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AND LATERALLY SPRADING GROUND USİNG THE MİHAMA BRİDGE, JAPAN CASE HİSTORY

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ABSTRACT

Liquefaction-induced ground failures have caused significant damage to bridges supported by deep foundations. Recent guidelines have provided an analytical framework for designing pile foundations located in laterally spreading ground. Field verification and benchmarking are required to confirm the applicability of the guidelines. Earthquake case history data related to deep bridge foundations subjected to liquefaction and lateral spreading are collected from 2011 Great East Japan earthquake. The data are analyzed to calibrate appropriate soil models and slope stability methods for evaluating pile performances using the guidelines and a beam on nonlinear Winkler foundation method. The result shows that the modulus of p-y curves significantly affects on evaluating ground displacements. For pile foundation design against liquefaction induced lateral spreading, using multiple soil models and several ground displacement are considered.

INTRODUCTION

Recent large earthquakes have provided researchers with data to benchmark earthquake design techniques. For instance, the design of pile foundations subjected to potentially liquefiable and laterally spreading soil. Bridges are particularly vulnerable to liquefaction-induced lateral spreading following an earthquake. The 2011 Great East Japan Earthquake, which had a moment magnitude, $M_W = 9.0$, caused widespread damage to coastal bridges via liquefaction-induced lateral spreading. Especially, Tokyo bay area experienced significant liquefaction, which caused large ground settlements and lateral spreading.

Yokoyama et al. (1997) analysed the bridge pile foundation performances suffered from lateral spreading using Hanshin –Awaji earthquake case history records. Effects of soil properties evaluated by SPT N-value and ground motion irregularity were considered in the analysis. The passive pressure of liquefied soil layer acting on pile foundations was estimated approximately 30% of the one of non-liquefied soil layers. Brandenberg et al. (2013) performed the displacement based lateral spreading analysis on bridge pile foundations that experienced various ground displacements using 3D finite element method and a beam on non-linear Winkler foundation (BNWF) method. Precise pile performances was predicted when available soil displacement data were applied. Above case history analysis for pile foundations are conducted for level grounds. However, abutment foundations typically locates in slopes, then the grater pinning effect is expected against ground displacement. Therefore,

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evaluating pile performances against lateral spreading in slopes should be performed to confirm the applicability of design methods.

Earthquake case history data of pile foundations from the 2011 Great East Japan earthquake, which include soil properties based on in-situ test, bridge damages, and ground damages, are collected to estimate recent developed analytical procedures by Ashford et al. (2011) and Shantz (2013). In this paper, the Mihama Bridge located in Chiba city selected from the collected data is described in followings, and analyzed. Then, computed results and observations are compared.

CASE HISTORY DESCRIPTION

The Mihama Bridge is located in Chiba City, Japan. Chiba City is approximately 368 km (epicentral distance) from the epicentre of the 2011 Great East Japan Earthquake. Chiba City experienced significant liquefaction and associated ground failures during the 2011 Great East Japan Earthquake (Ashford et al. 2011). A seismometer at Chiba City (CHB009: 35.61N 140.10E), which is operated by KiK-net, measured a peak ground acceleration (PGA) of 0.187 g during the 2011 Great East Japan Earthquake.

The Mihama Bridge crosses the Hanami River in Chiba City, and was constructed in 1985. This bridge has three spans that is composed of 54m, 67.5m, and 54m long respectively. The bridge deck width is 39 m. Two bridge piers support the bridge spans in the river, and two box-culvert type abutments support the bridge deck on the east and west sides of the river. The abutments are supported by steel pipe piles arranged in a 3 by 14 pile group. The steel pipe piles have three portions, and each portion has a diameter of 1,016 mm and a length of 11 m (i.e., the total length of the steel pipe piles are 33 m). The top portion (i.e., portion closest to the ground surface) has a thickness of 14 mm; the bottom two portions have thicknesses of 12 mm. The steel pipe piles are not filled with concrete.

Liquefaction-induced lateral ground displacements and settlements were observed in the vicinity of the east abutment following the 2011 Great East Japan Earthquake. Figure 1 shows some of the observed damage to the Mihama Bridge site. After the earthquake, a 7 cm gap was measured between the retaining wall and the east abutment, which indicates that the east abutment displaced laterally during the earthquake. Numerous noticeable ground cracks around the east abutment, ranging in size from 10 to 180 mm, were measured: The 10mm and 50mm cracks were measured at the east approach road, and approximately 100mm, 110mm, 125mm, and 180 mm cracks in front of the east abutment were also measured (Chiba City 2011). Ground settlements were measured approximately 630 mm at the face of the east abutment, and 260 mm at the side of the east abutment. Although ground settlements were measured in the vicinity of the east abutment, the east abutment itself did not settle (PWRI 2011).

Chiba City conducted a standard penetration testing (SPT) around the Mihama Bridge east abutment on April 23-26, 2011 (n.b., approximately 1.5 months after the earthquake). Figure 2 shows two SPT tests – one conducted on the backfill soils and one conducted on the soils located river side of the east abutment. The backfill soil is largely a loose sand, with SPT N-values ranging from 4 to 48 blows/ft (bpf), with a mean N-value of 11 bpf. A loose sand layer also exists on the river side of the abutment from +4.0 m to -6.0 m elevation, as seen in Figure 2, and it has N-values under 10 bpf. The groundwater table at the site is located at an elevation of +2.0 m; accordingly, the loose sand layer likely liquefied during strong ground shaking (Chiba City 2011). Various other soil strata are located below the top layer of loose sand, as shown in Figure 2, but their likelihood for liquefying is less because 1)

Figure 1. Observed cracks, settlement, and lateral spreading after 2011 Great East Japan Earthquake at the Mihama Bridge in Chiba City, Japan (Chiba City 2011)

they are too dense, 2) their plasticity indices are too high, and/or 3) they are too deep. Further details about the Mihama Bridge and earthquake-induced ground and bridge damage are given in Ashford et al. (2014).

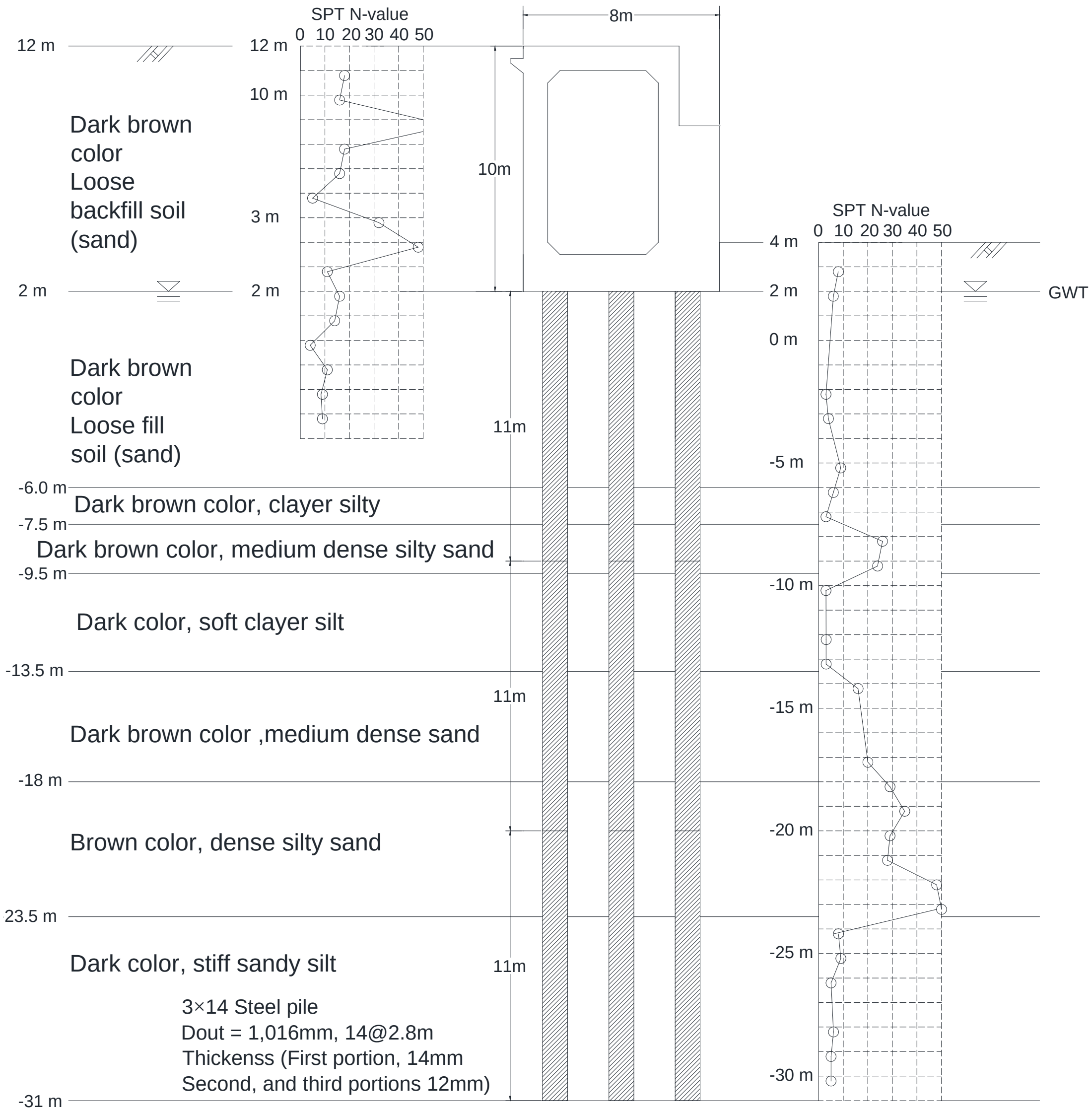

Figure 2. Post-earthquake SPT results from the east abutment of the Mihama Bridge, SPT is referred from Chiba city (2011), and the east abutment configurations is referred from Chiba city bridge register (1983).

ANALYSIS PROCEDURES

Ashford et al. (2011) developed the general procedures for designing pile foundations in laterally spreading soils, which describes pile designing procedures with considering ground lateral spreading, inertial loading from a column bent, and soil-pile interaction for both liquefied and non-liquefied layers. The procedures use a beam non-linear Winkler foundation (BNWF) method with considering the displacement of level ground and slope. Shantz (2013) provided the analytical procedures of soil-pile interaction and the evaluation of the liquefaction induced ground displacement following the Ashford et al. report. To support the second half of the paper, in this section, we briefly describe the analysis procedure described in Ashford et al. (2011) and Shantz (2013); readers are encouraged to consult the original references for a description of the full analysis procedures.

Pile group modelling

Pile group and the abutment sections are modelled as an equivalent non-linear single pile recommended by Mokwa (1999). The bending stiffness for an equivalent single pile is estimated: (1) multiplying the bending stiffness of a single pile in the pile group by the total number of piles, and (2), for abutment sections, multiplying the bending stiffness of a single pile by the number of piles and a factor of 100. For analysis purposes, the diameter of an equivalent non-linear single pile is the same as the diameter of a single pile within the pile group.

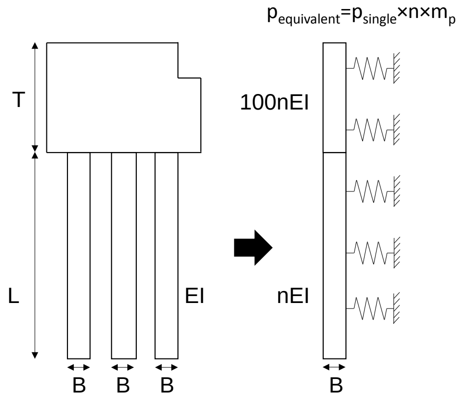

Figure 3. Pile group modeling

Subgrade reaction for equivalent single pile

The subgrade reaction for an equivalent non-linear single pile, $p_{equivalent}$, can be calculated by multiplying number of piles and modification factor m_p (equation (1)). This expression is derived from the pile group modelling.

$$p_{equivalent} = n \cdot m_p \cdot p_{single}, \tag{1}$$

where n is the number of piles, m_p is the pile group modification factor, and p_{single} is the subgrade reaction of a single pile. For non-liquefied soils, the m_p is estimated using the diagram by Mokwa and Duncan (2000). For liquefied soils, Shantz (2013) recommended the following equation (2) to estimate m_p.

$$m_p = 0.0031(N_1)_{60} + 0.00034[(N_1)_{60}]^2, \tag{2}$$

Developing p-y curves for abutment sections

Lateral load from crust layers acting on pile caps requires a critical consideration for estimating pile deflections because crust layers are softened due to cyclic loading and degradation during lateral spreading events (Brandenberg et al. 2005, 2007). These effects work on the abutment wall, too. To evaluate passive load of crust layer, two possible cases are considered (in Figure 4); Case A uses log-spiral theory, and Case B uses Rankine theory. For Case B, the crust layer is modelled as a composite block. The ultimate force, F_{ult}, acting on the abutment due to the laterally spreading soil is expressed as

$$F_{ult} = F_{passive} + F_{side} + F_{pile}, \tag{3}$$

where $F_{passive}$ is the total passive force, F_{side} is the frictional force on the side of the abutment or composite block, and F_{pile} is the lateral force acting on the piles above liquefied layers. For Case B, F_{pile} equals to zero. Frictional forces along the bottom abutment-soil interface are relatively small compared to the other forces, therefore, they are usually neglected during analysis. $F_{passive}$, F_{side}, and F_{pile} are expressed using the following equations.

$$F_{passive} = (\sigma'_V \cdot K_p + 2c' \cdot \sqrt{K_p}) \cdot T \cdot W_T \cdot k_w, \tag{4a}$$

$$F_{pile} = n \cdot m_p \cdot P_{ult} \cdot l_c, \tag{4b}$$

$$F_{side} = 2 \cdot (\sigma'_V \cdot \tan(\delta) + \alpha c') \cdot W_L \cdot T, \quad (4c)$$

where K_p is the coefficient of passive pressure, c' is the soil's cohesion intercept, T is the abutment height, W_T is the abutment width, k_w is an adjustment factor developed by Ovesen (1964), l_c is the distance from the bottom of the abutment to the liquefiable layer, δ is soil-abutment interface friction angle, and α is an adhesion factor. 0.5 is recommended by Shantz (2013). The ultimate lateral force on the pile, P_{ult}, is calculated from formulations developed by API (1993). Then, the ultimate subgrade reaction is estimating using equation (5).

$$p_{ult-equivalent} = F_{ult}/T. \quad (5)$$

Brandenberg et al. (2007) suggested a formula for estimating the maximum displacement of pile caps at an ultimate subgrade reaction as a function of a wall height,

$$\Delta_{max} = T \cdot (0.05 + 0.45 \cdot f_{depth} \cdot f_{width}), \quad (6)$$

where f_{depth} and f_{width} are modification factors given by Brandenberg et al. (2007). Equation (6) assumes that the pile cap has rigid walls, and is also used for estimating maximum displacements of the abutment.

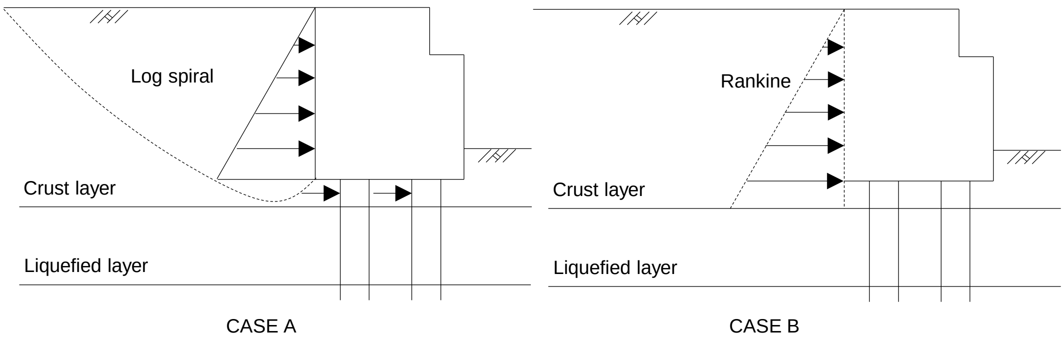

Figure 4**.** Passive pressure of crust layers: case A Log spiral theory, case B Rankin theory

Estimating liquefaction-induced lateral ground displacement considering pile pinning effects

Pile foundations restricts a movement of gently sloping ground if the lateral stiffness of the pile foundations is larger than driving forces from liquefied soil layers. This phenomenon is called as pile pinning effect. The following procedure is used to estimate liquefaction-induced lateral ground displacements of gently sloping ground with considering pile pinning effects (Ashford et al. 2011; Shantz 2013):

1) Estimate the residual shear strength of the liquefied soil layer.
2) Perform a slope stability analysis using the residual shear strength estimated in step 1 with increasing driving force R at the center of liquefied layers and horizontal acceleration k_y until the factor of safety equals to one.
3) As estimated driving force R, it is multiplied by the effective abutment width to account for embankment slope. The effective abutment width is calculated using equation (8).

$$W_{effective} = W_t + H \cdot m/2. \quad (8)$$

where, $W_{effective}$ is an effective abutment width, W_t is an abutment width, H is an abutment height, and m is an inclination of an embankment.

4) Calculate a slope displacement using the equation (10) developed by Bray and Travasarou (2007).

$$\text{In}(D) = -0.22 - 2.83\ln(k_y) - 0.333(\ln(k_y))^2 + 0.566\ln(k_y)\ln(PGA) + 3.04\ln(PGA) - 0.244(\ln(PGA))^2 + 0.278(M_w - 7) \quad (10)$$

Where D is slope displacement in centimeter. PGA is peak ground acceleration, M_w is moment magnitude.

5) Implement a pushover analysis for an equivalent non-linear single pile with increasing ground displacements. The inertial shear force of the piles in the liquefied layer is averaged to develop the shear force-displacement curve.
6) Plot the driving force R and the inertial shear force with ground displacements. The cross section point is the compatible ground displacement with considering the pinning effects.

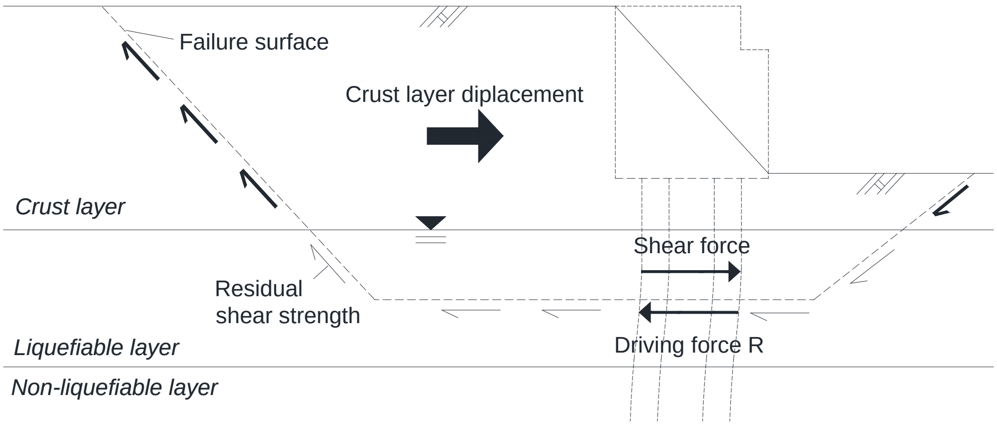

Figure 5. Concept of slope displacement estimation with considering pinning effect

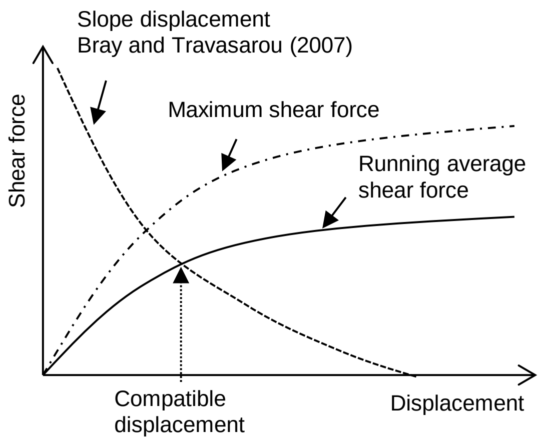

Figure 6 Determination of compatible ground displacement

ANALYIS OF THE PILE FOUNDATION OF THE MIHAMA BRIDGE ABUTMENT

Liquefaction potential evaluation using Idriss and Boulanger (2008) is performed to identify the liquefiable layer. Also, analysis of the Mihama Bridge abutment pile foundation is performed using LPile 2012 for pushover analysis and Geo Studio 2012 for slope stability analysis with several p-y curves and slope stability analysis model to calibrate possible ground displacements.

Liquefaction potential evaluation

Liquefaction potential evaluation is performed based on SPT N-value measured at the river side of the east abutment. Chiba prefecture (2013) reported that fine content at Mihama area for silty sand layers is from 6% to 32%. In this evaluation, clayer silt layers are identified as non-liquefiable layers because their plasticity indices are too high. Although factor of safety of the medium dense sand layer is less than one, liquefaction is not expected at this layer due to high vertical effective stress. Figure 6 shows factor of safety at Mihama Bridge site. Judging from this evaluation, the backfill soil from +2.0m to -6.0m is identified as a liquefiable layer.

Pile group modelling

Total 42 piles (3×12 pile group) is modelled as an equivalent non-linear single pile. The pile group is composed of 14mm and 12mm thickness pile portions. In this analysis, 14mm thickness is selected for pile modelling because 14mm thickness portion is embedded in the liquefiable layer, and this portion may resist ground displacements. Bending moment-bending curvature diagram of the equivalent single pile is shown in Figure 7.

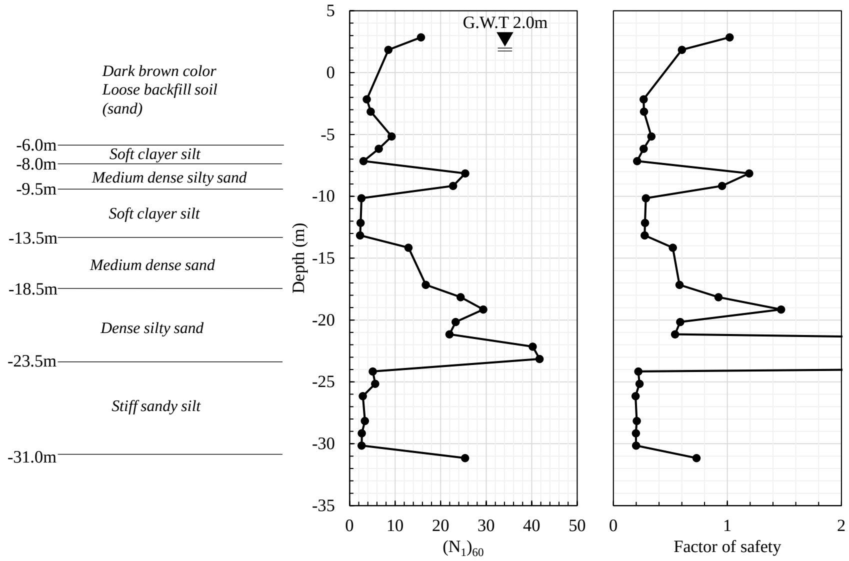

Figure 7. Liquefaction potential evaluation at the Mihama Bridge

P-y curve for the abutment section

Soil-pile interaction for the abutment section is modelled as a p-y response described in previous section. The smaller passive pressure estimated by Rankine theory (Case B) is selected because of considering the softening effect. The ultimate passive load per height p_{ult} 14,600 kN/m and Δ_{max} 1.76m are calculated, respectively. Figure 8 shows the p-y curve of the crust layer.

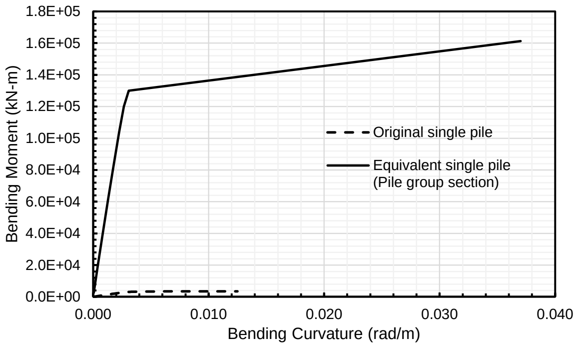

Figure 8 Bending moment – bending curvature diagram

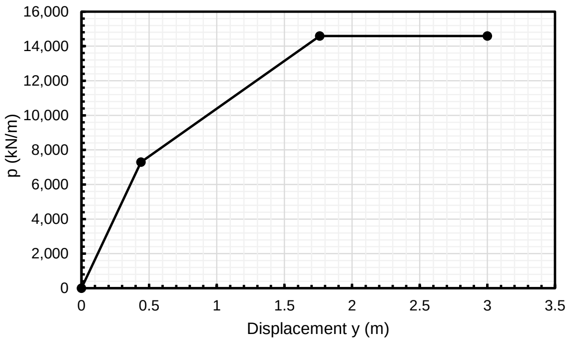

Figure 9 P- y curve for the abutment section

Pushover analysis

Pushover analysis is performed to develop the averaged shear force - ground displacement diagram with several p-y responses equipped in LPile 2012. The input parameters are shown in Table 2 and Table 3. Effective unit weight is estimated using Kulhawy and Mayne (1990). The p-multiplier m_p is estimated using Mokwa et al. (2000) for non-liquefiable layers. The internal friction angle is estimated using Hatanaka and Uchida (1996). Stain factor ε_{50} is determined by referring Reese and Van Impe (2011). The modulus of subgrade reaction is determined by referring Brandenberg et al. (2013). The residual shear strength of the liquefiable layer is estimated using the formula developed by Kremer (2008). The Matlock soft clay model is utilized for the residual shear strength model with strain factor ε_{50} equals to 0.05. The results are shown in Figure 9.

Slope stability analysis

Horizontal acceleration, k_y, is estimated using slope stability analysis with several slope stability theory when the factor of safety equals to one with increasing the driving force R. The slip surface does not extend a distance beyond 4H (H is the abutment height). The bottom of the slip surface is assumed locating at the center of liquefied layer because the loss of shear strength is expected in the liquefied layer due to generating excess pore water pressure during earthquake.

Moment magnitude 9.0 and peak ground acceleration 0.189g are applied. The results are also shown in Figure 9.

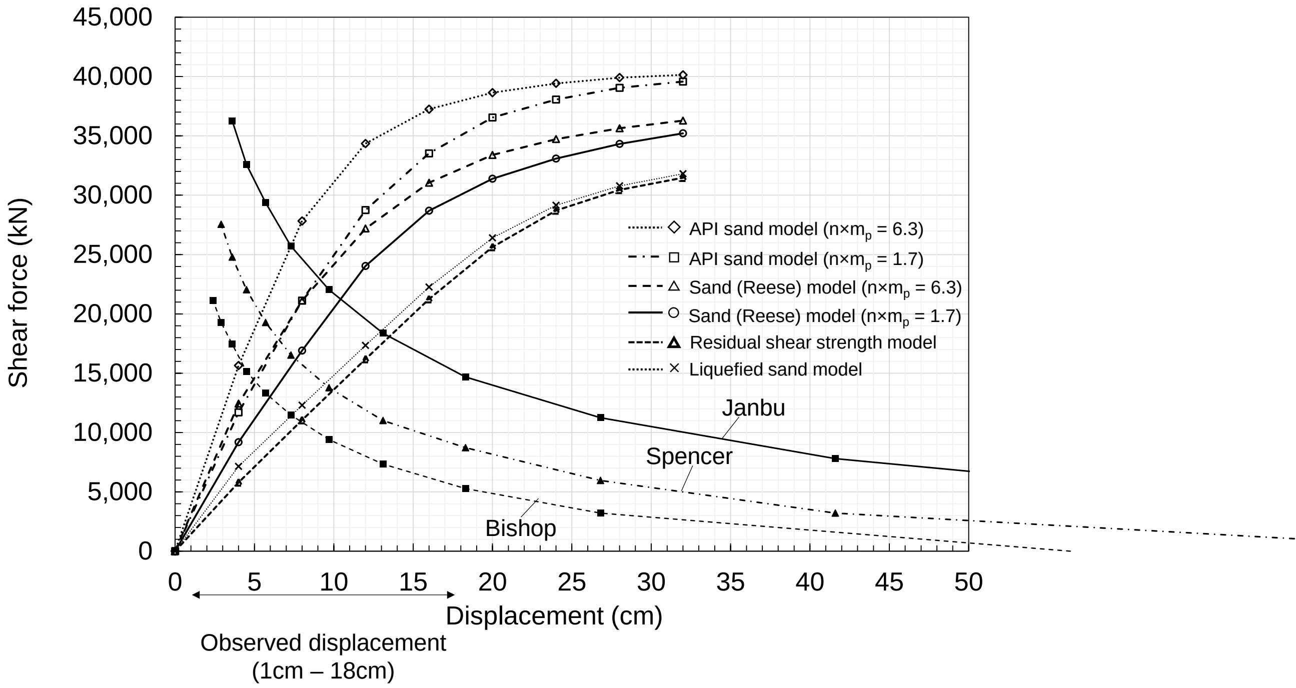

Figure 10. Comparison of the estimated displacement using several soil models and slope stability *theory* for the equivalent non-linear single pile

Ground displacement and pile foundation performance of Mihama Bridge abutment

Table 1 shows the comparison of the ground displacement, pile head displacement, and bending moment of the Mihama bridge abutment foundation. The estimated ground displacement is 4cm-13cm, which is within the observed displacement 1cm-18cm. The estimated pile head deflection is 3cm-7cm, and these values are close to the abutment displacement 1cm-5cm (the backfill displacement due to liquefaction corresponds to the abutment displacement because the abutment wall restricts the backfill movements).

Figure 10 shows the analysis example of the pile deflection, bending moment, shear force, and subgrade reactions with 6cm, 10cm, and 18cm ground displacement using Sand (Reese) model. In this analysis, the fixed pile head condition is applied to achieve no rotation of the abutment section although bending moment at the top of the abutment must be zero. The bending moment is approximately 57%-92% of the yield bending moment, then no damages on pile group are expected with estimated ground displacement, which much up with the observed performances. The recommended guidelines and procedures are predicted ground displacement, and abutment displacement well.

Table 1 Comparison the ground, pile head (abutment) displacement and bending moment of the Mihama bridge abutment

Displacement				Bending moment		
Estimated		Observed				
D_{ground} (cm)	D_{Pile} (cm)	D_{ground} (cm)	$D_{Abutment}$ (cm)	Estimated (kN-m)	Yield (kN-m)	Allowable (kN-m)
4-13	3-7	1 -18	1-5	75,000-119,000	130,000	161,000

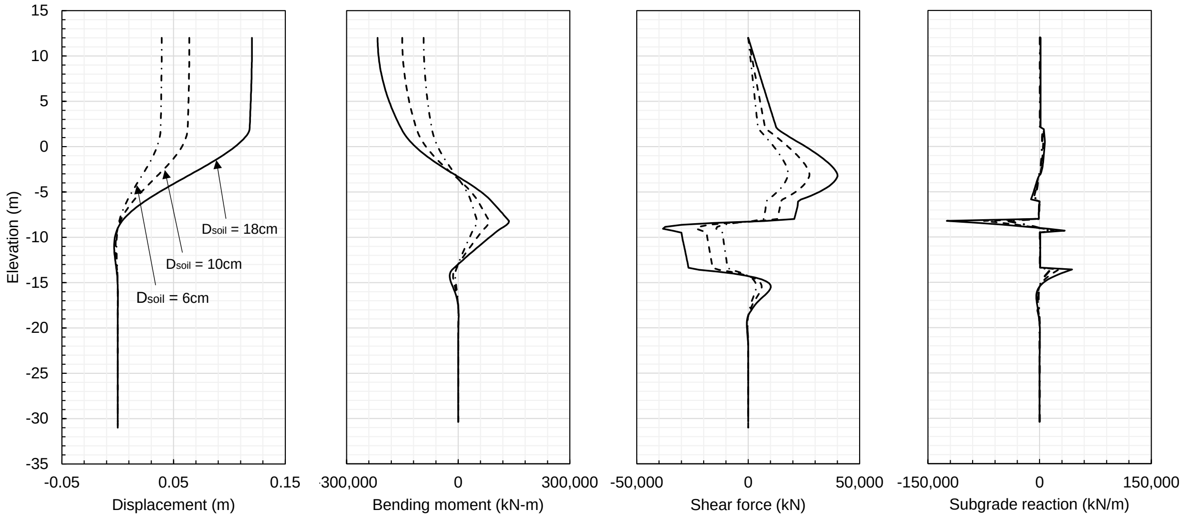

Figure 11 Analysis of Mihama Bridge abutment foundation, 6cm, 10cm, and 18cm soil displacement applied.

Table 2 Input parameters for the analysis of the Mihama Bridge abutment

Parameter	Elevation (m) +12 ~ +2	+2 ~ -6.0	-6.0 ~ -7.5	-7.5 ~ -9.5	-9.5 ~ -13.5	-13.5 ~ -18	-18 ~ -23.5	-23.5 ~ -31
Soil model	User input p-y curve	Several models used	Soft Clay (Matlock)	Sand (Reese)	Soft Clay (Matlock)	Sand (Reese)	Sand (Reese)	Stiff Clay with Free water (Reese)
Effective unit weight (kN/m^3)	17	17	17	18	17	17	20	18
ϕ' (°)	-	-	-	42	-	39	47	-
Strain Factor ε_{50}	-	-	0.02	-	0.02	-	-	0.007
c' (kPa)	-	-	23	-	23	-	-	50
n[1)]$\times m_p$	1	-	10.0, 18.9	28.1	28.1	28.1	28.1	28.1
Modulus of subgrade reaction (kN/m^3)	-	-	-	80,000	-	76,000	160,000	135,000

1) n is number of piles

Table 3 Input parameters for liquefiable soil layer (depth from +2m to -6.0m)

	Soil models: Sand (Reese)	API sand (O'Neill)	Liquefied soil (Rollins)	Residual shear strength model
Effective unit weight (kN/m^3)	17	17	17	17
ϕ' (°)	32	32	-	-
Strain Factor ε_{50}	-	-	-	0.05
c' (kPa)	-	-	-	14
$n \times m_p$	1.7, 6.3	1.7, 6.3	-	-
Modulus of subgrade reaction (kN/m^3)	35,000	35,000	-	-

DISCUSSION

Figure 9 clearly shows that p-y response for the liquefied layer is critical to the shear force of pile in liquefied layer. For example, using Bishop theory, the estimated displacement with residual shear strength model is twice as the API sand (O'Neill) model, and difference is approximately 4cm. Compatible displacement exhibits when the shear force in pile equals to the driving force. Then, the comparisons indicate that the required ground displacement is affected by the modelling of p-y curve for liquefied layer. Low ultimate subgrade reaction p-y curves require much ground displacement.

The maximum shear force exhibits at the 20cm~25cm ground displacement. The field investigations show that the lateral displacements were from 1cm to 18cm at the vicinity of the Mihama Bridge. If the liquefiable layer displaced 18cm, the pile foundation was possibly yielded. However, the no settlement of the abutment was observed, and the bridge was still in service after the earthquake, which indicate the pile foundations were not damaged due to liquefaction and lateral spreading. This comparison indicates that the ground displacement that the pile foundation experienced is smaller than the observed maximum ground displacement. Slope stability theories are also critical for the estimation of the ground displacement. The shear force exhibiting at the same displacement using Janbu is 2 or 3 times of the one of Bishop theory. The results show that the estimated horizontal acceleration is significantly affected by type of stability analysis theory.

The analysis and observations show that the wide range lateral displacement is expected, and it is difficult to determine ground displacements deterministically. For pile foundation design, several expected ground displacement should be considered.

CONCLUSIONS

The Mihama Bridge abutment foundation subjected to liquefaction and lateral spreading in 2011 Great East Japan earthquake are analysed using the guidelines and the procedures developed by Ashford et al. (2011) and Shantz (2013). The lateral ground displacements from 1cm to 18cm were observed at the vicinity of the Mihama Bridge abutment. The analysis estimated the displacements from 4cm to 13cm using several p-y responses and slope stability analysis theories with considering pinning effect.

1. The recommended guideline and procedures are predicted the ground displacement, abutment displacement well.
2. The analysis and observations show that lateral displacements due to liquefaction are exhibited in wide range at the vicinity of the abutment pile foundation. Several p-y responses and slope stability analysis theories should be considered to estimate possible range of soil displacements.
3. BNWF method can predict reasonable pile performances when several ground displacements are considered.

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