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STABILITY OF DOCK STRUCTURES FOUNDED ON LIQUEFIABLE SOILS

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ABSTRACT

During seismic events or wave loadings, the stability of dock and marine structures founded on liquefiable soils becomes a major concern. In this paper, a liquefaction assessment for a dock structure founded on liquefiable soils, subjected to different earthquake excitations, is presented. The liquefaction assessment is based on intensive cone penetration testing over water. The results of the study indicate that a significant increase in liquefaction potential occurs with the increase in water depth. The global stability of the dock structure is assessed using a pseudo-static approach. The study indicates that a soil treatment or densification zone equivalent to about twice the width of the slope under the dock is required to avoid the structural collapse during a major seismic event.

Keywords: dock structures, ports, liquefaction, earthquakes and global stability.

1. INTRODUCTION

Experience from past earthquakes (e.g., Tokachi-oaki and Kobe earthquakes, Japan in 2003 and 1995, respectively, Kocaeli earthquake, Turkey in 1999, Loma Prieta earthquake, USA in 1989) proved that the most significant source of damage to ports is the liquefaction of saturated loose to compact sandy soils.

Liquefaction is defined as the transformation of a granular material from a solid to liquefied state as a consequence of increased pore water pressure and reduced effective stress (Marcuson, 1978). In another meaning, liquefaction is a sudden loss of soil strength due to earthquake excitation or waves induced seabed instability. Earthquakes and/or waves exert dynamic pressure fluctuations on the sea floor, which generates excess pore water pressure within the soil Skelton. As liquefaction occurs, the soil softens allowing large deformation to occur. In loose soils, the softening is accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground (Youd and Idriss, 2001).

2. DOCK STRUCTURE AND SITE DESCRIPTION

In this paper, a case study of a dock structure to be founded on liquefiable soils in British Columbia, Canada is examined. The dock will be an approximately 940 m long, 40 m wide, pile-supported deck. The slope below the deck will be 19.5 m high, constructed at 2H:1V, and covered with riprap erosion protection, with the toe of the fill at the fender line as shown in Figure 1. Placement of significant quantity of fill into water will be required for constructing the slope.

Cone Penetration Tests (CPTs) were conducted over water at 8 locations along the proposed dock length. The testing was conducted from a spud barge using a B-80 drill rig. The depth of water at the test locations ranged from 2.7 to 15.7 m and the CPTs were advanced to 21 to 32 m below mudline. The approximate CPT locations were determined by Global Positioning System (GPS). Tip resistance, sleeve friction and pore pressure were electronically measured and recorded at 0.05 m intervals as the cones were advanced. The CPTs provide continuous profiles of penetration resistance, which can be used in soil classification and liquefaction assessment.

CPT results indicate that 2.5 to 5 m of soft to very soft silts were encountered below mudline and were underlain by 12 to 24 m of compact sands and silty sands with occasional thin layers of sandy silt or silt. Firm to stiff silt deposits were encountered below the sands. Thin layers of sand and/or silty sand were encountered within the silts at some locations.

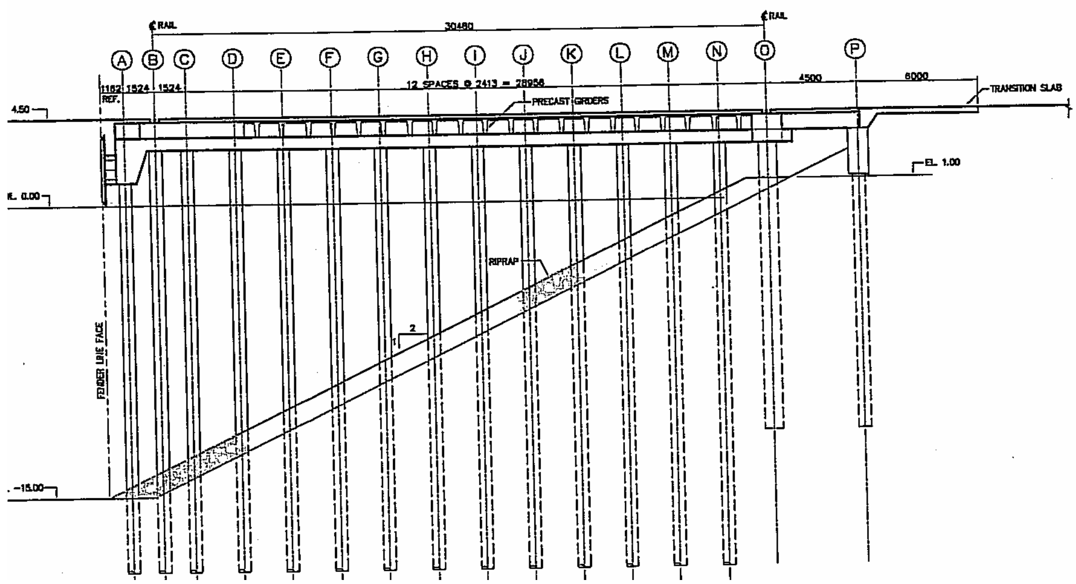


Fig. 1 The proposed dock structure

3. LIQUEFACTION ASSESSMENT

3.1 Methodology

The potential of soil liquefaction is computed by comparing the seismic demand placed on a soil layer, cyclic stress ratio (CSR) and the capacity of the soil to resist liquefaction, cyclic resistance ratio (CRR). The cyclic stress ratio (CSR) is presented by Seed and Idriss (1971) as follows:

$$CSR = \tau_{av} / \sigma'_{vo} = 0.65(a_{\max} / g)(\sigma_{vo} / \sigma'_{vo}) r_d \qquad (1)$$

where τ_{av} is the average cyclic shear stress, a_{max} is the peak horizontal acceleration at the ground surface, g is the gravity acceleration, σ_{vo} and σ'_{vo} are the total and effective vertical overburden stresses, respectively, and r_d is a stress reduction coefficient which can be calculated using the recommendation by Youd and Idriss (2001) as follows:

$$r_d = \frac{(1-0.4113z^{0.5}+0.04052z+0.001753z^{1.5})}{(1-0.4177z^{0.5}+0.05729z-0.006205z^{1.5}+0.001210z^{2}} \quad (2)$$

where z is the depth below ground surface in meters.

The cyclic resistance ratio (CRR) for earthquake of magnitude M of 7.5 is presented by Robertson and Wride (1997) as follows:

$CRR=0.833[(q_{c1N})_{cs}/1000]+0.05$ If $(q_{c1N})_{cs} < 50$ (3a)

$CRR=93[(q_{c1N})_{cs}/1000]^3+0.08$ If $50 \leq (q_{c1N})_{cs} < 160$ (3b)

Where $(q_{c1N})_{cs}$ is the clean sand cone penetration resistance.

At any depth, if CSR is greater than CRR, the soil is considered to be liquefiable. It should be noted that the above equations were based on extensive lab and field testing and were used primarily for onshore soils.

3.2 Effect of Seismic Acceleration on Liquefaction

In this paper, assessment of soil liquefaction potential during 1:100, 1:475 and 1:2475 year seismic events was conducted. Rock accelerations of 0.09g, 0.21g and 0.5g and amplified surface acceleration of 0.2g, 0.3g and 0.43g, respectively, were used for these seismic events. A factor of safety of 1.1 was utilized in the analysis.

Liquefaction assessment was conducted for all CPTs. For the purposes of presentation, a representative CPT advanced to about 30 m depth below mudline was chosen. Water at this CPT location was about 6 m depth. Figure 2 represents the tip resistance, sleeve friction resistance, friction ratio (Rf) and measured pore water pressure along depth for this CPT sounding.

Liquefaction susceptible zones for the 1:100, 1:475 and 1:2475 year seismic events below a constant water depth of 6 m are shown in shaded areas in Figure 3, where qcN1-noliq is greater than $(q_{c1N})_{cs}$. $(q_{c1N})_{cs}$ is the clean sand cone penetration resistance and qcN1-noliq represents the minimum penetration resistance for non liquefiable soil. This figure indicates that the soil is liquefiable to about 17 m depth below mudline for the 1:100 year earthquake and to about 22 m depth for the 1:475 and 1:2475 year earthquakes. As expected, the potential for liquefaction, as represented by the shaded areas, increases with the increase in acceleration. Therefore, more soil treatment or densification is required if an intensive earthquake excitation is utilized in the design.

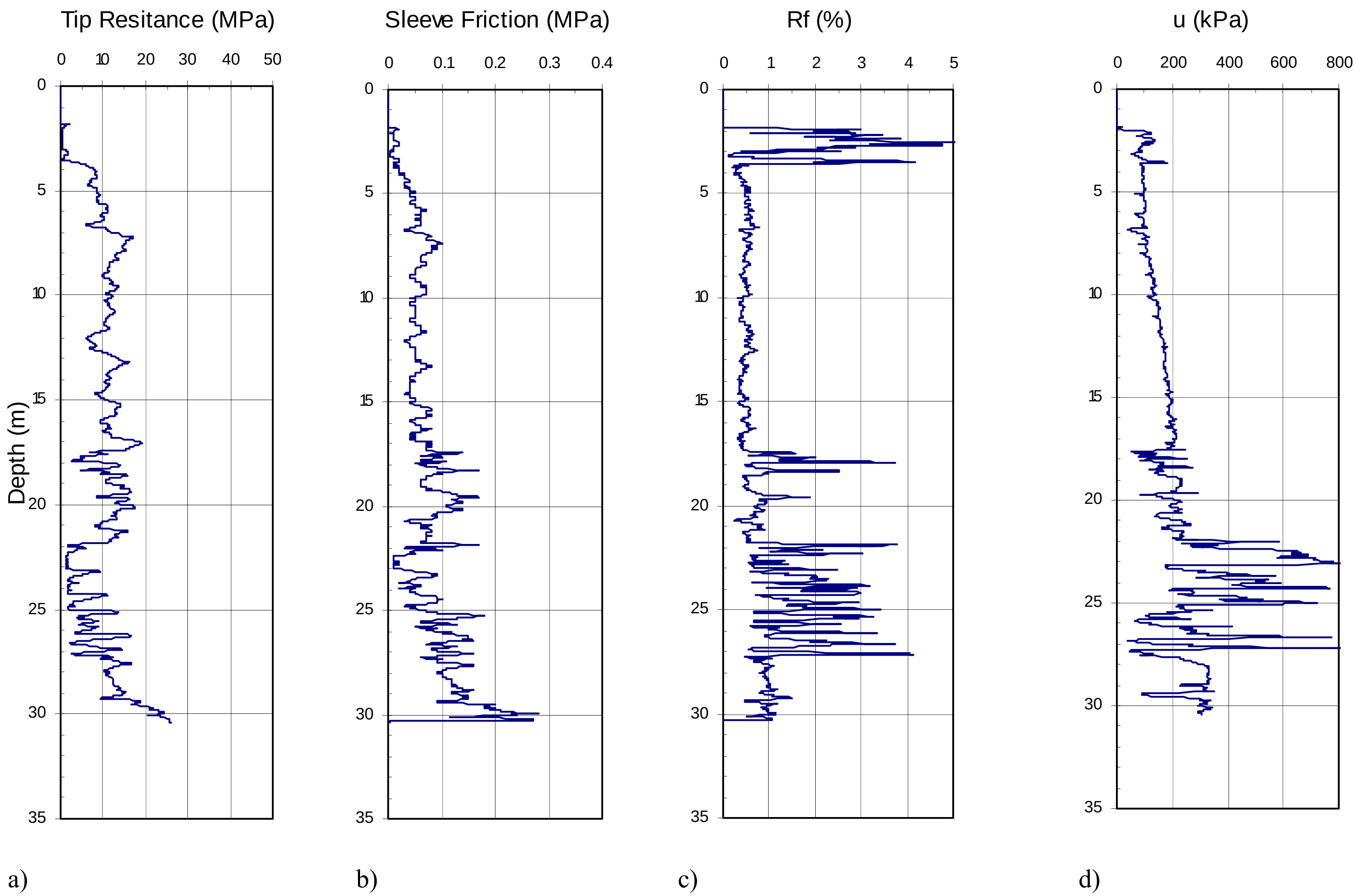
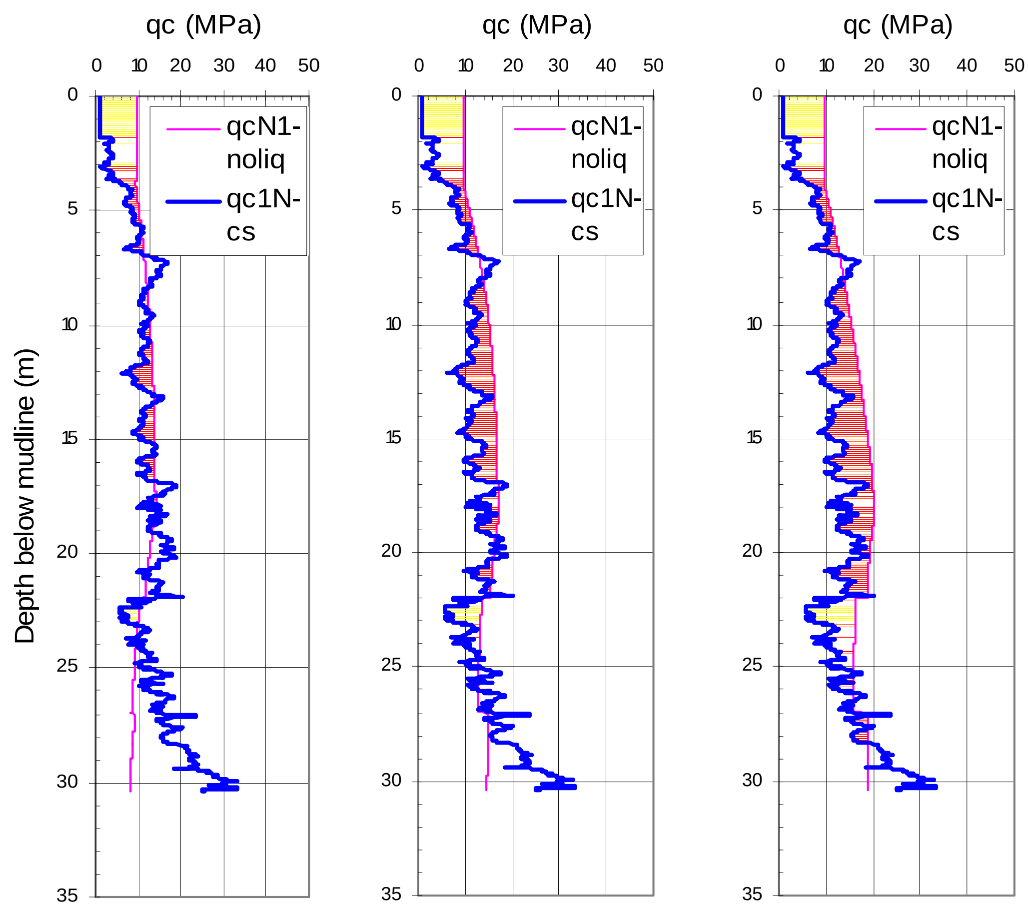


Fig. 2 A representative CPT row data, a) tip resistance (q); b) sleeve friction; c) friction ratio (Rf) and d) pore water pressure (u).



a) b) c)

Fig. 3 Liquefaction susceptible zones along depth for a representative CPT (water depth, H=6 m); a) 1:100 year earthquake; b) 1:475 year earthquake and c) 1:2475 year earthquake.

3.3 Effect of Water Depth on Liquefaction

Water depth was included in the calculations by adding the weight of water to the total overburden stress, σ_{vo}. The effective stress σ'_{vo} was calculated by subtracting the pore water pressure from the adjusted overburden stress. Figure 4 shows CRR and CSR for the 1:475 year earthquake along soil depth below mudline at water depths of 0, 6 and 15 m. It can be seen from Figure 4 that CSR increases dramatically with the increase in water depth. In other meaning, soil susceptibility to liquefaction increases significantly with the increase in water depth above mudline.

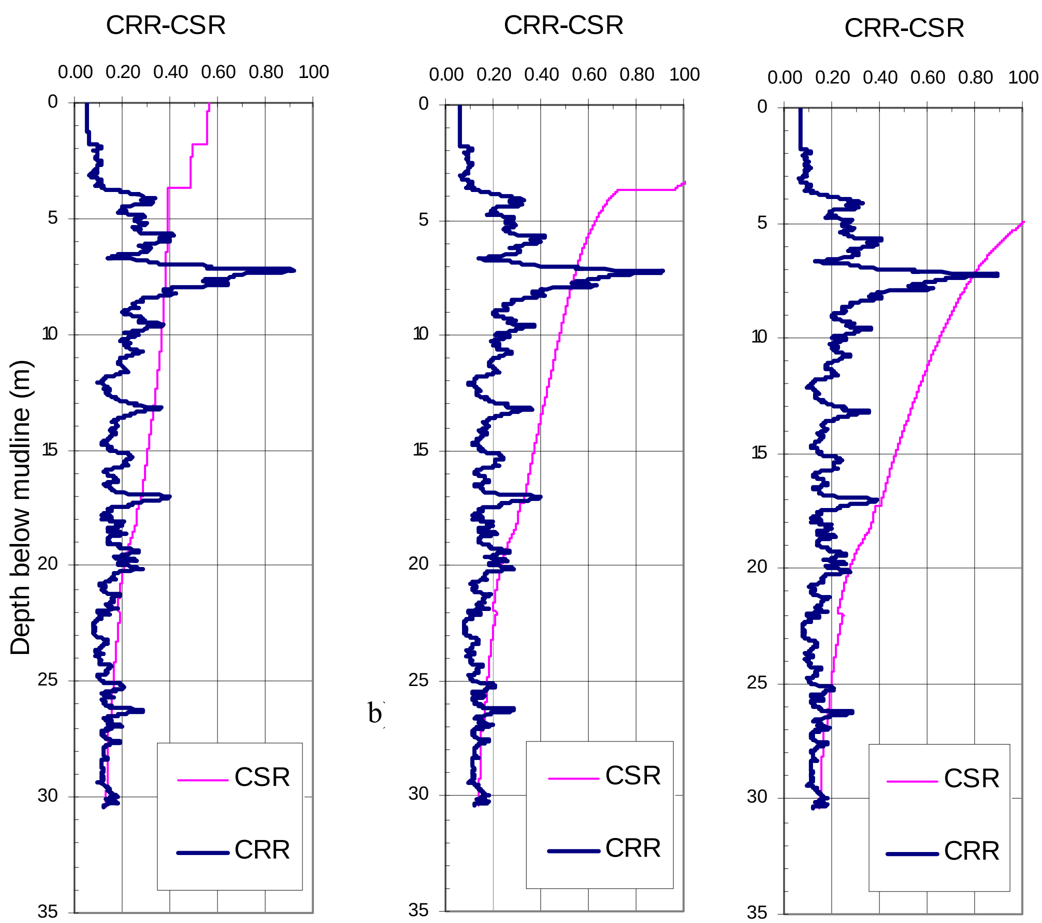


Fig. 4 Cyclic Resistance Ratio (CRR) and Cyclic Stress Ratio (CSR) along depth for a representative CPT (1:475 year earthquake) at different water depths (H); a) H= 0; b) H= 6 m and c) H = 15 m.

4. GLOBAL STABILITY

Global stability analyses were conducted using the software program Slope/W (2004) to assess the extent of densification required to stabilize the shoreline and dock. The analyses were conducted for a section through a representative CPT using a pseudo-static approach (immediately after an earthquake that induces soil liquefaction). The analyses contemplated residual strength for zones of liquefiable soils using the method provided by Seed and Harder (1990). Soil stratigrophy beneath the proposed dock structure is shown on Figure 5. The soil parameters used in the analysis for the different zones is presented in Table 1.

A sensitivity analysis was conducted to determine zones of required densification. The analysis indicated that the depth of densification zone is more critical than the width of densification zone. The analysis contemplated soil densification to El. –28.0 (28 m depth below water) over a width extending from 15 m south of the fender line to about 20 m behind the top of slope. The ideal total width of densification was found to be 75 m, which is about twice the width of the slope. The analysis indicated a factor of safety (FoS) against slope instability of about 1.1, which indicates that no excessive deformation is expected after the end of earthquake excitation. The most efficient method of soil treatment to relatively deep depth is vibroflotation, which is the densification of soil by vibration and compaction of backfill material.

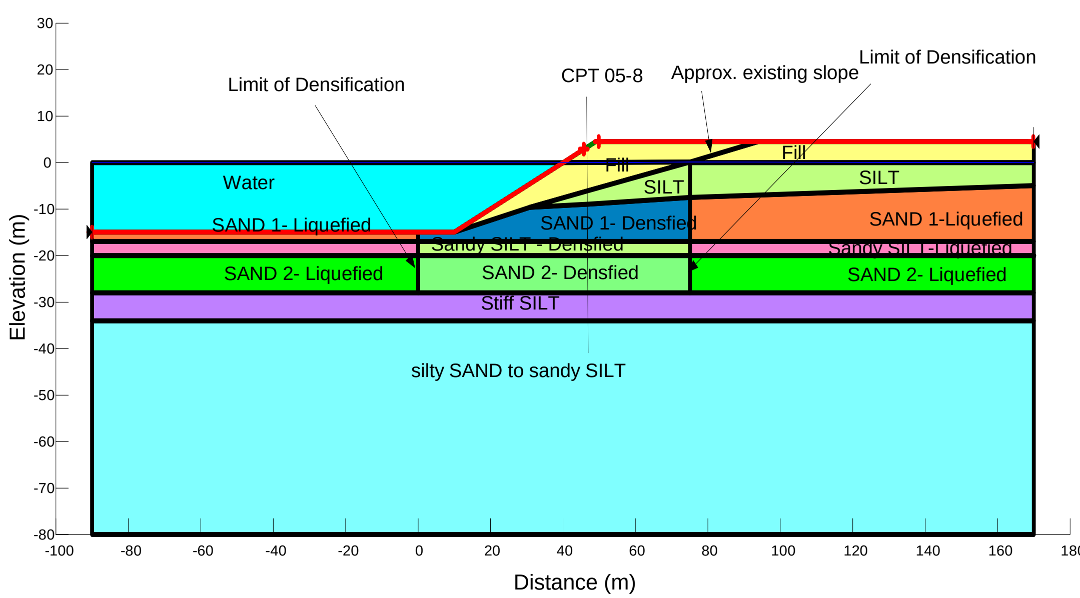


Fig. 5 Soil Stratigrophy beneath the proposed dock

Table 1: Soil Parameters used in the global stability analysis

Soil Type	**Unit Weight (kN/m^3)**	**Undrained Shear or Residual Strength (kPa)**	**Friction Angle**
FILL	19	0	34
SILT	17	30	0
SAND 1- Liquefied	18	15	0
SAND 2- Liquefied	18	30	0
SAND 1- Densfied	18	0	36
SAND 2 – Densfied	18	0	36
Stiff SILT	18	75	0
Silty SAND/sandy SILT	18	30	28

The required volume of densification along the dock length was calculated to be about 1,550,000, 1,700,000 and 1,800,000 m^3 for the 1:100, 1:475 and 1:2475 year earthquakes, respectively. The difference in volume of densification is attributed to the difference in the required depth of densification along the dock length for each seismic event.

5. CONCLUSIONS

In this paper, a case study of the stability of a dock structure founded on liquefiable soils is presented. An intensive CPT testing over water was conducted to assess the soil liquefaction. The following conclusions can be drawn:

1- The intensity of liquefaction increases with the increase in earthquake excitation.
2- Liquefaction increases significantly with the increase in water depth above seabed.
3- The global stability analysis is more sensitive to the depth of densification zone than the width of densification zone.
4- This study indicates that a densification zone equivalent to about twice the width of the slope under the dock is required to avoid the structural collapse during a major seismic event.

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