Pushover analysis for piles subjected to liquefaction-induced flow pressure and ground displacement

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ABSTRACT: This paper presents a pushover analysis approach for piles exerted by liquefaction-induced flow pressure and ground displacement. The flow pressure is estimated from seismic design specification of Japanese Road Association and the ground displacement was predicted by the method proposed by Tokimatsu & Asaka (1998), and the non-linear behavior of the pile is modeled by a tri-linear moment-curvature relationship. The push over analysis was conducted by the software SAP 2000. The capacity curve of the lateral pile was expressed in terms of the total flow force imposing on the pile and the pile head displacement. Then, the seismic performances of the pile can be clearly identified on the curve when subjected to different design earthquakes which cause varying flow pressures and displacements. The proposed approach is simple and clear, so it can be easily applied to perform the seismic performance-based design of pile foundation in flow ground.

1 INTRODUCTION

Performance-based design has been the future trend of the seismic design codes (CEN/TC250/SC8 2000, JRA 1996, SEAOC 1995, GBJ11-89 1989) in the world. In the recently developed seismic performance-based design (SPBD) approach, nonlinear analysis procedure plays a key role in identifying the damage patterns and levels for understanding the inelastic behavior and the failure mechanism of the structure during severe seismic events. Pushover analysis is a simplified, nonlinear, static, step-by step procedure where a predefined pattern of earthquake loads is applied incrementally to framework structures until a plastic collapse mechanism is reached. By this procedure, structure capacity is represented by a capacity curve. The most convenient way to plot the capacity curve is by tracking the base shear and the roof displacement of the structure during the pushover process. This curve can help engineers better understand how structures will behave when subjected to major earthquakes.

Numerous studies on SPBD of structures have been published in the past two decades. These research results have formed the bases of current SPBD codes in the world. Up to now, although there have been more and more SPBD-related studies focusing on structural and foundation engineering area, few have been devoted to liquefaction engineering field. To introduce the concept of SPBD to the problem of lateral pile response when subjected to liquefaction-induced flow pressure, this paper proposes a simple pushover approach for the problem. The major idea of the approach is based on that the capacity of the lateral pile can be obtained by tracking the total flow force exerting on the pile and the pile head displacement during the pushover process.

2 PUSHOVER APPROACH

The main procedure of this approach includes: (1) liquefaction analysis, flow pressure and flow ground displacement estimation, (2) set up nonlinear pile analysis model, (3) pushover analysis by software SAP2000 (SAP2000 V9 manuals 1995), and (4) plot the capacity curve of the lateral pile.

The procedures of (1) and (2) will be described in this section, and the procedures of (3) and (4) will be described in next section through a case analysis.

2.1 *Liquefaction induced flow pressure and ground displacement*

Liquefaction induced flow pressure

The liquefaction analysis and flow potential of a site can be assessed by the method suggested in the seismic specification of Japanese Road Association (JRA) in 2002. If a pile foundation site is identified to have high potential of lateral ground spreading, the JRA (2002) specification has proposed that the action of liquefaction-induced lateral spreading on the piles can be represented by the flow earth pressure, as shown in Figure 1, where q_{NL} and q_L are the linearly distributed earth pressures exerted on the piles by the non-liquefying and liquefying soil layers in the flow area. The earth pressures can be computed by Eqs. (1) and (2) as

$$q_{NL}(x) = C_s C_{NL} K_p \gamma_{NL} x \,, 0 \le x \le H_{NL} \,, \tag{1}$$

$$q_L(x) = C_s C_L \{\gamma_{NL} H_{NL} + \gamma_L (x - H_{NL})\},$$
$$H_{NL} \le x \le H_{NL} + H_L \,, \tag{2}$$

where:

$q_{NL}(x)$: Flow earth pressure (kN/m^2) of a non-liquefying layer acting on a pile at a depth of x;

$q_L(x)$: Flow earth pressure (kN/m^2) of a liquefying layer acting on a pile at a depth of x;

C_s: Modification factor based on the distance from the waterfront, as shown in Table 1;

C_{NL}: Modification factor for the ground flow force in a non-liquefying layer. The value according to liquefaction potential index P_L (m^2), which is defined by $P_L = \int_0^{20} (1 - F_L)(10 - 0.5x)dx$, is shown in Table 2 where F_L is the safety factor to resist liquefaction; when $F_L > 1$, set $F_L = 1$;

C_L: Modification factor for the force of the ground flow in a liquefying layer (the suggested value is 0.3);

K_p: Passive earth pressure coefficient using Coulomb's $K_p = (1+\sin\phi)/(1-\sin\phi)$;

γ_{NL}: Average unit weight (kN/m^3) of a non-liquefying layer;

γ_L: Average unit weight (kN/m^3) of a liquefying layer;

x: Depth below the ground surface (m);

H_{NL}: Non-liquefying layer thickness (m);

H_L: Liquefying layer thickness (m).

Table 1. Modification factor Cs based on the distance from the waterfront (JRA 1996)

Distance from waterfront, s (m)	Modification factor, C_s
$s \le 50$	1.0
$50 < s \le 100$	0.5
$100 < s$	0

Table 2. Modification factor CNL for the ground flow force in a non-liquefying soil layer (JRA 1996)

Liquefaction potential index, P_L (m^2)	Modification factor, C_{NL}
$P_L \le 5$	0
$5 < P_L \le 20$	$(0.2P_L - 1)/3$
$20 < P_L$	1.0

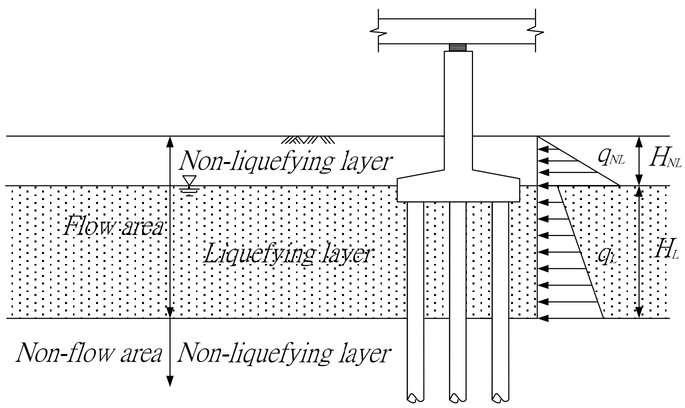


Figure 1. Analysis model of a pile group subjected to the flow earth pressures (JRA 1996)

To convert a pile group problem to a single pile one, some simplifying assumptions must be made. The assumptions include:

(1) The geometrical dimensions and material properties of all the piles are the same.
(2) The pile cap is perfectly rigid, so that the horizontal displacements of all the piles are the same.
(3) The flow earth pressure on every pile is the same.

Given the above assumptions, the analysis of a pile group can be simplified as the model of a single pile, as shown in Figure 2. Multiplying the flow earth pressures from Eqs. (1) and (2) by the effective foundation width B, we can obtain the flow forces per unit depth for the pile group. For a pile foundation, the foundation width B is defined as the distance between the external edges of the two outermost piles. The foundation width of the pier and the pile cap is the width of the pier and the cap. These definitions are shown in Figure 3. By dividing the total flow force per unit depth with the number of piles, one can obtain the flow force per depth of one pile. Thus, the flow forces per unit depth q_{n1}, q_{n2} (kN/m) at the top and the bottom of non-liquefying layer for each pile can be expressed as

$$q_{n1} = \frac{B \times (C_s C_{NL} K_p \gamma_{NL} D_f)}{N_{total}}, \tag{3a}$$

$$q_{n2} = \frac{B \times [C_s C_{NL} K_p \gamma_{NL} (D_f + h_{nl})]}{N_{total}}, \tag{3b}$$

The flow force per unit depth q_{l1}, q_{l2} (kN/m) at the top and the bottom of liquefying layer for each pile can be expressed as

$$q_{l1} = \frac{B \times \{C_s C_L [\gamma_{NL} (D_f + h_{nl})]\}}{N_{total}}, \tag{4a}$$

$$q_{l2} = \frac{B \times \{C_s C_L [\gamma_{NL} (D_f + h_{nl}) + \gamma_L h_l]\}}{N_{total}}, \tag{4b}$$

where:
B: Effective width (m) for computing the flow force;
D_f: Embedded depth (m) of the pile cap;
h_{nl}: Thickness of the non-liquefying layer (m) for a single pile;
h_l: Thickness of the liquefying layer (m) for a single pile;
N_{total}: Number of piles.

The flow forces per unit depth at any depth in the non-liquefying and liquefying layers can be linearly interpolated by Eqs. (5) and (6).

$$q_1(x) = q_{n1} + (q_{n2} - q_{n1})\frac{x}{h_{nl}}, 0 \le x \le h_{nl}, \tag{5}$$

$$q_2(x) = q_{l1} + (q_{l2} - q_{l1})\frac{x - h_{nl}}{h_l}, h_{nl} \le x \le h_{nl} + h_l, \tag{6}$$

where:
$q_1(x)$: Flow force per unit depth (kN/m) at a depth of x in a non-liquefying layer;
$q_2(x)$: Flow force per unit depth (kN/m) at a depth of x in a liquefying layer;
x: Depth (m) from the pile head.

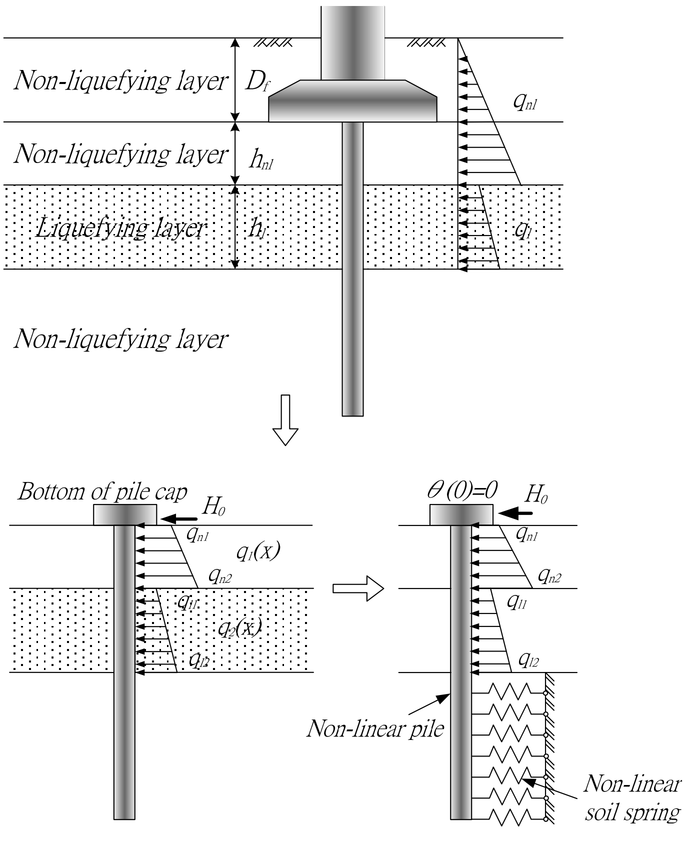


Figure 2. Analysis model of a single pile subjected to flow earth pressure

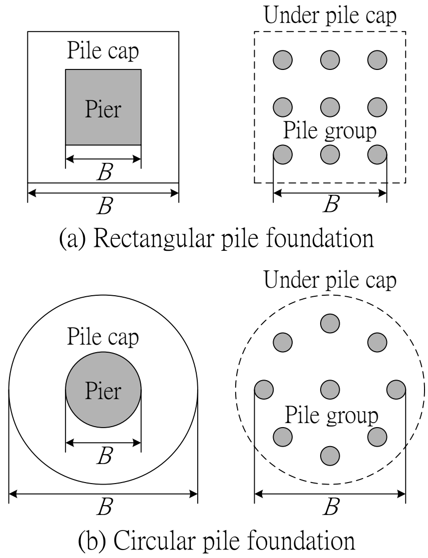


Figure 3. Effective width for computing flow earth pressures

Liquefaction induced flow ground displacement

The flow ground displacement can be estimated by the method proposed by Tokimatsu and Asaka (1988). The flow ground displacement was estimated by the following equations and shown in Figure 4.

$$u(x, s) = D(s), x \le x_w, \tag{7}$$

$$u(x, s) = D(s)\cos(\pi(x - x_w)/2H), x \ge x_w, \tag{8}$$

$$= D(s)(1 - (x - x_w)/H), \tag{9}$$

where:
$u(x,s)$: Horizontal displacement profile at the site with a distance s from the water front;
$D(s)$: Uniform displacement of the non-liquefying upper layer at the site with a distance s from the water front;
x_w: Depth of the non-liquefying upper layer;
x: Depth in consideration;
H: Thickness of the liquefying layer;
L: Horizontal range of flow ground displacement.

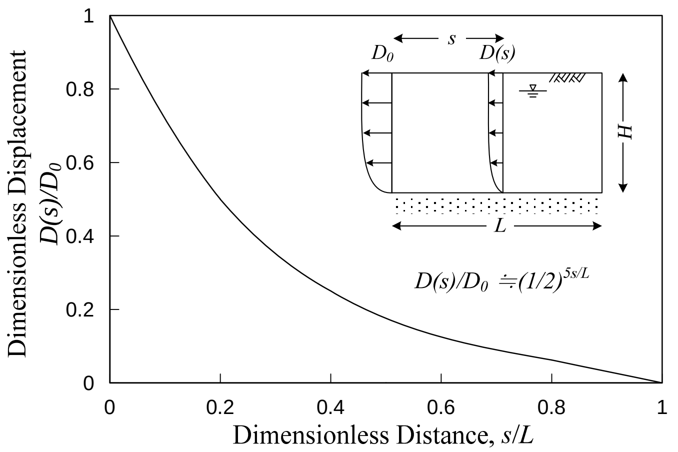


Figure 4. Relation of distance to water front and ground displacement

However, directly using the ground displacement of the upper non-liquefying layer in the pushover procedure can cause numerical problems. To solve this problem, an equivalent force was deduced and applied on the corresponding part of the pile in the upper non-liquefying layer according to the suggestion proposed by Cubrinovski & Ishihara (2004) that found the problem first. The pushover model using ground displacement is shown in Figure 5. In this model, the soil spring of the liquefying layer will be softened by multiplying the original spring constant with a reduction factor β which can vary in the range of 0.1-0.001.

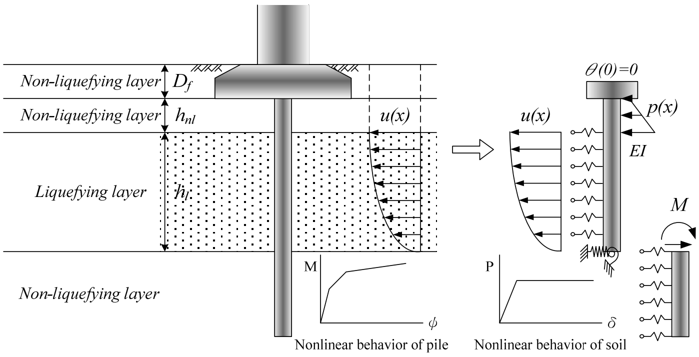


Figure 5. Analysis model of a single pile subjected to flow ground displacement

2.2 *Setting up nonlinear pile analysis model*

The modeling of a single pile subject to the action of lateral flow pressure can be simplified as a free standing pile with the lower part embedded in the underlying non-liquefying deposit and the upper part exerted by the flow pressures and ground displcement, as shown in Figures 2 and 5. A pile embedded in an underlying non-liquefying soil layer in a non-flow area can be simulated by a nonlinear pile laterally supported by a series nonlinear soil springs, while the pile in the flow area can be simulated as a nonlinear beam model either exerted by the flow pressures or by ground displacement with softening soil spring due to liquefaction effect and both with a zero rotation restraint at the top of the pile.
The nonlinear behavior of the pile is modeled by a tri-linear moment curvature relation and the nonlinear behavior of the underlying non-liquefying soil is modeled by an elastic-perfectly plastic soil spring. Any commercial programs having nonlinear beam-column and spring elements can be used to analyze this nonlinear soil-pile interaction problem.

3 CASE ANALYSIS

To demonstrate how to conduct pushover procedure using the above analysis model, one pile damage case that occurred during the 1995 Kobe earthquake was chosen as an analysis example. This case had been reported in detail by Ishihara & Cubrinovski (2004). In this case, the 69 pre-stressed high-strength precast concrete piles, supporting an oil-storage tank with a storage capacity of 2450 *kl* , were seriously damaged due to a lateral ground displacement estimated to be about 35- 55 *cm*. The piles are 23 *m* long and 45 *cm* in diameter. The moment-curvature relationship of the pile is shown in Figure 6, where D_0 is the diameter of the pile and N is the axial force on the pile. It can be seen in the figure, that the cracking moment capacity M_c, yielding moment capacity M_y and ultimate moment capacity M_u are approximately 105, 200, and 234 *kN-m*, respectively. The geological profile and the cross section of tank TA72 and its foundation are shown in Figure 7. Sand compaction piles have been installed around the perimeter of the tank foundation to strengthen the foundation soils. The depth of the improvement is approximately 15 *m*. A detailed field investigation using a bore-hole camera and inclinometer was conducted to inspect for damage to the No.2 and No.9 piles of the tank. The outcome is shown in Figure 8. The figures show that the piles developed multiple cracks and that the greatest damage occurred at a depth of approximately 8 to 14 *m*, which is about the depth of the interfacial zone between the liquefying deposits and the underlying non-liquefying silty layer. There was significant shear-induced damage at a depth of 14 to 15 *m* on the No.2 pile. The joint of the No. 9 pile seems to have broken and slipped at a depth of 10.5 *m* due to the large shear force.

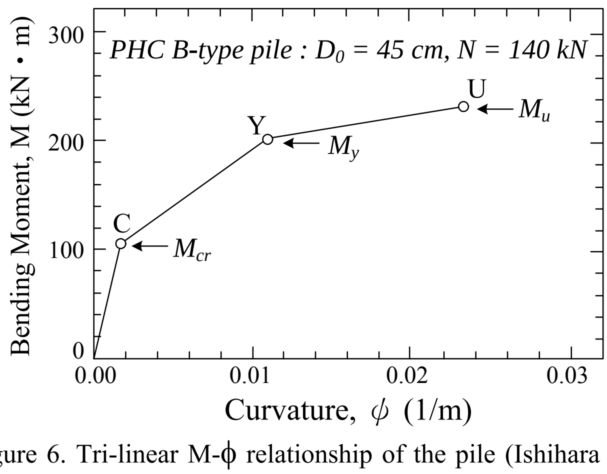


Figure 6. Tri-linear M-ϕ relationship of the pile (Ishihara and Cubrinovski 2004)

3.1 *Liquefaction evaluation-flow pressure and ground displacement*

The results of liquefaction analysis are shown in Table 3. It can be seen that the soil deposits from the surface to a depth of 13.5 *m* are in the flow area. The non-liquefying deposits go from the surface to a depth of 2.5 *m* and the liquefying deposits run from a depth of 2.5 *m* to a depth of 13.5 *m*. The soil deposits below 13.5 *m* are in the non-flow area. An

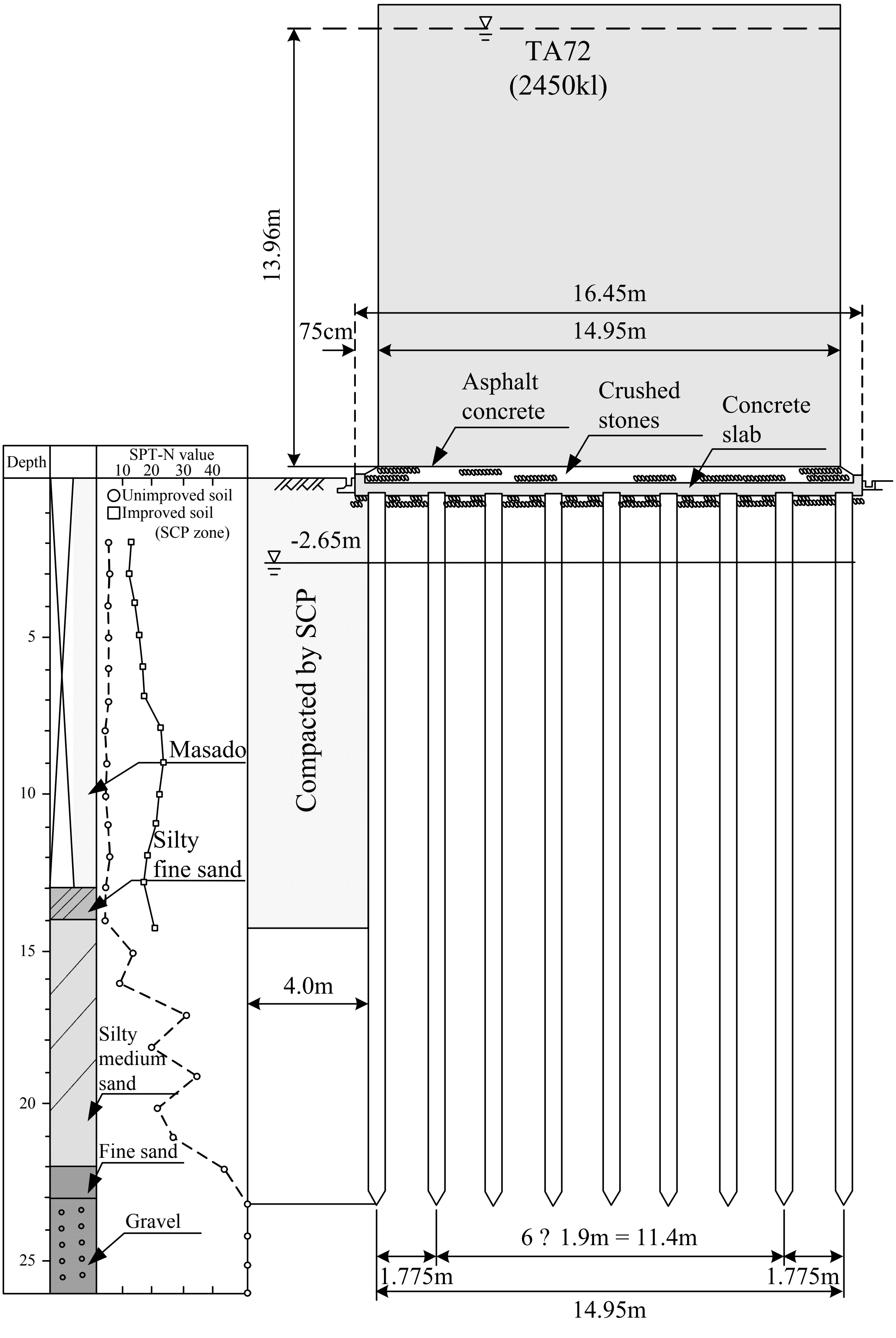


Figure 7. Cross sectional view of tank TA72 and its foundations (Ishihara and Cubrinovski 2004)

average value of three seismic records with a *PGA* of 0.4 *g*, is used in the analysis. We use the liquefaction assessment method proposed by the JRA (1996).

The computed liquefaction potential index P_L for the flow area is 16.54. In this case, the horizontal force acting on the pile head comes from the flow force at the top of the non-liquefying layer 0.5 *m* below the ground surface. The horizontal force is about 1.19 *kN* per pile. The flow forces per unit depth of the non-liquefying and liquefying layers in the flow area can be calculated by Eqs. 3 and 4. The unit flow forces at the top and bottom of the non-liquefying layer are 4.47 *kN/m* and 23.48 *kN/m*, respectively. The unit flow forces at the top and bottom of the liquefying layer are 3.01 *kN/m* and 16.27 *kN/m*, respectively. The surface horizontal displacement at this site was estimated to be about 50cm according to Cubrinovski & Ishihara (2004).

Table 3. Soil profile and the results of liquefaction analysis

Depth (m)	Soil type	SPT-N value	Unit weight (kN/m^3)	Friction angle (degree)	F_L	Liquefy or Non-liquefy	P_L
0.0~0.5	Masado Soil	7	18	29.1	-	Non-liquefy	-
0.5~2.5	Masado Soil	12	18	30.6	-	Non-liquefy	-
2.5~10.0	Masado Soil	17	18	30.4	0.68	Liquefy	11.85
10.0~13.5	Masado Soil	17	18	30.4	0.59	Liquefy	4.69
13.5~14.0	Silty Sand	25	18	34.5	1.29	Non-liquefy	0.00
14.0~17.0	Silty Sand	30	18	36.0	1.57	Non-liquefy	0.00
17.0~20.0	Silty Sand	30	18	39.0	3.26	Non-liquefy	0.00
20.0~23.5	Fine Sand	40	20	39.0	1.88	Non-liquefy	0.00

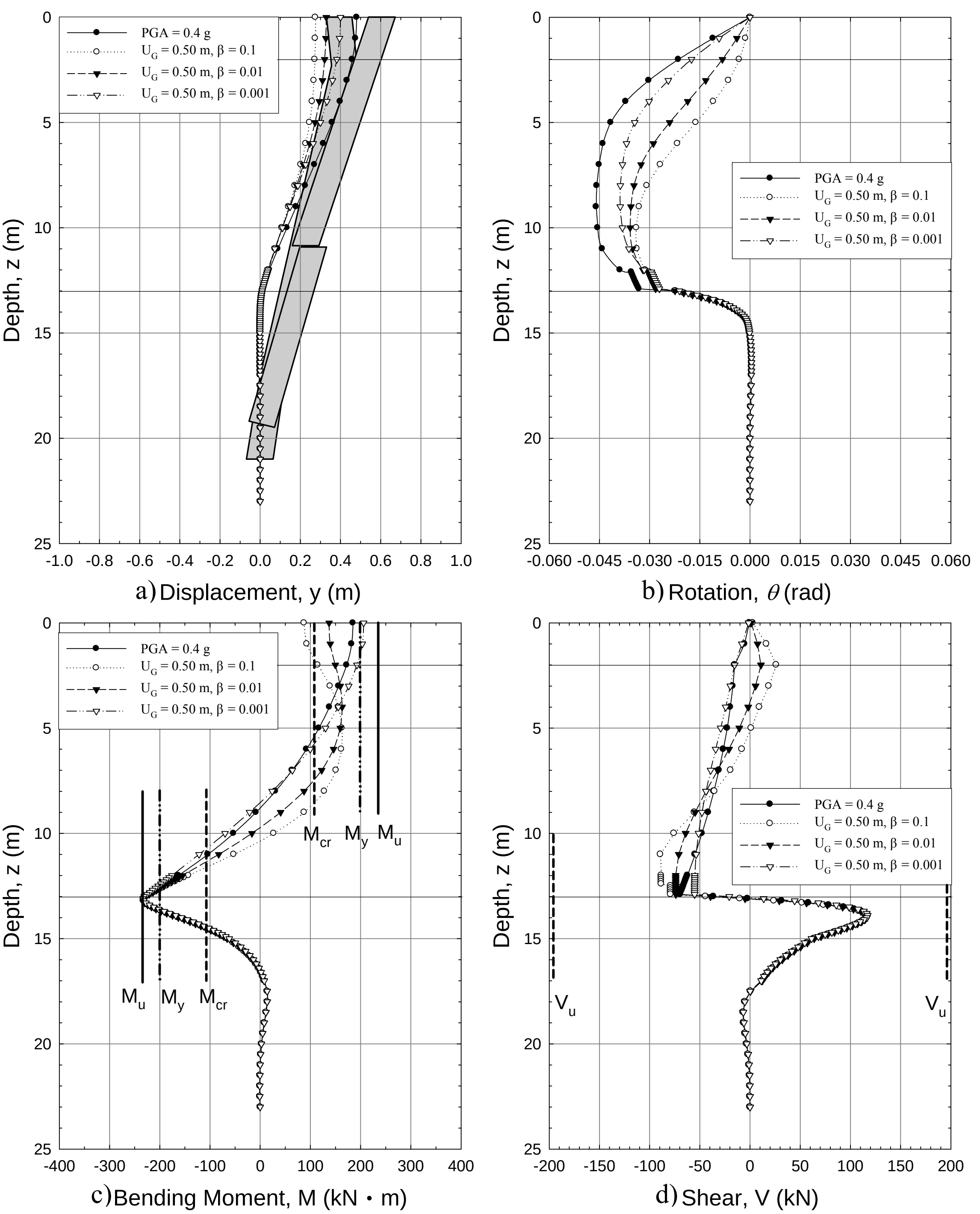


Figure 8. Analyzed deformations and internal forces along the pile

3.2 *Pushover analysis by SAP2000*

We use the software SAP2000 to perform the pushover analysis. The software provides the nonlinear static analysis function which can incrementally apply loads and displacements in multi-stages and the function of defining hinge properties assigned to frame elements to model their nonlinear behaviors. The user-defined hinge properties assigned to the pile and the soil spring are shown in Figure 9. Figure 10 shows the two above analysis models established by SAP2000. To accurately monitor the evolution of the plastic hinge, A great number of pile elements and soil springs are installed near the interface of the flow and non-flow areas, where dramatic changes of the internal forces of the pile are expected. Figure 11 shows the capacity curves of the pile in terms of the total flow force and the pile head displacement by both two models. Thus, different stressed states, including cracking, yielding, and the ultimate state can be identified on the capacity curves.
Incidentally, the development of the plastic hinges at the pile and variations in pile deformation during the different stressed stages can be clearly seen shown in Table 4. It can be seen that the pile will reach the ultimate state when pile head displacement is 0.48m in the flow pressure model. In the flow displacement model, the ultimate state displacement depends on the parameter β chosen. It was found the result of $\beta = 0.001$ most agreed with that of the flow pressure model. The distributions of the pile deformations and internal forces under the action of *PGA*=0.4g and ground displacement=0.5m with different β values are shown in Figure 8. It can be seen that the displacement and rotation of the nonlinear pile are very close to those of the field case, and the maximum bending moment and the shear force of the pile were constrained by the ultimate moment M_u. There are two plastic hinges that occur on the pile under the action of the flow pressure. The first one occurs at the interface between the liquefying layer and the non-liquefying layer. The second one occurs at the top of the pile due to the restraint of the pile cap. At these two locations, the rotation angles of the pile change abruptly due to the formations of the plastic hinge angles as shown in Table 4. It also can be found in Figure 8 that the result of $\beta = 0.001$ was more close to that of flow pressure model and both simulated the pile damage pattern well.

3.3 *Correlation of PGA with pile damage state*

The proposed flow pressure assumption and the nonlinear pile analysis model can tentatively provide a practical and valuable methodology, which can be applied to improve seismic performance-based design of the pile when subjected to different liquefaction-induced flow pressures. By the above analysis, we can correlate peak ground accelerations with different damage states of the pile. As we all know, the larger the peak ground acceleration, the more severe the ground liquefaction, and the larger the induced flow pressure. Based on the JRA method, the variations of the total flow force and the liquefaction potential index with the *PGA* for the case site is show in Figure 12. The total flow force increases from zero to a maximum value about 150 kN while the *PGA* varies from 0.295 g to 0.448 g. Since the original pile size is too small to resist the actual flow force caused by o.4 g, the bending failure occurred at the pile section near the interface of the flow and non-flow areas. To completely show the whole pushover process to the ultimate state, another B-type PHC pile with a larger diameter of 0.6m is used in the analysis. The pushover analysis is re-carried out and Figure 13 shows the capacity curve of the new pile. The cracking yielding and ultimate states of the pile are marked on the curve with open circles and the *PGAs* inducing different flow forces are marked with solid circles. Based on this figure, the non-linear lateral responses of the pile can be clearly traced when subjected to flow pressures caused by different seismic intensities. Besides, we can understand where and when the plastic hinge will occur and capture the damage mechanism of the pile. We can also easily check if the pile behavior satisfies the seismic demand under the action of the design *PGA* or not. In this case analysis, the maximum moment of the pile is in the ranges between the cracking and

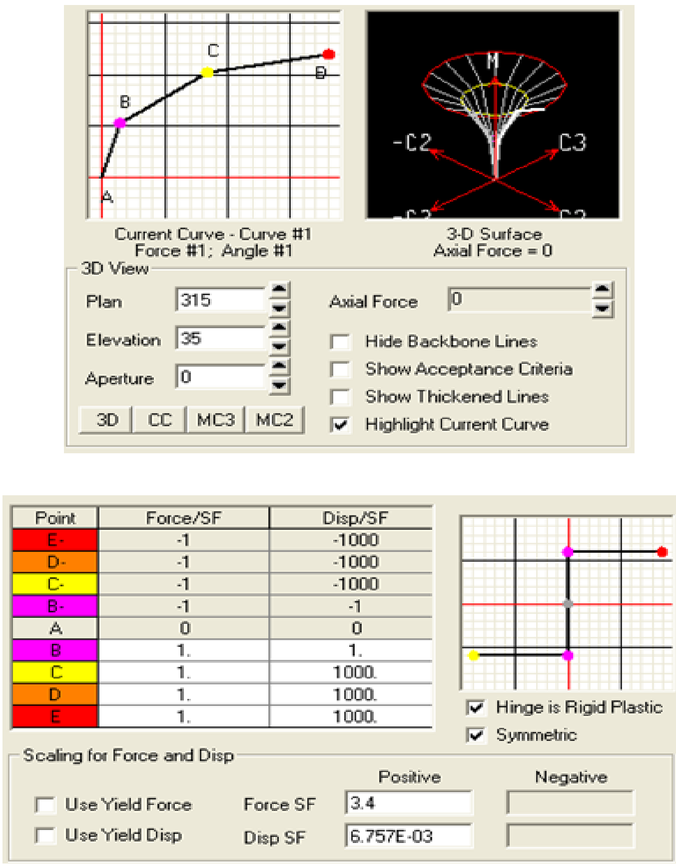
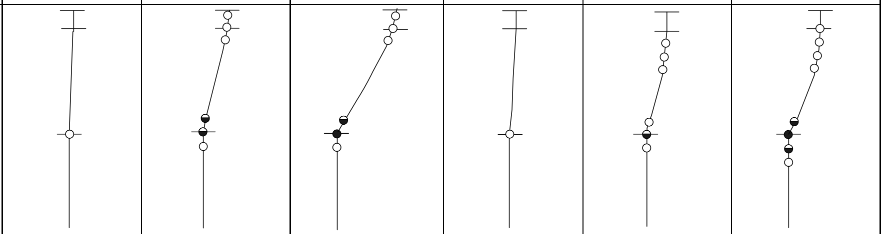
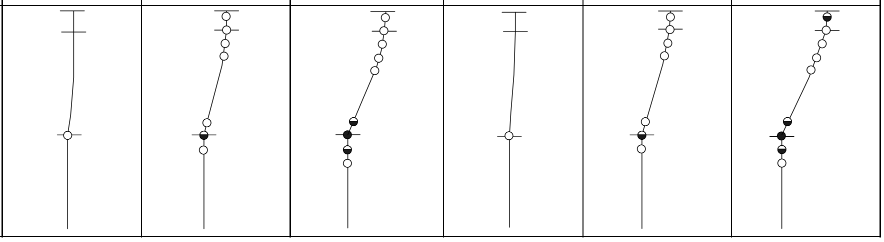
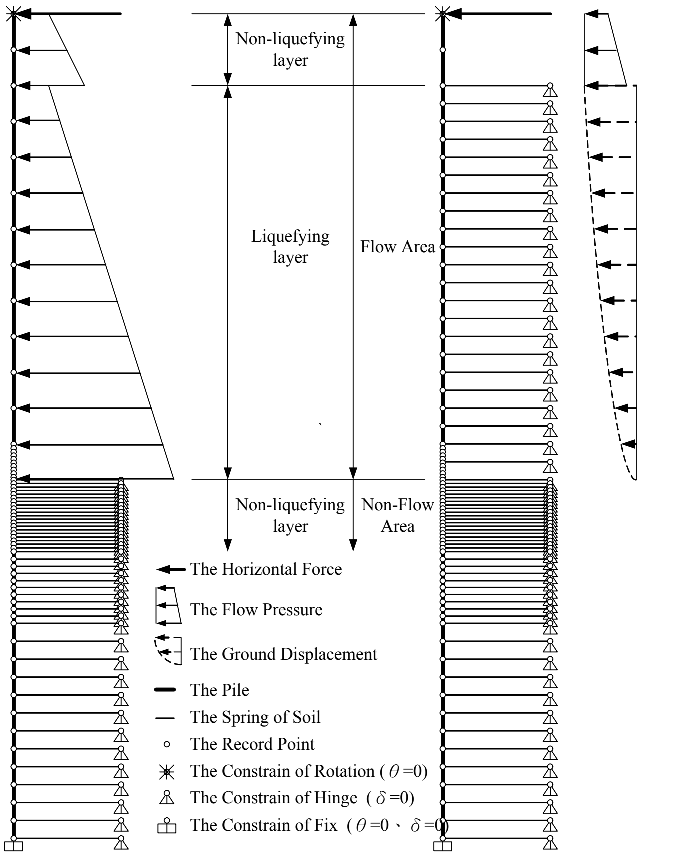


Figure 9. Hinge properties of the pile and soil spring

Table 4. Pile deformations from two analysis models

Analysis Mode	Pressure Mode			Displacement Mode		
	PGA=0.4g			B=0.1		
Status	Crack	Yielding	Ultimate	Crack	Yielding	Ultimate
Displacement of pile head (m)	0.054	0.284	0.481	0.042	0.177	0.275
Deformation ○ Crack point ◒ Yielding point ● Ultimate point						
Analysis Mode	Displacement Mode			Displacement Mode		
	B=0.01			B=0.001		
Status	Crack	Yielding	Ultimate	Crack	Yielding	Ultimate
Displacement of pile head (m)	0.050	0.202	0.329	0.056	0.256	0.400
Deformation ○ Crack point ◒ Yielding point ● Ultimate point						



(a) Flow Pressure Model (b) Flow Displacement Model

Figure 10. Pushover model of the lateral pile

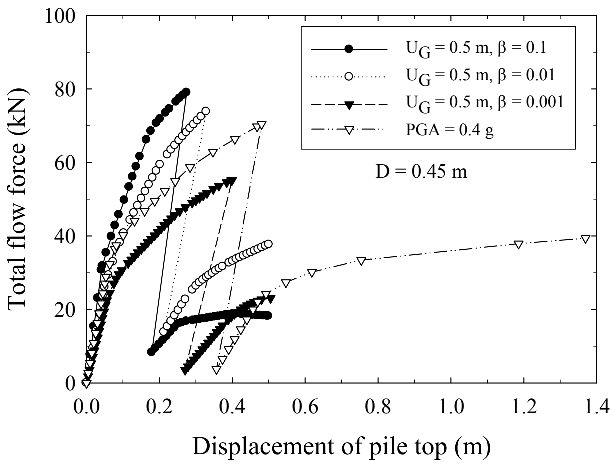


Figure 11. Capacity curve of the pile under tank TA72 using SAP2000

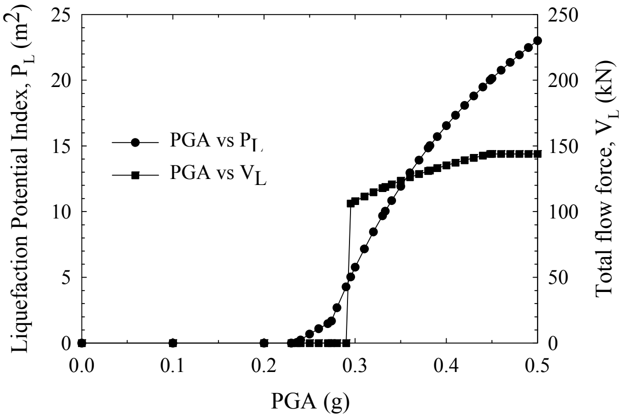


Figure 12. Variations of total flow force and PL with PGA

the yielding moment when the *PGA* varies from 0.295g to 0.382g. The maximum moment exceeds the yielding moment when the *PGA*=0.4g, and reaches the ultimate moment when the *PGA*=0.448g. As compared with the original pile, the maximum moment of the new pile only exceeds the yielding moment a little and does not reach the ultimate state under the action of real *PGA*=0.4g in Kobe earthquake. The top displacement of the new pile is about 0.2m, which is significantly smaller than that of the original pile. This shows that the seismic capacity of the new pile is better than that of the original pile.

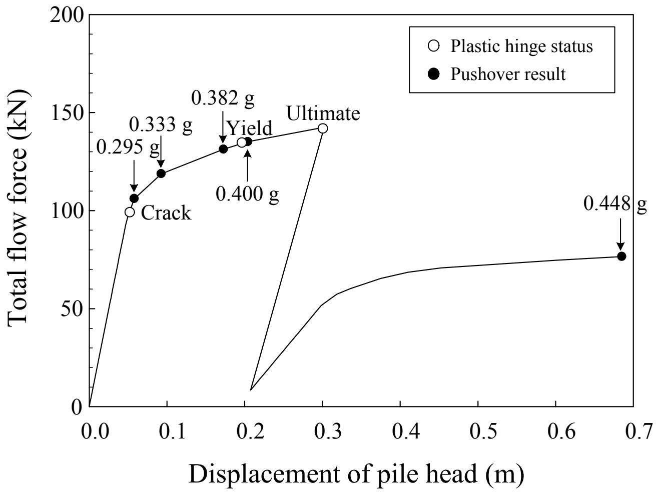


Figure 13. Capacity curve of the new pile (D=0.60m)

4 CONCLUSION

This paper presents a simple pushover analysis for the piles subjected to liquefaction induced flow pressure and ground displacement. The analysis is a non-linear static one. It models pile as a tri-linear beam-column member and soil-pile interaction as an elastic-rigid plastic spring. The analysis converts a grouped pile problem to a single pile one, and thus greatly simplifies the problem. The analysis can produce the seismic capacity curve of the lateral pile and provide a close link to the design peak ground acceleration. Based on the curve, engineers can easily perform the seismic performance design in the relevant problems.

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