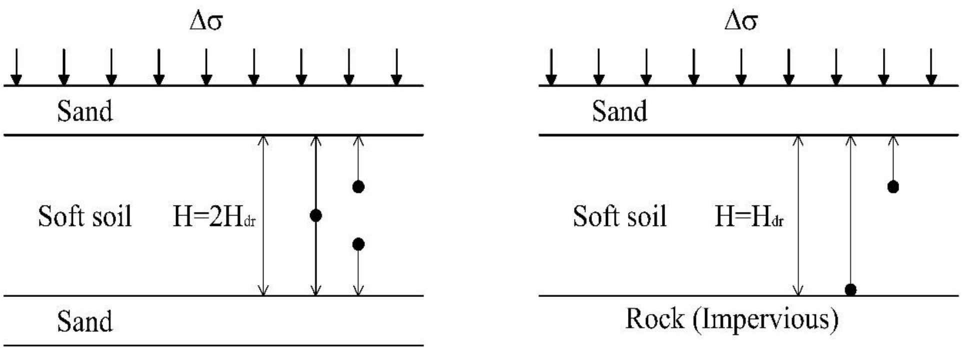

Figure 2. (a) two – and (b) one – way drainage conditions

V. CONCLUSION

In the design method of CDM column, the settlement of the column and the settlement of the soil between the CDM columns are assumed the same. Reality, they are difference because the different stiffness.
Problem in design of CDM column: In design of CDM column, besides the evaluating reduce of settlement, increase of stability, increase of bearing capacity. Construction cost should be reduced. The problem in the design is how to define the optimum configuration of CDM column that is a set of length, diameter and spacing of columns.

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