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Dynamic analysis of pile-soil-structure interaction behavior of transmission towers

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ABSTRACT: The 2011 off the Pacific coast of Tohoku Earthquake was a massive earthquake of magnitude of 9.0 with long duration of approximately 5 minutes. During the earthquake, an electrical accident of short-circuit occurred due to contact of transmission lines by excitation at a site where more than 300km away from the epicenter. Since the observed peak ground acceleration near the site was as small as about 50 gal, the cause of occurrence was not easily understood. In this study, we conducted the dynamic pile-soil-structure interaction analysis of two transmission towers to clarify effect of ground condition including soil liquefaction on response of cables. It was found that transmission lines could show large seismic response and possibly cause the contact between cables at a location where ground conditions between transmission tower foundations were clearly different in terms of liquefaction susceptibility, Vs30 values and depth of bedrock.

1 INTRODUCTION

The 2011 off the Pacific coast of Tohoku Earthquake on 11th March (hereafter, '3.11 earthquake') was a massive earthquake of magnitude of 9.0 with long duration of approximately 5 minutes. During the earthquake, an electrical accident of short-circuit occurred due to contact of transmission cables by seismic excitation. The site is located in Kofu basin in Yamanashi prefecture, Japan, which is located more than 300 km away from the epicenter. Since the observed peak ground acceleration near the site was as small as about 50 gal, the cause of large seismic response of transmission cables was not easily understood.

Some valuable knowledge has been obtained from previous studies about seismic response of transmission tower and cables. Sato and Izuno (2013) carried out numerical analysis of transmission tower-line system of four spans to examine the effect of duration time. They concluded that long period component of ground motion caused the large displacement of cables. One of authors in this paper made a series of numerical studies on the site during 3.11 earthquake (Ohta et al. 2014), and found that phase difference of displacement as much as 0.7 seconds between tower foundations could develop large response of cables. Liu et al. (2016) studied on seismic responses of transmission tower in consideration of pile-soil-structure system. They found that effect of the system on dynamic interaction was remarkable especially in soft soil site.

Most previous studies have been focusing on tower-line system, whereas few studies have been conducted considering pile-soil-structure system with nonlinearity of soils including liquefaction. In this paper, we conducted the dynamic pile-soil-structure interaction analysis of two transmission towers considering pile foundations and soil liquefaction in order to clarify the possible reason of large response of cables during 3.11 earthquake. To account for geometric nonlinearity of cables, we used the finite strain dynamic effective stress analysis program of "FLIP TULIP" (Total and Updated lagrangian LIquefaction Program, Iai, Ueda, 2013). Parametric study was conducted as well in which some analysis conditions were varied.

2 MODEL OF PILE-SOIL-STRUCTURE INTERACTION ANALYSIS

2.1 *Pile-soil-structure model*

The site is located at southern part of Kofu basin in Yamanashi Pref., Japan. Figure 1(a) illustrates geological framework model in the cross section along transmission line between towers. This model was created using geological data near the transmission towers (Tamari, et al., 2018b). Depth of bedrock is estimated about 120m at tower No.2, 80m at tower No.3. Figure 1(b) depicts finite element mesh for the pile-soil-structure interaction analysis. The mesh includes bedrock and subsurface soil, river banks (Height: 4m to 5m), pile foundations, towers and cables of which initial configuration is assumed to be a catenary.

2.2 *Subsurface conditions*

The model used for soil is a strain space multiple mechanism model (Iai et al., 1992). Table 1 summarizes model parameters of soils from previous study (Tamari et al., 2018b). As liquefaction susceptibility is reported at the north side of the river where is categorized as marsh land in the rear (Yamanashi Pref., 2013), cyclic stress ratio is specified for layer B of top soil below water table and layers As2 and Ag1 of alluvial sand and gravel (see, Table 1(a)). Model parameters for liquefaction characteristics are summarized in Table 2. Reproduced liquefaction resistance curves using those parameters are illustrated in Figure 2.

2.3 *Pile foundations*

Transmission towers are built on pile foundation of which specifications and dimensions are summarized in Table 3. Three dimensional (3D) configuration of the foundation is accounted by controlling the width of two dimensional (2D) elements at each part, and by using pile-soil interaction nonlinear spring for dynamic interaction in horizontal direction (Tamari, et al., 2018a). Figure 3(a) illustrates side view of the foundation No.2 as typical example, (b) depicts 2D finite element mesh with the width of each part of element. Soils are modeled by plane strain nonlinear element. Pile cap is modeled by linear elastic element, and piles by linear beam element. Pile-soil interaction nonlinear spring is illustrated in Figure 3(c) with parameters for configuration. Nonlinear properties of the spring are automatically calculated in the program step by step referring to state of nonlinear soil element adjacent to the pile element.

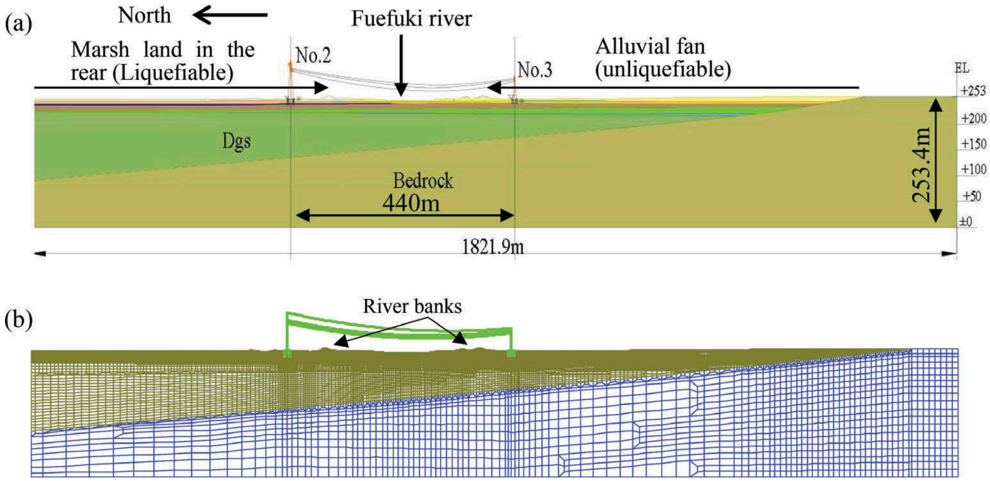


Figure 1. Pile-soil-structure model.: (a) Geological frame work model with transmission towers. (b) Finite element mesh.

Table 1. Model parameters for physical and dynamic deformation characteristics of soils.
a. Area of marsh land in the rear. (Vs30=200m/s, around tower No.2)

Layer	H (No.2) (m)	ρ (t/m^3)	V_s (m/s)	G_{ma} (kPa)	$-\sigma_{ma}'_{(2D)}$ (kPa)	φ_f (deg)	h_{max}	φ_p (deg)	Cyc.Str.Ratio DA=5.0%
B	1.40	1.6	120	23040	15.1	39.0	0.24	-	-
B	1.17	1.6	120	23040	15.1	39.0	0.24	28.0	0.188
As2	2.17	1.8	150	40500	36.6	43.9	0.24	28.0	0.239
Ag1	4.37	2.0	180	64800	59.0	43.9	0.24	28.0	0.255
Dc	2.64	1.8	180	58320	82.8	0.0*1	0.20	-	-
Dg1	9.47	2.1	200	84000	128.9	43.9	0.24	-	-
Dgs2	6.55	2.1	260	141960	193.6	43.9	0.24	-	-
Dgs	86.74	2.1	400	336000	503.2	43.9	0.24	-	-
Bedrock	135.32	2.1	700	-	-	-	-	-	-

Vp of Bedrock=3569m/s, $V_{s,ave}$=200m/s, *1 Cohesion of layer Dc: 467 kPa, Water level: GL-1.4m at tower No.2

b. Area of alluvial fan. (Vs30=300m/s, around tower No.3)

Layer	H (No.3) (m)	ρ (t/m^3)	V_s (m/s)	G_{ma} (kPa)	$-\sigma_{ma}'_{(2D)}$ (kPa)	φ_f (deg)	h_{max}	φ_p (deg)	Cyc.Str.Ratio DA=5.0%
As2	3.61	1.8	200	72000	36.6	43.9	0.24	-	-
As1	3.72	1.9	250	118750	60.2	40.0	0.24	-	-
Ag1	1.07	2.0	250	125000	59.0	43.9	0.24	-	-
Dg1	9.50	2.1	300	189000	128.9	43.9	0.24	-	-
Dgs2	10.33	2.1	350	257250	193.6	43.9	0.24	-	-
Dgs1	1.81	2.1	350	257250	193.6	43.9	0.24	-	-
Dgs	44.99	2.1	400	336000	503.2	43.9	0.24	-	-

Vp of Bedrock=3569m/s, $V_{s,ave}$=300m/s, Water level: GL-4.6m at tower No.3. *H:* layer thickness; ρ: density; V_s :shear wave velocity; G_{ma}: elastic shear modulus at a confining pressure of $(-\sigma_{ma}')$; $-\sigma_{ma}'$: reference confining pressure; φ_f :shear resistance angle; and φ_p :phase transformation angle, $\sigma_{ma}'_{(2D)}=\sigma_v'\bullet(1+K_0)/2$, K_0: earth pressure at rest (=0.5), σ_v': effective vertical pressure at the center depth.

Table 2. Model parameters for liquefaction characteristics in marsh land in the rear.

Layer	S_1	w_1	p_1	p_2	c_1
B	0.005	4.35	0.50	0.98	1.57
As2	0.005	4.88	0.50	0.93	1.73
Ag1	0.005	6.20	0.50	0.99	1.79

Parameter controlling ; p_1: initial phase, p_2: final phase, w_1: overall, of dilatancy. Parameter controlling ; S_1: ultimate limit, c_1: threshold limit, of dilatancy.

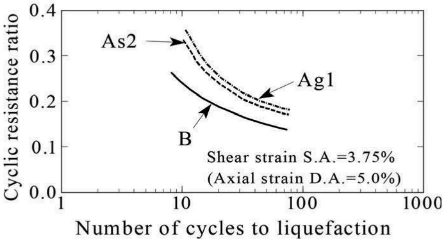
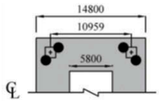
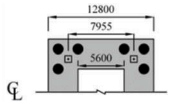


Figure 2. Reproduced liquefaction resistance curves.

Table 3. Specification of pile foundations of transmission towers.

Items	Tower No.2	Tower No.3
Dimension of pile cap	14.8m×14.8m, t=2.0m	12.8m×12.8m, t=1.6m
Pile types, numbers and pile group configuration	Cast in place pile (RC), n=8 Diameter =1200mm 	Cast in place pile (RC), n=12 Diameter =1200mm 
Pile bottom depth	GL-12.50m	GL- 9.50m

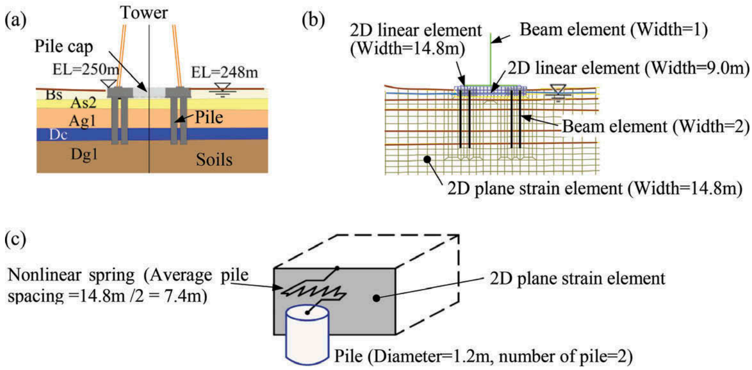


Figure 3. Model of foundation and pile-soil interaction. (typical example of tower No.2): (a) Side view of pile foundation. (b) Finite element mesh. (c) Pile-soil interaction spring.

2.4 *Transmission towers and line*

Towers are simply modeled by cantilever which has the same natural period of first mode using linear beam elements. Table 4 shows properties of transmission tower model.

Table 5 summarizes the condition of transmission line. Cables originally should have characteristics in which the bending stiffness is small enough to be negligible. In this study for simplicity, cables are modeled by beam elements with very small bending stiffness of $EI/100$ (I: moment of inertia of area) and small effective shear sectional area of $e_f/10$. Counter weights on the cable are considered as of the same condition in previous study (Ohta et al. 2014).

3 INPUT GROUND MOTION AND ANALYSIS

3.1 *Reproduction of input ground motion*

Input ground motions for the dynamic analysis are reproduced using observed accelerations at the surface of recording station K-NET Kofu (YMN005, 2011/03/11–14:46, 38.103N,

Table 4. Properties of Transmission Towers.

Items	Tower No.2	Tower No.3
Height (m)	74.0	44.0
Natural Period of the First Mode (sec)	0.59	0.29
Damping Ratio (%)	2.0	2.0

Table 5. Specification of transmission line.

Items	Ground cable	Conductor cable
Specification of line	GWS70mm^2	TRACSR610mm^2
Number of conductor	1	1
External diameter (mm)	10.5	34.2
Self-weight/meter (kg/m)	0.533	2.320
Cross-sectional area (mm^2)	67.35	691.8
Equivalent Young's modulus E (GPa)	171.5	78.3
Line tension (N)	7350	28910
Natural Period (sec)	First 7.52, Second 3.75	First 7.87, Second 3.94
Damping Ratio (%)	0.4 (1.0s to 10.0s)	0.4 (1.0s to 10.0s)

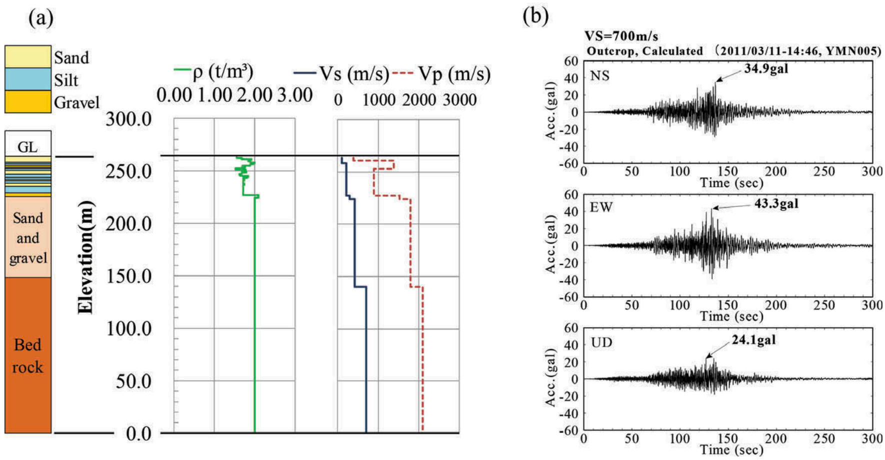


Figure 4. Reproduction of input ground motion for dynamic analysis.: (a) Soil column model at recording station used in calculation. (b) Reproduced ground motion.

142.860E, M9.0, NIED), of which location is about 7km north from the site. Figure 4(a) depicts developed soil column at the recording station. Figure 4(b) shows the reproduced input ground motion at the bottom of the model.

3.2 *Pile-soil-structure interaction analysis*

Before the dynamic analysis, two stage of static analysis was conducted by gravity in order to simulate the initial stress of soil and section force in piles, towers and cables. First, gravity was applied to ground, and second, to the foundation, transmission tower and cables. Horizontal displacement was constrained at side surface of ground model and both horizontal and vertical displacement was fixed at the base through static analysis. With these initial conditions, a dynamic analysis was conducted on the pile-soil-structure model. The NS and UD components of reproduced motion with duration time of 300 seconds were used simultaneously as the input motion. The analysis was conducted with undrained conditions in order to simplify the analysis. Total lagrangian was considered for large deformation calculation. The time integration was numerically done using the Wilson-θ method (θ=1.4) using a time step of 0.01 seconds. Reyleigh damping of α=0.0, β=0.001 was used to ensure stability of the numerical solution. Reyleigh damping of β_{stru}=0.004 in tower No.2, β_{stru}=0.002 in No.3, was used individually so that it became equivalent to damping ratio of tower (h=2.0%) at the first mode of natural period. For cables, α_{cable}= 0.0091, β_{cable}=0.00058 is used considering damping ratio of h=0.4%.

4 RESULTS AND DISCUSSIONS

4.1 *Typical result*

Figure 5 illustrates typical results of dynamic pile-soil-structure interaction analysis. It is seen in Figure 5(a) that ground cable seems to be vibrating by 2nd mode, while conductor cable by 3rd mode at the moment. Excess pore water pressure ratio builds up slightly with the maximum value of about 0.2 around the tower foundation No.2. This is consistent with no report of liquefaction around the foundation at 3.11 earthquake. Pore pressure ratios of 0.4 to 0.9 are calculated at surrounding ground near the river.

Figure 6 depicts calculated earthquake responses at tower foundations of No.2 and No.3 for (a) accelerations, (b) rotation angles, (c) horizontal displacements with respect to bottom of the model, (d) horizontal relative displacements between towers.

It is seen from figure (a) that the calculated maximum acceleration is $0.70 m/s^2$ at foundation No.2, being apparently larger than $0.57 m/s^2$ at No.3. From figure (b), the rotation angle at No.2 gradually decreases (rotates clockwise) with time while increases (rotates unti-clockwise) at No.3. This is considered due to nonlinear behavior of soil foundation system. In is indicated in figure (c) that the maximum horizontal displacement at No.2 where ground is relatively soft is 0.03m, being larger than that of at No.3 of 0.01m. It is clearly seen in figure (d) that time

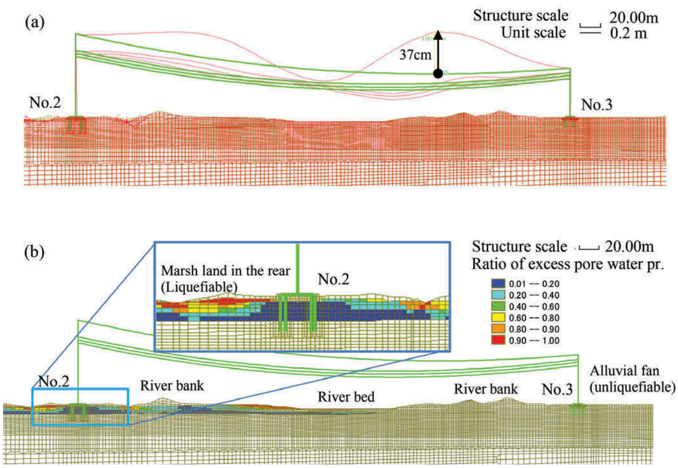


Figure 5. Typical results of dynamic pile-soil-structure (foundation-tower-line) interaction analysis.: (a) Deformed mesh at t=140.0s when the input ground motion becomes almost maximum. (b) Distribution of maximum excess pore water pressure ratio through all duration time. (t=0.0 to 300.0s)

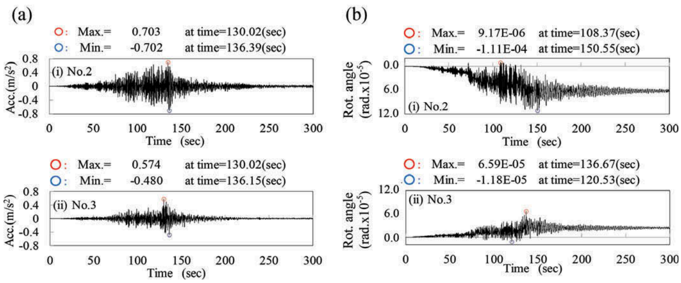


Figure 6. Calculated earthquake responses at tower foundations of No.2 and No.3.: (a) Horizontal accelerations. (b) Rotation angles.

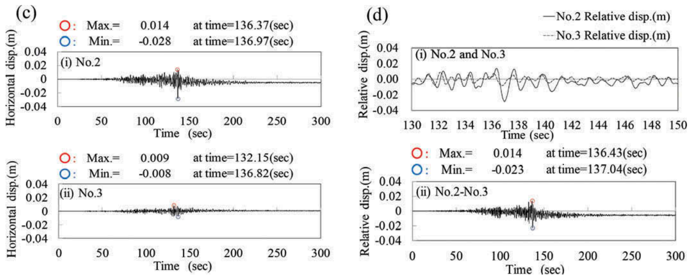


Figure 6. Calculated earthquake responses at tower foundations of No.2 and No.3.: (c) Horizontal displacements. (d) Relative displacements between towers.

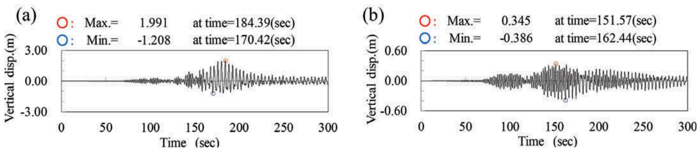


Figure 7. Calculated vertical displacements of cable at the point of maximum response.: (a) Ground cable. (b) Conductor cable.

histories of displacement at No.2 and No.3 are very different with respect to amplitude and phase, resulting in the maximum horizontal displacement of about 2cm at around 137s.

Figure 7 illustrates time histories of calculated vertical displacements for (a) ground cable (b) conductor cable at the point about one third of span from No.3 tower. Vertical displacement of cables became the maximum 2.0m at 176s for ground cable, 0.41m at 162s for conductor cable. It is noted that the cable displacement becomes maximum 30 to 40 seconds after intense motion of foundation.

The result of the simulation suggests that first, different ground condition in which foundation No.2 is soft and liquefiable while No.3 is hard, produces dynamic relative displacement as much as 2 to 3cm between foundations during 3.11 earthquake. Second, long duration in addition to the relative displacement of foundation could make vertical response of cables more significant.

4.2 *Parametric study*

Since the numerical model and input conditions is thought to contain various uncertainties, we conducted some cases of analysis in which analysis conditions were varied to examine effects on the behavior of foundations and cables. The varied conditions were, i) intensity of input motion (original, double), ii) liquefaction condition at around tower No.2 (liquefy, non-liquefy), iii) configuration of bedrock upper boundary (see Figure 8), iv) existence of nearby earth structures of river banks (see Figure 9).

We focused on maximum acceleration at foundations and vertical displacement of grand cable as typical response. Table 6 summarizes the result of parametric study. It is seen from Table 6(a) that maximum acceleration response becomes more than double at foundation No.2 (soft and liquefiable) while less than double at foundation No.3 (hard and non-liquefiable). Results in Table 6(b) indicate that the maximum acceleration at foundation No.2 becomes smaller as much as 26% when surrounding ground does not liquefy under strong motion of 70gal. Ground cable displacement decreases as much as 24%. This suggests that softening due to pore water pressure buildup has a significant effect on seismic response of pile foundation and response of cable.

It is seen in Table 6(c) that slight difference of maximum accelerations due to different configuration of bedrock upper boundary but clear difference in cable displacement. It is thought that different configuration of bedrock possibly could cause different ground motion between

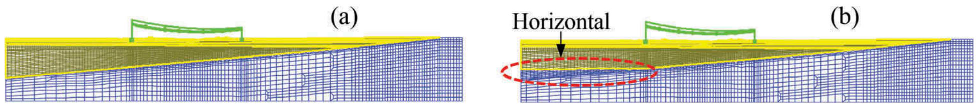


Figure 8. Difference of shape of bedrock.: (a) Uniformly inclined bedrock surface. (b) Partially horizontal bedrock surface.

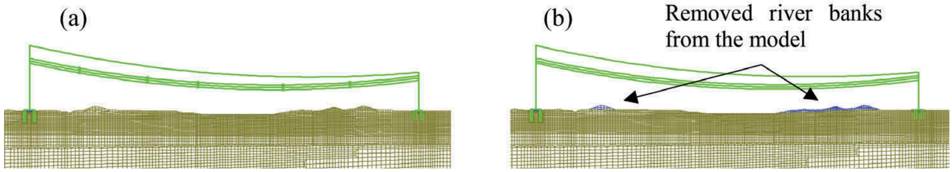


Figure 9. Existence of nearby earth structures (river banks) in the model.: (a) Model with river banks (H=4 to 5m). (b) Model without river banks.

Table 6. Results of parametric study.

a. Intensity of input motion. (Soil liquefaction is considered.)

Location	Max. values	Observed input Motion (i) (Peak acc. 35gal)	Strong input Motion (ii) (Peak acc. 70gal)	Ratio of (ii) / (i)
Foundation No.2	Acc.(gal)	70	156	2.2
Foundation No.3	Acc.(gal)	57	106	1.9
Ground cable No.2 to 3	V. disp.(m)	1.99	4.75	2.4

b. Soil liquefaction at around foundation No.2. (peak acc. 35gal and 70gal is applied.)

		Liquefy		Non liquefy		Difference	
Location	Max. values	35gsl	70gal	35gal	70gal	35gal	70gal
Foundation No.2	Acc.(gal)	70	156	68	116	3%	26%
Foundation No.3	Acc.(gal)	57	106	58	105	2%	1%
Ground cable No.2 to 3	V. disp.(m)	1.99	4.75	1.93	3.63	3%	24%

c. Configuration of bedrock upper boundary. (Observed input motion (peak acc. 35gal) is applied.)

Location	Max. values	Uniformly inclined bedrock	Partially horizontal bedrock	Difference
Foundation No.2	Acc.(gal)	70	65	7%
Foundation No.3	Acc.(gal)	57	59	4%
Ground cable No.2 to 3	V. disp.(m)	1.99	1.28	36%

d. Existence of nearby earth structures. (Model with partially horizontal bedrock is used.)

Location	Max. values	Model with river banks	Model without river banks	Difference
Foundation No.2	Acc.(gal)	65	66	2%
Foundation No.3	Acc.(gal)	59	58	2%
Ground cable No.2 to 3	V. disp.(m)	1.28	1.28	0%

foundations. It is seen from Table 6(d) that no change of seismic responses occurred due to existence of river banks which is 60m to 70m apart from tower foundations.

5 CONCLUSIONS

The major findings as obtained from present study are shown as follows;

(i) It is shown that difference of dynamic displacements of about 2cm to 3cm could occur between two transmission tower foundations during 3.11 earthquake. This is because of different ground conditions between transmission towers of Vs30 values (No.2: 200m/s and No.3: 300m/sec), liquefaction susceptibility (No.2: liquefiable and No.3: non-liquefiable), and depth of bedrock. Those different conditions are thought to result in prominent excitation of cables of as much as 2m.

(ii) Based on the parametric study, it is found that the intensity of input ground motion and liquefaction occurrence could be very sensitive to seismic response of transmission tower cables. Configuration of bedrock upper boundary affects a few to seismic response. Existence of river banks with height of about 4m has no effect on the cable response.

We conclude from the study here that transmission lines could seismically excite significantly at the location where the ground conditions of foundations were very different in terms of liquefaction susceptibility, Vs30 values and depth of bedrock.

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