A case-history of damaged piles due to lateral spreading: evaluation by a simplified analysis

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ABSTRACT: The damage features of the spherical storage tanks at the time of the 1995 Kobe earthquake were introduced, together with the in-situ soil conditions. In an aim to clarify the cause of the distress, back-analysis was made for the foundation piles based on a simplified model proposed previously. The results of the analysis disclosed that the bending moment near the pile head and at the depth of the boundary between the liquefied and the underlying unliquefied layer reached values close to or in excess of the moment at yielding or at failure of the pre-cast reinforced concrete piles.

1. INTRODUCTION

In the 1995 Kobe earthquake, a majority of piled foundations in the waterfront area suffered damage due to the lateral spreading of the ground. Several cases of the pile injury during the Kobe earthquake were addressed and examined by Ishihara (1997) including detailed in-situ investigation on the damage features, soil conditions and ground displacements. Furthermore, many attempts have been done to clarify the mechanism of the pile damage by means of large-scale shake table tests and centrifuge tests. New concepts and analysis procedures have also been proposed in an effort to explore design methodologies for piles undergoing lateral spreading. These include the works by O'Rourke et al., (1994), Tokimatsu and Asaka (1998), Hamada (2000), Yasuda and Berrill (2000). Recently, Cubrinovski and Ishihara (2004) developed an equivalent linear method to analyze the response of piles undergoing the lateral displacement. In this method, the pile is assumed to be embedded in a deposit consisting of three soil layers in which soils in each layer and the pile itself are postulated to obey bi-linear and tri-linear load-deformation relationships, respectively. This method of analysis will be applied to back-analyze and elucidate the cause and mechanism of the damage to the pile foundations beneath spherical tanks which occurred at a tank farm located in Mikagehama Island.

2. STORAGE TANKS IN MIKAGEHAMA

There are several tank farms for storage of oil and liquid propane gas (LPG) in the man-made islands in the port area of Kobe. Storage tanks with various capacities were shaken very severely and affected more or less seriously by the ground damage due to liquefaction (Ishihara, 1997). Among several farms, the storage tanks and facilities in Mikagehama Island were those severely damaged and hence investigated in details as to the deleterious effects caused by the liquefaction of the ground. The location of Mikagehama Island is shown in Fig. 1. The triangular section in the southeast part shown in Fig. 2 is occupied by tanks and other related facilities for storage of LPG.

As a result of extensive liquefaction, lateral spreading took place over the island causing cracking, large settlements and ground distortion on the premises of the LPG tank yard. The overall feature of the ground devastation is demonstrated in the sketch of Fig. 2. It may be seen that there are depressions just behind the quay walls accompanied by large ground movements. Numerous fissures or cracks are seen in the figure having developed farther inland from the waterfront.

2.1. Soil conditions

Site investigations including SPT and tube sampling were conducted before and after the earthquake at various locations on the premises. One of the soil profiles is shown in Fig. 3 where it may be seen that the man-made Masado deposit consisting of sand with silt and gravel exists down to a depth of about 17 m. The SPT blow count in the Masado layer is low on the order of 5 to 15 and as such the reclaimed soils were under conditions susceptible to liquefaction. Underneath the reclaimed deposits, a layer of silty clay exists which was the seabed deposit before the reclamation was made in 1960's. The gravel-containing silty sand in the reclaimed deposits had developed liquefaction due to the intense shaking during the earthquake.

2.2. Lateral spreading

After the earthquake, lateral spreading of the terrain was investigated by the method of ground surveying (Ishihara et al., 1997). Four alignments chosen for surveying in the tank farm are shown in the inset of Fig. 4. The features of ground distortion observed along the cross section M-3 nearest the tanks TA106 and TA107 are displayed in Fig. 4 where it may be seen that the lateral displacement was manifested on the ground surface to a distance of 95 m inland from the waterfront.

2.3. Spherical Tanks

The subject of this study is the LPG tank TA107 which is located in the southern part of the island, 18 m inland from the revetment line on the south (Fig. 2). The tank has a diameter of 7.69 m and storage capacity of 100 kl. The tank is supported by 6 columns and the footing of each column is supported by four pre-cast reinforced concrete piles (PC-pile) 30 cm in diameter, as shown in Fig. 5 where plan view of the foundation is shown. The piles were driven to a stiff soil stratum at a depth of 20 m.

3. DAMAGE FEATURE

Features of the damage to the pile foundation of tank TA107 were investigated by lowering a video camera down the hole of one of the hollow-cylindrical piles to examine development of cracks around the pile-wall. Results of

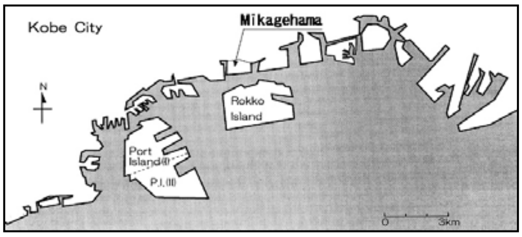

Fig. 1 Location of Mikagehama Island in the port area of Kobe

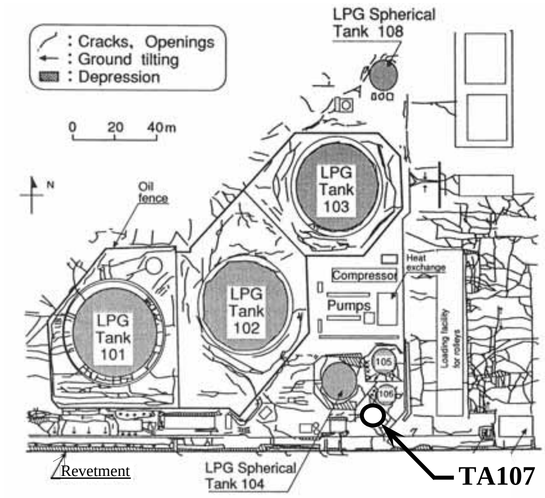

Fig. 2 Ground distortion in the LPG tank farm

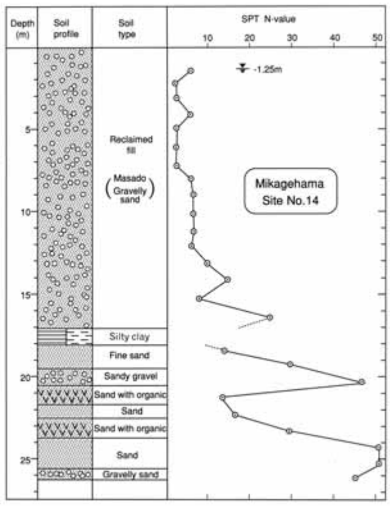

Fig. 3 Soil profile at site P14 near the spherical tank

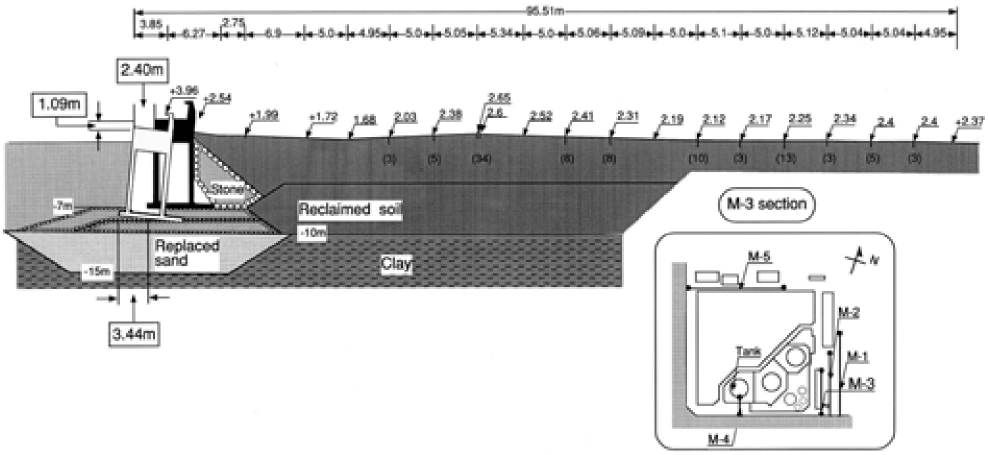

Fig. 4 Quay wall movement and ground distortion in the backfills along section M-3

the survey are presented in Fig. 6 where it may be seen that cracks developed predominantly at depths between 5 and 10m. Unfortunately, it was not possible to pull down the camera lower than 10m because of clogging of the hole. In addition, survey of pile deformation was carried out by lowering an inclinometer successively down the hole. By integrating the measured data on tilts through the depth, configuration of deformed piles was obtained as shown in Fig. 6a where the relative magnitude of lateral deformation is displayed. It is to be noted that the displacement of 50 cm shown in the figure is the horizontal deflection of the pile between the pile head and pile point at the depth of 10 m below the pile top and that the pile displacement below this depth is not known. It is likely however, that the pile must have moved almost in unison with the surrounding soil, and if this assumption is valid, the pile head movement is estimated to have been about 150 cm, as shown in Fig. 7. Unfortunately there is no documented record as per visual observation of injury at the pile-head and around its connection to the footing.

4. BACK-ANALYSIS FOR THE TANK

4.1. Method of analysis

It is considered sufficiently simple and yet competent enough to represent the pile behavior undergoing lateral flow by means of the three-

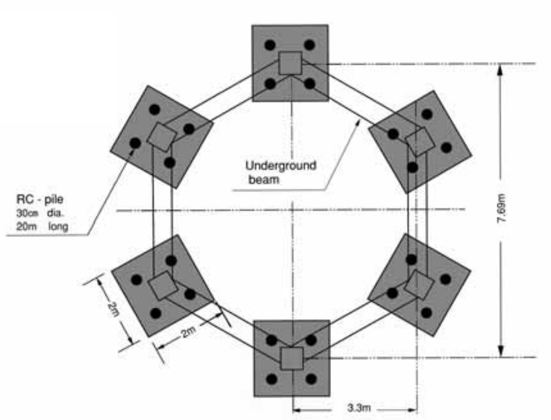

Fig. 5 Plan view of the foundation of spherical tank

layer model as shown in Fig. 8 (Cubrinovski and Ishihara, 2004). In this model, the pile is represented by a continuous beam and it is postulated that the non-liquefied layer near the ground surface and the base layer exhibit a bi-linear load-displacement relations. The liquefied soil, on the other hand, is assumed to be represented by an equivalent linear p-δ relation or secant stiffness $\beta_2 k_2$ that takes into account the significant reduction in stiffness due to soil liquefaction by way of the degradation factor β_2. When the liquefied deposit in the middle layer moves laterally, the surface unliquefiable soil layer is assumed to move together. Thus, the pile is subjected to the lateral pressure p_1 at the top portion. The pile is pushed in the direction of ground movement also by the liquefied soils undergoing lateral spreading. The displacement

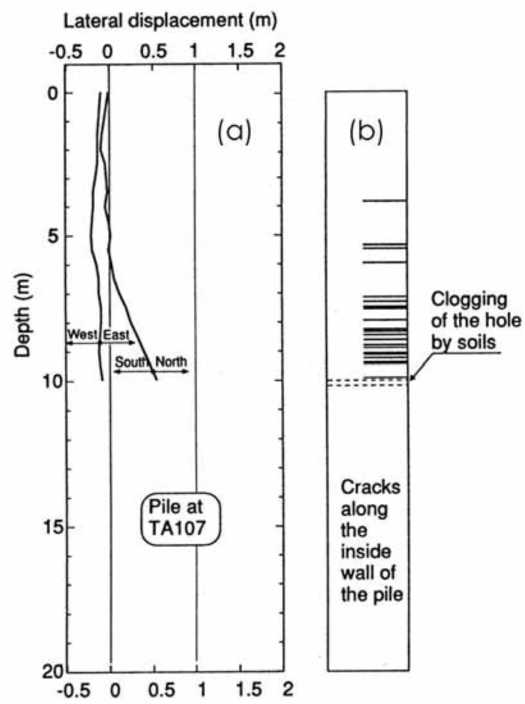

Fig. 6 Observed displacements and damage to the pile: (a) Lateral displacements; (b) Cracks as identified by bore-hole camera

of the ground is assumed to have a cosine function with lateral displacement at the top of the liquefied layer, U_{G2}, as indicated in Fig. 8.

Even though nonlinear behaviour is considered in the three-layer model, the method is based on a closed-form solution for linear behaviour and use of the equivalent linear modelling approach. Details of the analysis method including derivation of the closed-form solution, development of equivalent linear models, overall calculation procedure and determination of model parameters can be found in Cubrinovski and Ishihara (2004).

In performing the analysis with the simplified model, it is first necessary to give the lateral displacement of the ground as an input. This can be done with reference to the measured data shown in Fig. 7. As shown in this figure, the lateral displacement is read off as having been 1.5m at the location of the tank in question. The subgrade reaction coefficients of k_1 = 2.4 MN/m^3, k_2 = 21.8 MN/m^3 and k_3 = 13.1 MN/m^3 were evaluated based on an empirical correlation between k and the SPT blow count N.

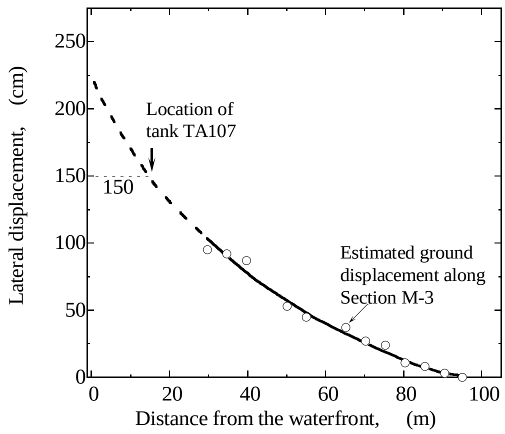

Fig. 7 Lateral ground displacement versus distance from the waterfront at the site of tank TA107 as estimated from the ground surveying

The bending stiffness of the pile is represented by a conventional trilinear relation between the bending moment, M, and curvature, ϕ. Schematic plot of the adopted analytical model for the piles is shown in Fig. 9. Note that the analysis was made for a single pile and that F represents the resultant force which is generated from the pressure of the surface layer on the footing. In calculating this force it was assumed that the total force acting on the footings and horizontal beams (over width of 7.2m) is evenly distributed to all 24 piles, each of the piles thus receiving an amount of 1/24 of the total force.

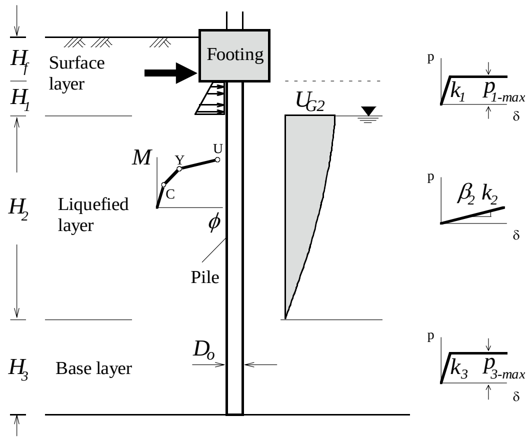

Fig. 8 Three-layer model for simplified analysis of piles subjected to lateral spreading

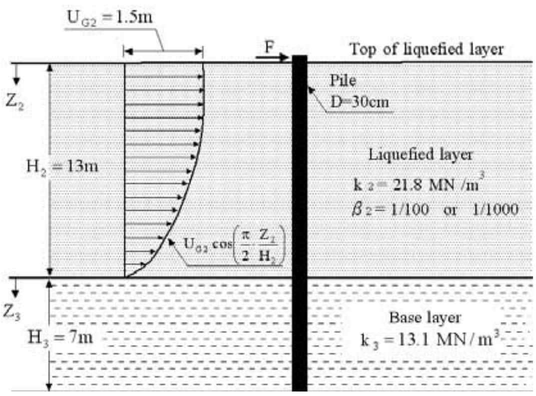

Fig. 9 Soil-pile model used in the back-analysis

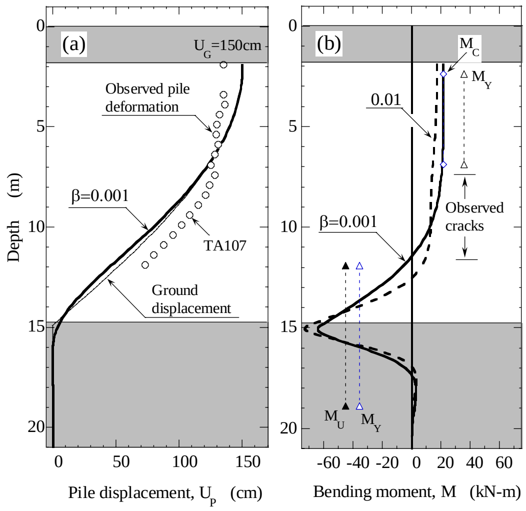

Fig. 10 Computed displacements and bending moments

5. RESULTS OF BACK-ANALYSES

Following the above procedure, the pile response was evaluated in this study for an assumed lateral displacement at the ground surface of U_G = 150 cm and for the stiffness degradation in the liquefied soil of β_2 = 1/100 and 1/1000.

The outcome of the back-analysis is shown in Fig. 10 in terms of the pile displacement and bending moment plotted versus depth. The influence of the stiffness degradation parameter β_2 on the pile displacement was found to be negligibly small and therefore the computed pile displacement is shown only for the case of β_2 = 0.001 in Fig. 10a. The pile displacements monitored by the inclinometer are quoted from the data in Fig. 6 and shown superimposed in Fig. 10a. Note that the in-situ measurements were started downwards from the exposed top of the pile after excavation and taking off the footing. Therefore, the zero depth in Fig. 6 is actually at the depth of 1.9 m from the ground surface. Thus, the monitored pile displacement is shown in Fig. 10a so that it coincides with the depth from the ground surface. It is noted that the monitoring was done only down to the depth of 12 m. In addition, the result of the back-analysis indicated that the lateral displacement of the pile was about 50 cm in the lower part of the liquefied layer, say, between the depths of 12-15m where the monitored data are unfortunately missing. Therefore, the measured pile displacement is displayed in Fig. 10a after it was adjusted so that the measured values at the upper part of the pile are in agreement with the computed displacements. It is to be noted that the comparison would hold valid only for the relative pile displacement in the depth range between 1.9 m and 12 m. Under this limitation, it may be mentioned that the coincidence between the measured and back-calculated pile displacements is satisfactory. It should be mentioned again that there must have been fairly large pile deformation in the lower part of the liquefied layer, but there is no way, unfortunately, to confirm it.

With respect to the bending moment, the computed value is plotted in Fig. 10b for the case of β_2 = 0.01 and 0.001, together with the characteristic bending moments of the pile at cracking, M_C, yielding, M_Y, and failure, M_U. The depth range at which cracks were observed by the video camera inspection is also indicated in Fig. 10b. It may be seen that the computed bending moment is slightly below the bending moment required to cause cracking and in this regard it is not exactly consistent with the observed development of cracks. The reasons for this may be speculated in some ways as follows. In the above analysis, the lateral load at the pile top has been applied in the direction of ground movement because the scenario envisaged was that both the surface layer and liquefied layer move together in the direction of lateral spreading and that horizontal resistance is mobilized merely by the unliquefied soil in the base layer. This scenario may be regarded true for the case of a single pile standing in the free field without any interaction with other structural members. In the case of TA107, each footing is connected by horizontal beams to adjacent footings in a hexagonal arrangement, as shown in Fig. 5. Thus, there must have been some kind of interaction between these structural elements to generate more complicated load-transfer mechanism which could exert some influence on the pile, particularly, on its upper part near the footing. This type of interactive loads may be obtained if analysis is made considering the interaction amongst structural members and surrounding soils such as that performed by Tokimatsu and Asaka (1998). It is also likely that inertial effects during the main shaking as well as axial loads of the top-heavy tank played some role in the deformational behavior and damage to the piles.

Regarding the bending moment in the lower part of the liquefied layer, it is seen in Fig. 10b that the computed value becomes greater than the ultimate moment at the interface between the liquefied layer and underlying base soils thus indicating that the pile suffered the largest damage at this depth. As mentioned earlier, however, the in-situ inspection of the pile could not reach this depth and therefore it was not possible to directly confirm whether this type of damage occurred or not. It is to be emphasized here, however, that the development of injury on piles near the boundary between liquefied and unliquefied deposits is a characteristic feature of the damage during lateral flow which was addressed by many investigators after the Kobe earthquake in 1995.

6. CONCLUSIONS

The damage to the pile foundation of a spherical tank during the 1995 Kobe earthquake was studied by applying a simplified methodology for back-analysis. The outcome of the analysis was compared with the results of in-situ investigations on the damage features of the piles carried out after the earthquake. The results of the study are summarized as follows.

1. The pre-cast reinforced concrete piles 30 cm in diameter and 20 m long supporting the spherical LPG storage tanks had suffered severe damage at the time of the Kobe earthquake. It was found that cracks had developed in the pile at the depths of 6-10 m below the pile top. The result of the back-analysis showed that the computed maximum bending moment at this depth was slightly below the moment required to cause cracking of the pile. Other loads and deformation patterns were suggested as possible causes for the damage to the piles which can not be explained by a conventional single-pile analysis.

2. At the depth of 13-15m, the computed bending moment becomes greater than the bending moment required to cause failure in the pile body. Since in-situ investigation was not carried out down to this depth, there is no data to check whether this computed result is correct or not. However, in view of many other data available, it is conjectured that the injury to pile body must have occurred at this depth.

7. REFERENCES

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