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MITIGATION OF SEISMIC DEFORMATION OF ANCHORED QUAY WALL BY COMPACTING

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ABSTRACT

The anchored quay walls or bulkheads are commonly used in coastal areas. Previous studies on the seismic behavior of these quay walls have shown a significant effect of the liquefaction on the performance of the walls. Considerable length of an anchored sheet pile wall in Rajaii port, Iran, has been embedded in liquefiable sand. This study tries to clarify failure mechanism of the wall during earthquake and to identify more effective zone for improvement of the wall stability. Numerical modeling by DIANA software and physical modeling by shake table were presented. Results show that the main reason for extensive deformation of the system is the liquefaction of the soil adjacent to the root of the wall. The compaction of this zone improves the performance of the system and prevents large displacements. The mitigation plan was proven by comparison among the measured results, including the final shape, displacements and excess pore water pressure ratio.

INTRODUCTION

In order to reduce damages to existing strategic structures during upcoming earthquakes, additional site investigation has recently been done on the site of Rajaii quay wall, Iran. This reinforced concrete wall is a sheet pile one and has totally ۳۵m height. The wall was anchored to concrete tie back of about ۱۳m height by cables.
Exploration shows that along some considerable length, the wall has been embedded in a loose silty sand layer, which is susceptible to liquefaction. It seems that the behavior of the wall is strongly dependent on this liquefiable layer. Therefore the performance of the wall during earthquake was studied numerically and experimentally.
The main objective of this study is to identify the failure mechanism of the wall and to propose appropriate improvement zone. Numerical modeling was conducted using DIANA finite element software (TNO DIANA, ۲۰۰۹) and experimentally Investigation was done using shaking table facilities of university of Tehran. Details of each study are presented in following parts.

NUMERICAL MODELING

In order to study liquefaction, authors used Towhata-Iai model developed based on generalized plasticity theory. The main feature of this model is that the concept of the multiple mechanisms, within the framework of plasticity theory defined in strain space, is used as a vehicle for decomposing the complex mechanism in to set of one dimensional mechanism. Using the model it is possible to obtain incremental and integrated constitutive relations, (Iai et.al, ۱۹۹۰a) and (Iai et.al, ۱۹۹۰b).

All dimensions, material properties and pretension loads are selected based on the simulating producer of shaking table test on soil-structure- fluid model in ۱g field published by (Iai, 1988), having the scaling factor of ۱/۷۰.

Regarding to construction stages of anchored sheet pile walls and in order to simulate initial stresses, the "phased analysis" method was adapted. During the first phase, a level ground model subjected to self weight loading was analyzed to obtain vertical and horizontal stresses using K_0 condition. In this stage Mohr-Columb criterion and drained condition were used. In the second phase the sheet pile wall and the tie back were embedded in the soil as infinite shell elements. After equilibrium under gravity loading, the soil in front of the wall was removed to the level of the anchor. In the fourth stage two equal loads having opposite directions were applied to pair nodes having zero distance from each other, on the wall and on the anchor. The model was analyzed to obtain stresses due to pretension loading then remained soil elements in front of the wall was removed to seabed level. For all these stages Mohr-Columb criterion and drained condition were used.

The stresses obtained in the end of these phases were used as initial stresses in the following dynamic analysis. In order to separate the dynamic phase results the strains of the static phases were suppressed and undrained condition was activated. As constitutive model for dynamic phase the Towhata- Iai model was used.

The soil mass was modeled using QU۸EPS elements, which are four-node quadrilateral isoparametric plane strain elements. They are based on linear interpolation and Gauss integration. Diaphragm wall and tie back were modeled by infinite shell, CL۹PE elements, while the anchor was modeled by a node-to-node spring, SP۲TR element, (TNO DIANA, 2009).

According to (Kramer ۱۹۹۶) the maximum spatial element size was selected based on the wavelength associated with the frequency through the loose sand.

While translating freedom of the base boundary was fixed at horizontal and vertical directions, the vertical sides are only fixed at horizontal direction.

Model parameters used for dynamic phases are outlined in Table ۱. Here, K_{ref} and G_{ref} are bulk and shear modulus, respectively that are given for ($-\sigma'_{ref}$) =۹۸ kPa. φ_f and φ_p are the shear resistance angle and phase transformation angle of the soil, respectively and S۱, w۱, p۱, p۲, c۱ are dimensionless dilatancy parameters of the soil. (Iai et.al, ۱۹۹۰b).

Table 1. Material Parameters for Towhata – Iai Constitutive Model

Soil DR%	K_{ref} (MPa)	G_{ref} (MPa)	φ'f	φ'p	P۱	P۲	W۱	S۱
85%	19.5	7.5	38	23	0.3	15	20	0.005
25%	1.82	0.7	32	23	0.2	30	5	0.005

The geometry of numerical model is presented in Fig.۱. Fig. ۲ shows the applied base acceleration time history for numerical and physical models.

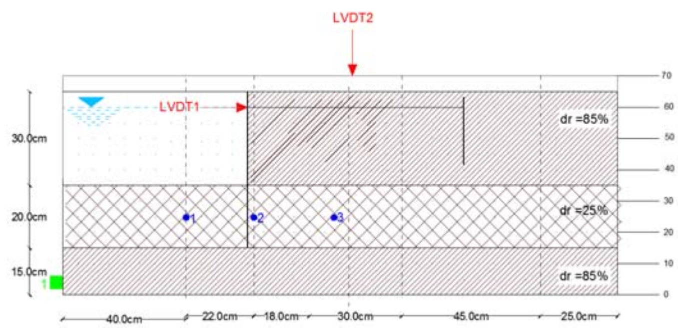

Fig 1. Geometry of numerical model and Test ۱.

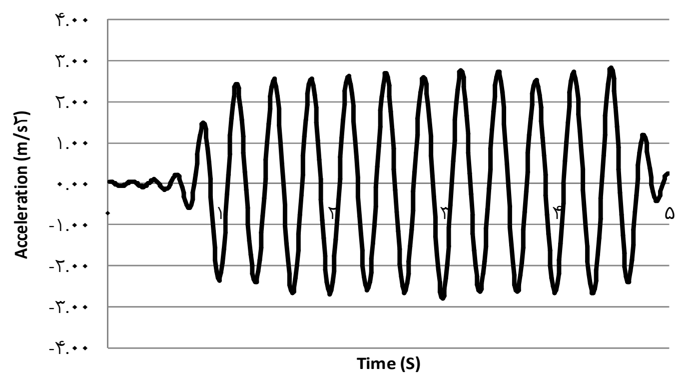

Fig 2. Base acceleration time history.

As shown in Fig.۳ the final mesh of the numerical model indicates large heave of seabed in front of the wall, grater displacement of bottom of the wall than that of the top and seaward movement of the tie back.

This mode of failure may be considered as a critical type and it is essential to be prevented by mitigation methods.

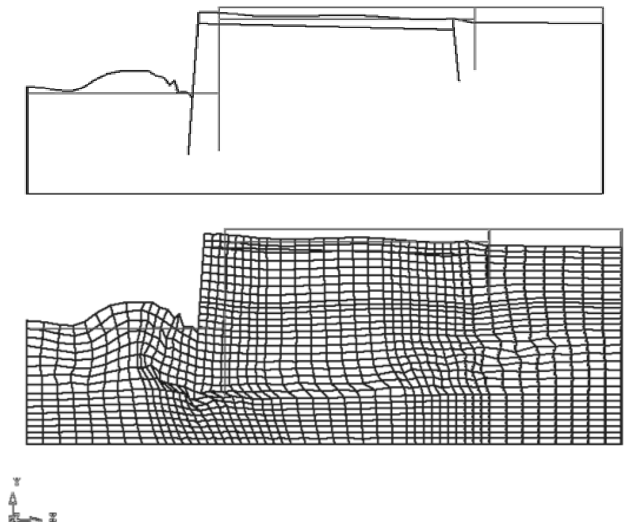

Fig 3. Deformed mesh of numerical model.

Diagram of resultant displacement of node ۱ and ۲ in Fig. 2, is presented in Fig. 4.

In Fig.5 shear strain of element ۱ and element ۲ are shown. These elements are the nearest elements to node ۱ and ۲, respectively.

According to diagrams, although the total displacement of points ۱ and ۲ are approximately equal, the shear strain of point ۱ is much bigger than that of the point۲.This comparison shows that the main reason of the displacement of soil located backward of the wall root is the extensive deformation of the soil located in front of the wall.

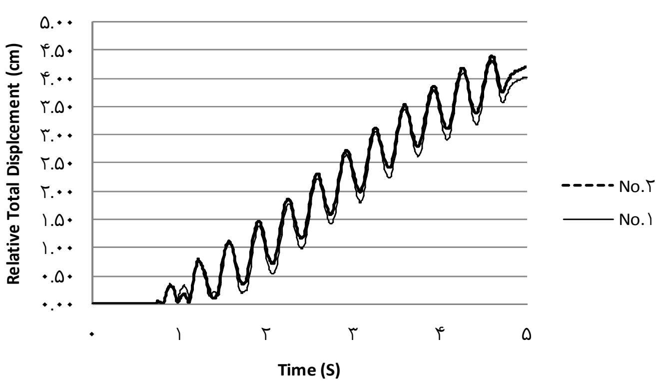

Fig 4. Numerical total displacement of node ۱ and ۲.

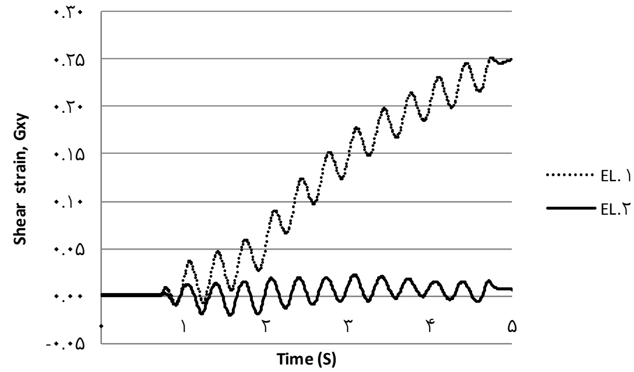

Fig 5. Shear strain of elements ۱ and ۲.

Following the deformation of this zone and sliding of backward soil of wall root, extensive settlement occurs in backfill surface. Simultaneously tie back loses its support and moves to seaward. The movement of the tie back prevents tie rods to perform appropriately.

Consequently it seems that such mitigation measures that improve the soil in front of wall root will be more efficient and more economic.

PHYSICAL MODELING

In order to confirm the numerical results, a physical model was tested using shake table.

In first model a layer of ۱۵ cm thickness of Firuzkooh sand was compacted to Dr=۸۵٪ , according to the wet temping method. Then the wall elements were placed and foundation layer was compacted in four ۵-Cm layers having Dr=۲۵٪ . Both sides of the wall were filled by the sand with Dr=۸۵٪ from the seabed level to the bottom level of anchors. Following e placement of anchors, filling was continued to the level of the tie rods. After the connecting of the rods to the wall and anchors, pretension loads were applied to the cables and filling was continued to the ground level. Following the excavating of soil in front of the wall, system was saturated by water. The geometry of SPM۱ is identical to that of the numerical modeling, as shown in Fig.۱.

Verifying the deformed mesh of numerical model, ultimate shape of SPM۱ proves that the main deformation is concentrated at loose layer, Fig.4.

The tilting of the wall and tieback, as well as the settling of the backfill are similar to the deformed mesh of numerical analysis.

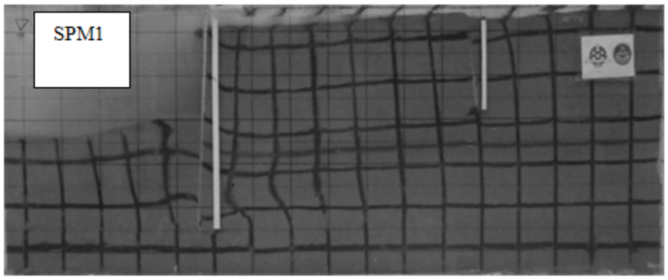

Fig 6. Deformed shape of SPM ۱.

As a mitigation plan, the second model was prepared. Construction manner of the second model, SPM۲, is similar to that of SPM۱, except for the foundation layer. The second model was improved by compacting of the liquefiable layer in both sides of the wall root. Geometry of SPM۲ is shown in Fig. 5. The recorded data and its interpretations to show the model behavior are presented in next section.

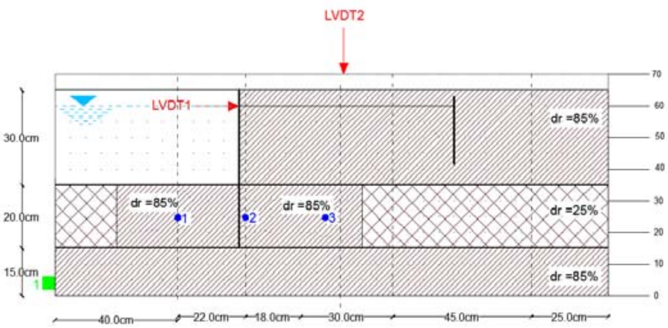

Fig 7. Geometry of SPM ۲.

RESULTS & OBSERVATIONS

Deformation

According to Fig.6 that shows the model SPM۲ after shaking, compaction of adjacent area to the wall root causes extremely reduction in the displacement of the foundation and the wall. Beside this negligible deformation, the displacement of the tieback is limited considerably, too.

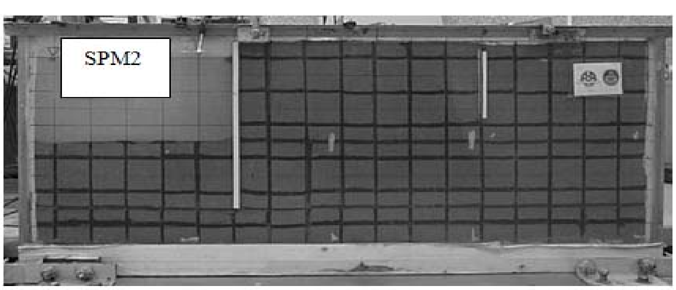

Fig 8. Deformed shape of SPM ۲.

Excess pore water pressure ratio

Diagram of excess pore pressure ratio, $r_u = \frac{u}{\sigma'_v}$ of point ۱ and ۲ proves the efficiency of the mitigation plan. Based on the Fig. ۶ the pore water pressure ratio of soil near transducer PWP۱ builds up to ۱ after about ۱.۵second of shaking. This ratio indicates that the soil at this zone liquefies. Following liquefaction of this part, extensive heave are observed.

The compaction of this area reduces excess pore water pressure significantly and prevents liquefaction of the soil.

Although the r_u in PWP۲ does not reach to ۱ during shacking, compacting of mentioned region reduces the pore pressure at this zone, too.

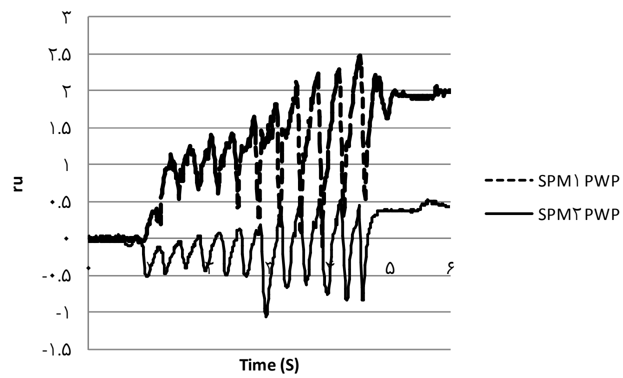

Fig 9. Time history of excess pore water pressure ratio, PWP ۱.

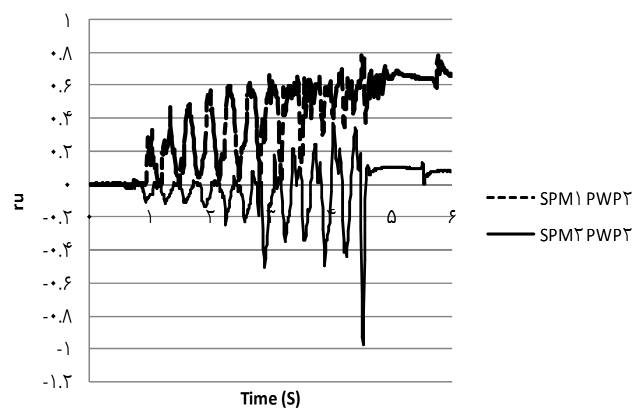

Fig 10. Time history of excess pore water pressure ratio, PWP ۲.

Lateral displacement of the wall

The data of LVDT1 of SPM۱ indicates that the top of the wall will displace about ۴.۵cm, i.e. about ۱۲٪ of free height of the wall, permanently. The mitigation measure decreases the lateral displacement of the wall to about ۰.۵ cm.

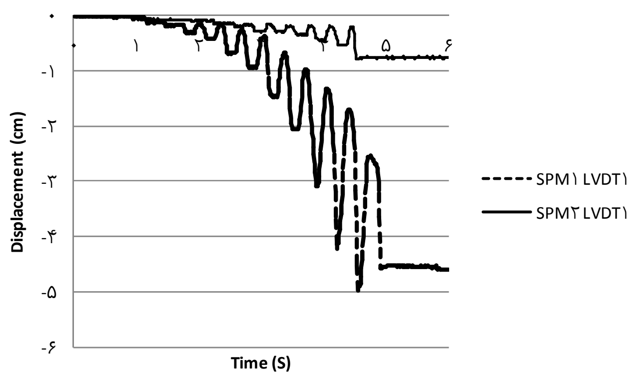

Fig 11. Time history of lateral displacement of the wall, LVDT1.

Settlement of the backfill

According to the Fig.10, compaction of mentioned zone will limit the settlement of the backfill, significantly. While the permanent settlement of backfill is about ۵ cm in SPM۱, it is only about 3 mm, in SPM۲.

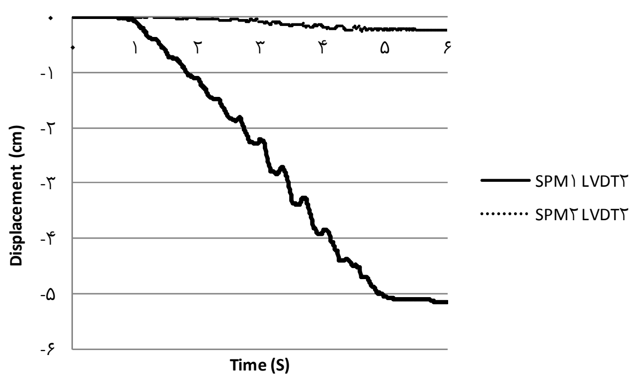

Fig 12. Time history of displacement of the backfill, LVDT2.

CONCLUSIONS

Effects of liquefiable layer in foundation of an existing anchored quay wall in Iran were studied numerically and experimentally from the liquefaction occurrence point of view. Results prove that:

1. Dynamic behavior of the wall is strongly dependent on the performance of liquefiable layer which in the wall has been embedded.
2. The most deformable zone is located adjacent to the root of the wall.
3. Due to the liquefaction of this zone and consequently displacement of backfill, tie back fails and tie rods does not work properly.
4. As a mitigation plan, compaction of this zone may cause considerable improvement of the system.

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