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Dynamic Characteristics of Pile Foundation Buildings with High Aspect Ratio in Relation to Tilting Earthquake Damage

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Abstract

Tilting damage of pile foundation buildings with high aspect ratio is one of typical earthquake damage and very important issue in earthquake engineering. The author has contributed to investigate the earthquake damage and also implemented measurement of vibration characteristics of the damaged buildings together with dynamic response characteristics using a simple model. It is suggested that the lack of tension resistance of pile head led to partial uplifting followed by compression failure at pile head on the opposite side of the foundation.

This paper describes interpretation of the earthquake damage based on analytical investigation for one of the damaged 14-story pile-foundation buildings with aspect ratio of about 3.7 during the 2011 Tohoku earthquake. The building was constructed in 2000 at the site with high contrast surface layer on rock. It has 1 span in the transverse direction and 5 spans in the longitudinal direction comprising 12 cast-in-place-concrete piles with diameter of 2.2m (2200φ) and length of 26m. Thin Layer Element Method is used for considering dynamic soil-structure interaction, such as pile group effects of soil impedance and foundation input motion of pile foundation. Using soil-structure coupling models, earthquake response characteristics of the building are qualitatively and quantitatively investigated by considering changes of building stiffness and piles' elastic stiffness. The moment response values applied to the foundation are compared to the threshold value of partial uplifting of the building. As input motions defined as bedrock motions, earthquake observation records during the 2011 Tohoku earthquake obtained at the JMA station near the site are used together with pulse-like motions as Ricker pulse and Fling-Step pulse with a dominant period of 1s. The moment response values are discussed for seismic countermeasures for the tilting problem and highlighting those considering resonance phenomena.

Keywords: Earthquake Damage, Pile Foundation Building, Soil-Structure Interaction, Partial Uplifting, Resonance phenomena

1. Introduction

Tilting damage of pile foundation buildings with high aspect ratio is one of typical earthquake damage, as was reported for 12-story pile-foundation building during the 1978 Miyagi-ken Oki earthquake (Shiga, 1979)[1]. The similar tilting damage of 14-story buildings with high aspect ratio was recognized due to the 2011 Tohoku earthquake. These are pile foundation buildings with single span in the transverse direction, and multi-spans in the longitudinal direction. The author has contributed to investigate the earthquake damage (eq. Motosaka and Mitsuji, 2013)[2] and also implemented measurement of vibration characteristics of the damaged buildings together with dynamic response characteristics using a simple model (Fukuoka, Motosaka, et al, 2018)[3]. It is suggested that the lack of tension resistance of pile head led to partial uplifting followed by compression failure at pile head of the opposite side. This means the applied rotational moment at the foundation compared to the critical moment determined from the building weight becomes important factor to ascertain black and white for the inclination damage. Although the seismic design's regulation of foundation for a building with larger aspect ratio of 4 become severe, general buildings with aspect ratio less than 4 are not necessary sufficient. The investigated two buildings have common factors, namely, 14-story SRC building with aspect ratio of about 3.7 constructed at the alluvial soil site with the soft surface layer of about 25m thickness. During severe shaking, dominant period of the equivalent surface layer velocity of 100m/s becomes about 1 second, which lead to resonance amplification of the pile foundation building. There are many buildings which have the similar conditions to raise resonance not only in Japan but also all over the earthquake prone countries. But the severe resonance problem is not sufficiently considered yet in the seismic design. In the future the similar tilting damage of the pile foundation building with high aspect ratio can occurr.

This paper describes interpretation of the earthquake damage based on analytical investigation for one of the damaged 14-story pile-foundation buildings during the 2011 Tohoku earthquake. In the following Section 2, the objective pile-foundation building is explained together with the simplified site conditions. The building was constructed in 2000 at the site with high contrast surface layer on rock, which has 1 span in the transverse direction and 5 spans in the longitudinal direction comprising 12 cast-in-place-concrete piles with diameter of 2.2m (2200φ) and length of 26m. In Section 3, dynamic soil-foundation interaction analysis is described using Thin Layer Element Method (TLEM). The impedance and the foundation input motion (FIM) characteristics of the 12-pile foundation are described including pile group effects to impedance and effect of piles' stiffness reduction to impedance and FIM. In Section 4, dynamic response characteristics of the building using Soil-Structure Interaction (SSI) models are described. The dynamic response characteristics of the building are qualitatively and quantitatively investigated by considering changes of building stiffness and piles' elastic stiffness. The moment response values applied to the foundation are compared to the threshold value of partial uplifting. As input motions defined as bedrock motions, earthquake observation records including those during the 2011 Tohoku earthquake obtained at the JMA station near the site are used together with pulse-like motions as Ricker pulse and Fling-Step pulse with dominant period of 1s. The moment response values are discussed for seismic countermeasures for the tilting problem and highlighting those considering resonance phenomena. Conclusions are described in Section 5.

2. Objective Pile-foundation Building

The objective building damaged during the 2011 Tohoku earthquake has 1 span in the transverse direction and 5 spans in the longitudinal direction with dimension of 10.8mx32.5m and height of 39.5m. Fig.1 shows the photo of the building and the feature of the tilting with 1/105 in the transverse direction and 1/500 in the longitudinal direction. The building is 14-story SRC building with ramen structure in the longitudinal direction and ramen + shear wall in the transverse. Its aspect ratio is 3.66 in the transverse direction and 1.21 in the longitudinal direction. The building is constructed at the soft soil site with 25m layered on rock .Its foundation comprises 12 cast-in-place-concrete piles with length of 26m and diameter of 2.2m (2200φ) .

Fig. 2 shows illustrative figure of the pile foundation building. Fig. 3 shows the plan of the building assumed to be the same span length in this study. It is noted that surface layer is modelled as a single layer with shear velocity of 100m/s and the supporting rock is considered as shear velocity of 1000m/s.

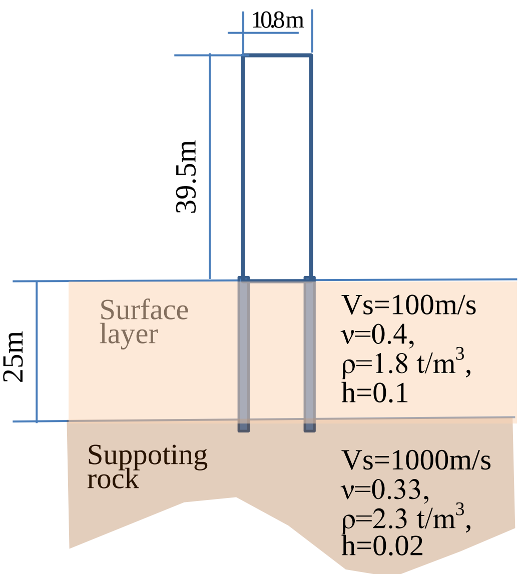


Fig.2- llustrative figure of the pile foundation building

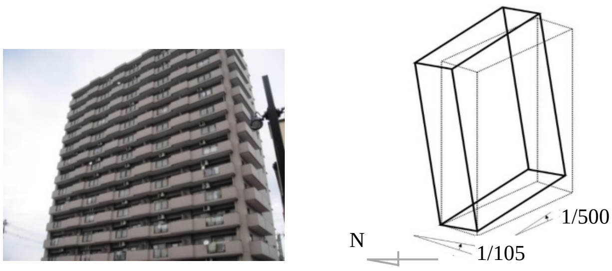


Fig.1- Damaged building and feature of the tilting[3]

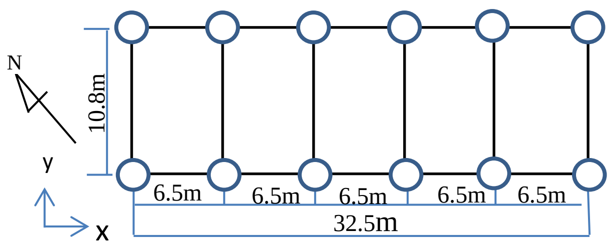


Fig.3 - 12 piles' layout of the building

Table 1 shows information on weight and shearing stiffness of each floor of the building based on structural calculation sheet. It is noted that the natural period of the building for fixed-base condition is 0.93s in the longitudinal direction and 0.48s in the transverse direction.

Table 1 – Information on weight and sear shearing stiffness of each floor of the 14-story building

Floor	Weight	Shear Force	Story def.-y (cm)	Story stiff.-y (t/cm)	Story def.-x (cm)	Story stiff.-x (t/cm)
14fl	561	312	0.208	1501	0.295	1059
13fl	566	500	0.208	2402	0.423	1181
12fl	569	661	0.212	3116	0.528	1251
11fl	574	806	0.213	3783	0.595	1354
10fl	580	938	0.204	4598	0.613	1530
9fl	591	1060	0.198	5353	0.637	1664
8fl	593	1171	0.191	6130	0.668	1753
7fl	597	1271	0.182	6985	0.651	1953
6fl	606	1363	0.169	8064	0.624	2184
5fl	612	1445	0.154	9381	0.636	2271
4fl	615	1517	0.136	11152	0.637	2381
3fl	623	1580	0.110	14359	0.620	2548
2fl	631	1633	0.089	18347	0.583	2801
1fl	675	1679	0.103	16298	0.316	5312

3. Dynamic Soil-Foundation Interaction Analysis

3.1 Impedance characteristics of the pile foundation

(1) Analytical method and model

In this study, impedance characteristics of the 12-pile foundation for horizontal and rocking excitation by using Thin Layer Element Method (TLEM) are investigated [4]. For the 2-layer soil model shown in Fig.2, the 17 layer TLEM model with each thickness of 2m for the surface layer and 5m for the supporting soil with exception of the border with 1m thickness element is used. The viscous damper is considered as the bottom boundary condition. In the impedance analysis, single pile’s impedance is calculated and compared with the 12 piles’ impedance from the view point of group pile effect. Two piles’ impedances are also calculated for rocking excitation. The impedance characteristics for the different stiffness of the pile are also investigated considering material stiffness reduction and pile’s diameter.

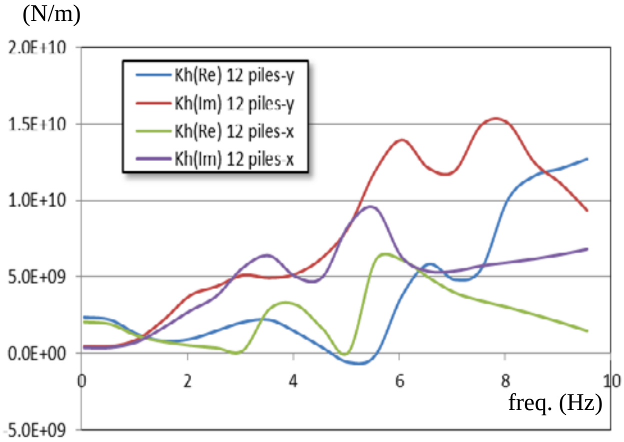
Original stiffness of the cast-in concrete pile with 2.2m diameter is assumed to be 2.7×10^{10} N/m^2 for Young’s modulus and Poisson’s is 0.2. The stiffness reduced cases for 50% and 25% are investigated.

(2) Analytical results

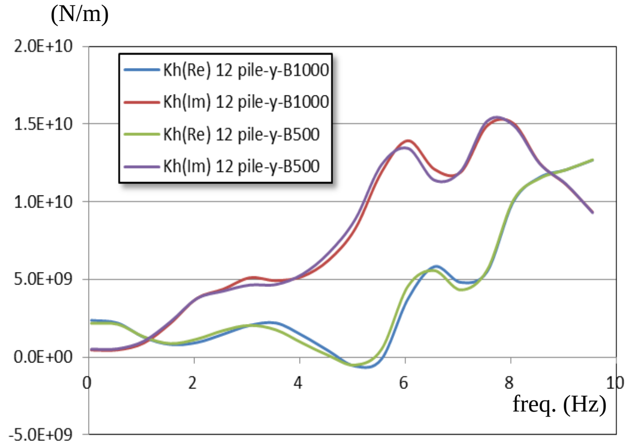
Fig.4 shows comparizon of horizontal and rocking impedance of the 12 pile foundation in the transverse (y) direction and longitudinal (x) direction. It is recognized that the horizontal impedance is almost the same in the two directions. The rocking impedance of the transverse direction is quite smaller compared to that of the longitudinal direction. The frequecy dependency is quite different between the horizontal and the rocking impedances. To investigate effect of suporting rock velocity to impedance characteristics, comparison of impedance of the 12 pile foundation in the transverse (y) direction for different suporting rock velocity, 1000m/s and 500m/s, is shown Fig.5. It is found from this figure that almost no difference in horizontal impedance but the clear difference appears for the rocking impedance.

Fig.6 shows group pile effect of impedance of 12 pile foundation in the transverse (y) by comparing the 12 times values of single pile’s impedance. The horizontal impedance is compared to that of 12 times of single pile impedance. For the rocking impedance, 6 times of the two piles corresponding to the single span is compared. The group pile effect is clearly recognized, namely the 12 piles horizontal impedance is 26% at

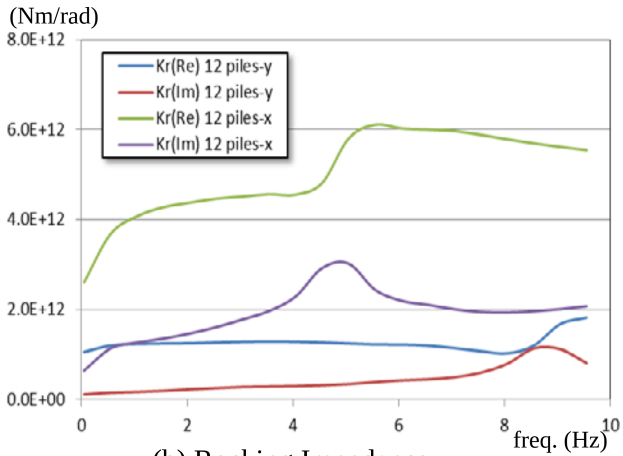
1Hz compared to 12 times of the single pile impedance. For the rocking impedance, the 12 piles impedance is 85% at 1Hz compared to 6 times of the 2 piles rocking impedance.



(a) Horizontal impedance

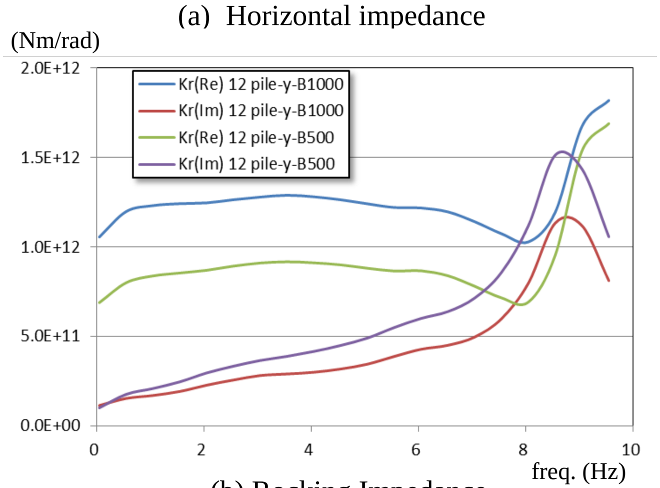


(a) Horizontal impedance



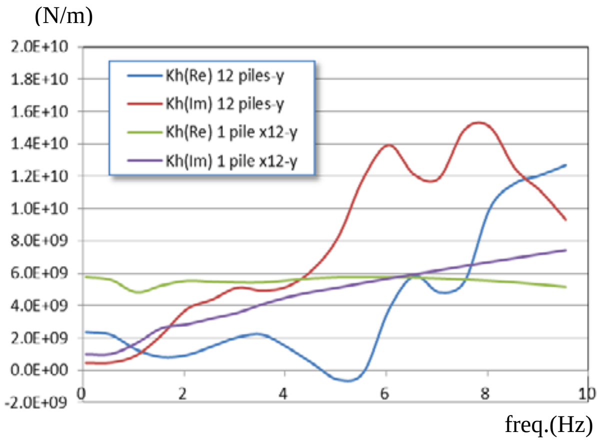
(b) Rocking Impedance

Fig.4 - Comparison of Impedance of 12 pile foundation in the transverse (y) direction and longitudinal (x) direction

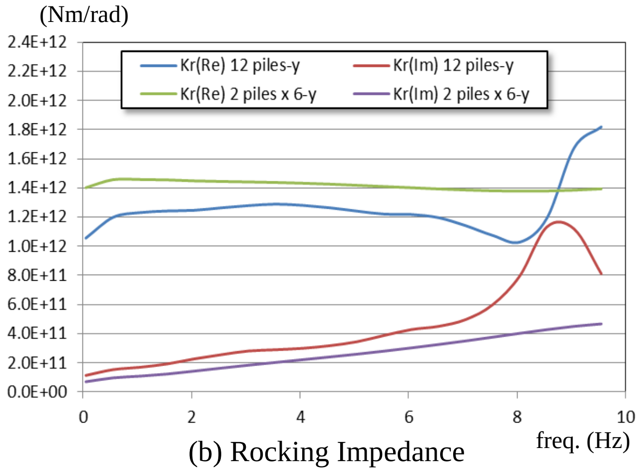


(b) Rocking Impedance

Fig.5 - Comparizon of Impedance of 12 pile foundation in the transverse (y) direction for different suporting rock velocity



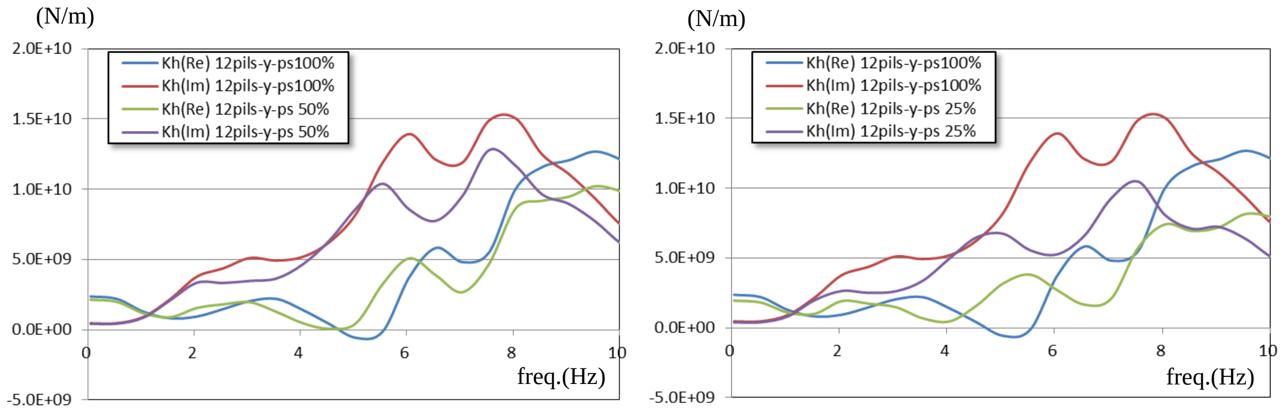
(a) Horizontal Impedance



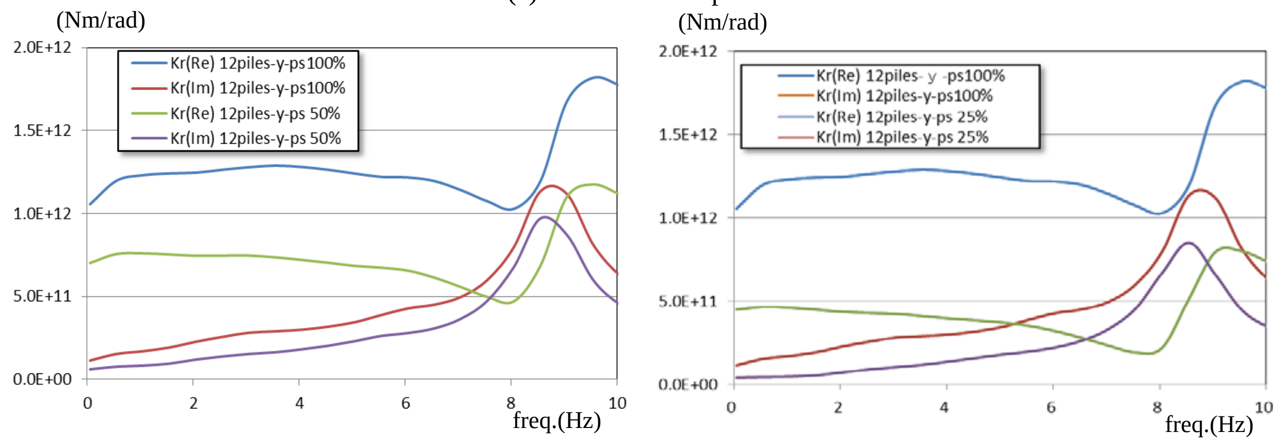
(b) Rocking Impedance

Fig.6 - Group pile effect of impedances of 12 pile foundation in the transverse (y) by comparing the 12 times values of single pile's horizontal impedance and 6 times values of the 2 piles rocking impedance

Fig.7 shows impedance of 12 pile foundation in the transverse (y) for different stiffness of the piles. The left figure shows comparison of 100% and 50% and the right figure shows comparison of 100% and 25%. It is regonnized that the change of horizontal impedance due to pile stiffness reduction is very small at around the 1Hz but the stiffness reduction effect appears clealy in the rocking impedance



(a) Horizontal impedances



(b) Rocking impedances

Fig.7 - Impedance of 12 pile foundation in the transverse (y) for different stiffness of the piles

3.2 Foundation input motion characteristics

(1) Analytical method

Based on the 17-layer TLEM model combined with 12 pile model, a simple soil spring model consisting of side spring and shear spring corresponding to each thin layer is used[5],[6] for calculating foundation input motion (FIM). By applying the free field motion in each thin layer from the axial spring, FIM is calculated. The frequency domain analyses of free field motion and FIM are performed in the frequency domain using the transfer functions to bedrock motion.

(2) Analytical results

Fig.8 shows transfer functions of foundation input motions to bedrock motion compared with free surface motion for different pile stiffnesses in the transverse (y) direction and those in the longitudinal(x) direction. It is noted that the left figures show the transfer functions in the transverse (y) direction and the right figures show those in the longitudinal (x) direction. It is recognized that the amplification at around 1Hz of FIM indicates quite smaller compared to bedrock motion even though the amplification of FIM increases with decrease of pile stiffness. The FIM characteristics of the transverse direction and that of the longitudinal direction is quite similar.

Fig.9 shows waveform of FIM for Ricker wavelet with dominant period of 1s in the transverse (y) direction. The left figure shows comparison of FIM for 100% pile stiffness and bedrock motion, and the right figure shows FIM for 50% and 25%. It is recognized that the maximum acceleration of FIM increases with decsease of pile stiffness as expected from the transfer functions in Fig.7.

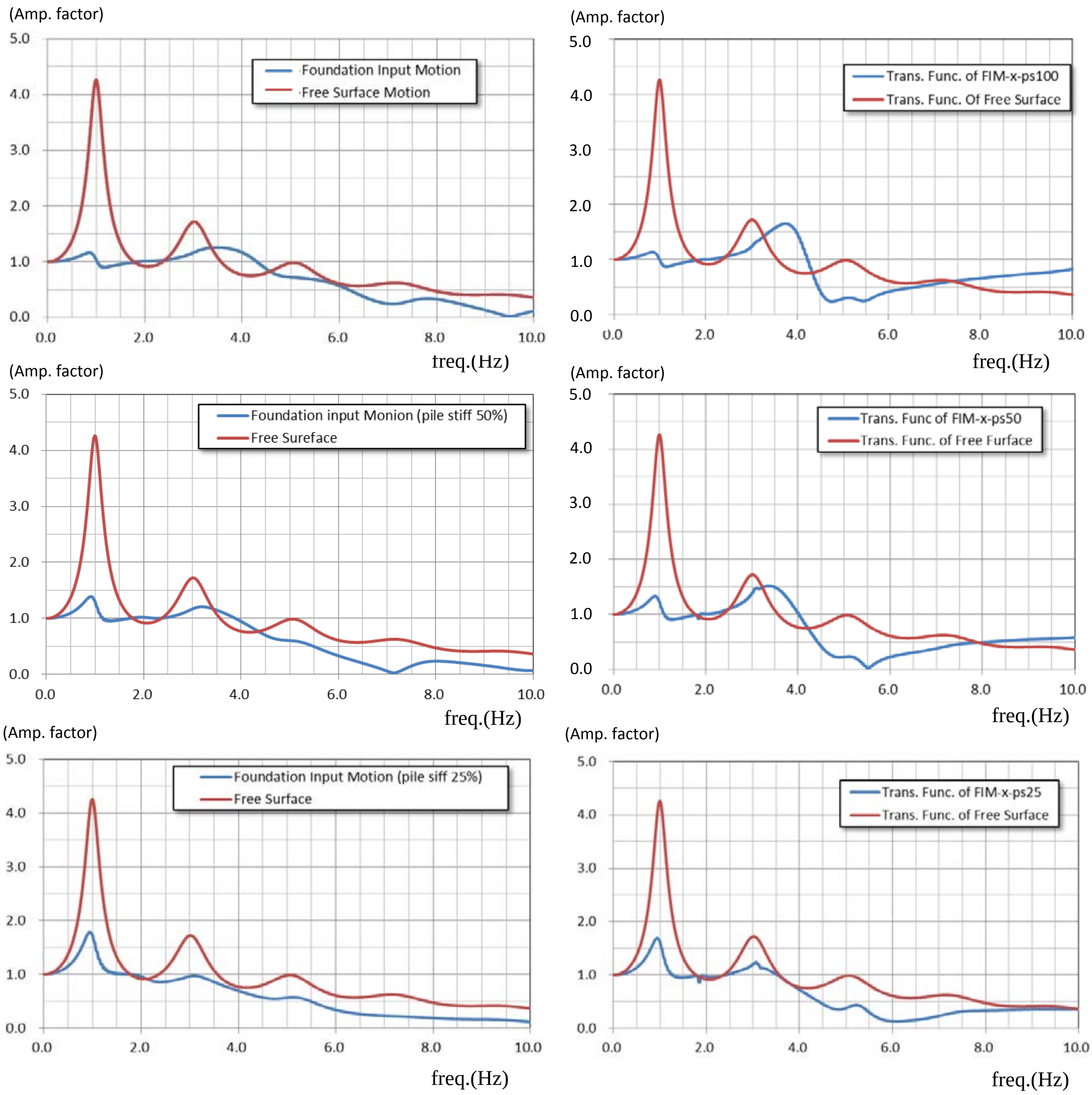


Fig.8 -Transfer functions of foundation input motions to bedrock motion compared with free surface motion for different pile stiffnesses. Left: Transfer functions in the transverse (y) direction, Right: those in the longitudinal (x) direction

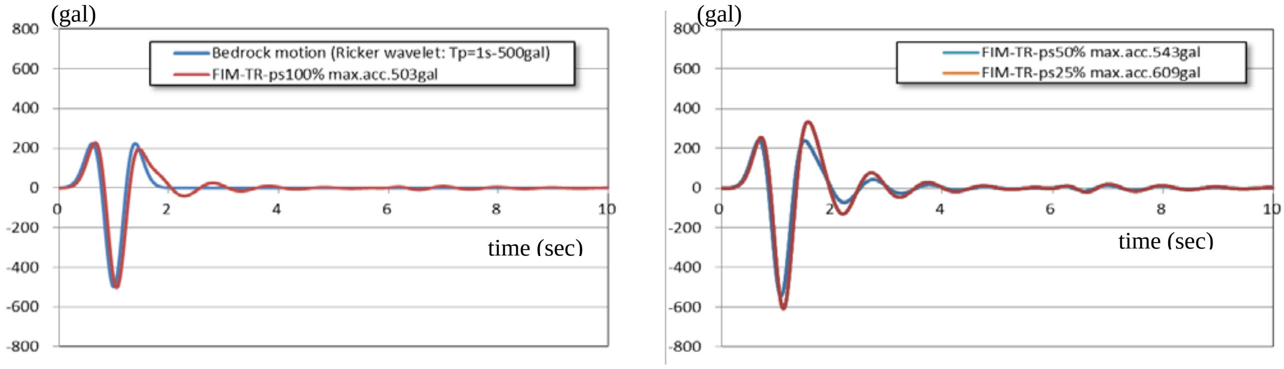


Fig.9 - Foundation input motion for Ricker wavelet with dominant period of 1s in the transverse (y) direction. Left: comparison of FIM for 100% pile stiffnes and bedrock motion, Right:FIM for 50% and 25%

4. Dynamic Response Analysis of the Building

4.1 Analytical model and critical uplifting moment

Analytical modelling of super-structure is based on the weight and stiffness of each floor as shown Table1. Swaying and rocking motions are considered based on the impedance analysis taken into account the group pile effect as described in Section 2 and the foundation input motion in Section 3. The pile foundation building is modelled as 16-DOF SSI system. It is noted that the total weight of the building is 9,693 ton by assuming foundation weight of 1,300 ton, and that damping factor of the superstructure is 0.05 (5%). Dynamic response characteristics are investigated using the SSI model for the foundation input motion. In this study, the time history response of the SSI system is calculated by frequency response analysis using the transfer functions of each part of building to bedrock motion. The maximum response values are investigated focused on the uplifting of the building.

The critical uplifting moment of single span building in the transverse direction, Mc, is evaluated with the following equation (1) using the building width (B) and the weight (W), which is derived from the condition that induced vertical force (F_v) of equivalent vertical spring corresponding to the rocking impedance exceeds the half of the total weight (W). The corresponding vertical displacement d_{vc} at the foundation to the critical moment Mc is also discussed defined as the following equation (2) using the rocking stiffness Kr in order to investigate the orthogonal effect from building response by longitudinal input motion.

$$Mc = F_v \cdot B \geqq W/2 \cdot B \tag{1}$$

$$d_{vc} = (B/2) \cdot Mc/Kr \tag{2}$$

It is noted that the critical moment Mc=5.23E+6 [t-cm], and the critical vertical displacement d_{vc} =0.628cm based on the rocking impedance at 1Hz in the pile stinness case of 25%, 0.353cm for 50% and 0.217 cm for 100%.

The dynamic response characteristics due to two horizontal motions are investigated considering that the induced vertical motion due to input motion in the longitudinal direction is contribute to the axial force of the corner of the pile foundation and related to the uplifting.

4.2 Analytical conditions and input motions

As analytical conditions, the reduction of building stiffness is considered for 50% and 25% compared to the original stiffness shown in Table 1. Also the pile stiffness reduction cases are investigated for 50% and 25% of which impedance characteristics are discussed in Section 3.1, and foundation input motion in Section 3.2. These are investigated as parametric analyses focused on the resonance response at around 1s period. Table 2 shows basic dynamic characteristics of the SSI system for (a) Transverse (TR) direction and for (b) Longitudinal (LN) direction, respectively.

Regarding the basic dynamic characteristics in TR direction, it is found that change of natural frequency due to pile stiffness reduction is larger for the original building stiffness and rocking ratio becomes larger, while in case of 50% building stiffness, the natural frequency change is small at about 1Hz, the resonance frequency range for input motion with dominant frequency of 1Hz. It is noticed that the rocking ratio increases with decrease of pile stiffness, that increase of rocking ratio is consistent with the increase of the induced moment to the foundation. While, in the LN direction, it is found that change of the natural frequency is smaller for the pile stiffness reduction even though the natural frequency change is larger for the building stiffness change. It is also recognized that the sway-rocking ratio is quite small in LN direction. The natural frequency for the original building stiffness is in the resonance frequency range for input ground motions with dominant frequency at around 1Hz.

Table 2(a) - Basic Dynamic Characteristics of SSI system for Stinnsess Change (TR)

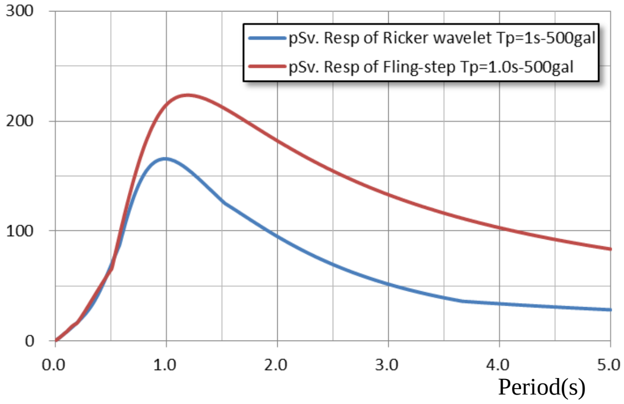
Building Stiffness	Pile Stiffness	Top disp by Triangle Load (cm)	Sway-Rocking Ratio	Rocking Ratio	Natural Freq (Hz)
Original	100%	9.5	0.579	0.353	1.538
	50%	11.5	0.650	0.442	1.280
	25%	14.6	0.725	0.546	1.074
50%	100%	13.5	0.407	0.249	1.123
	50%	15.5	0.482	0.328	1.050
	25%	18.6	0.568	0.428	0.952

Table 2(b) - Basic Dynamic Characteristics of SSI system for Stinnsess Change (LN)

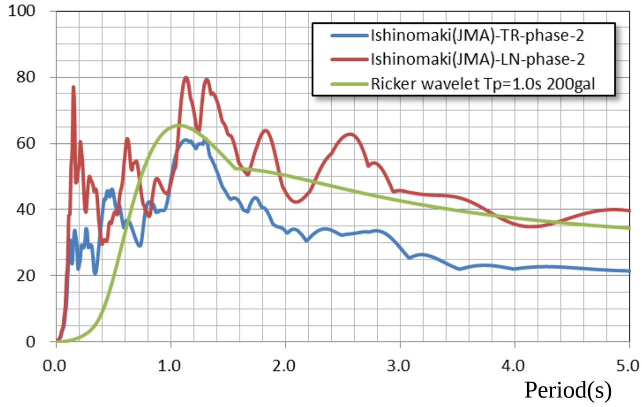
Building Stiffness	Pile Stiffness	Top disp by Triange Load (cm)	Sway-Rocking Ratio	Rocking Ratio	Natural Freq (Hz)
Original	100%	17.8	0.216	0.077	0.952
	50%	18.4	0.245	0.097	0.928
	25%	19.4	0.283	0.128	0.903
50%	100%	31.7	0.121	0.043	0.732
	50%	32.3	0.140	0.055	0.708
	25%	33.3	0.165	0.075	0.708

Regarding input ground motion at bedrock, Ricker wavelet and Fling-Step pulse with dominant period of 1s are considered as pulse-like motions. As earthquake ground motions, the observation records at JMA Ishinomaki during the 2011 Tohoku earthquake are used as the site motion near the tilting damage building. In this case, the observed ground motions in the NS direction and EW directions are converted to the transverse direction (NE-WS) and longitudinal direction (NW-SE) of the building to investigate the dynamic responses not only for the transverse direction but also for orthogonal effect from the longitudinal response.

Fig.10 shows pseud velocity spectra (h=0.05) of input ground motions at bedrock for (a) pulse-like motions and (b) the earthquake ground motions at JMA Ishinomaki, respectively.



(a) Ricker wavelet and Fling-Step Pulse (500 gal, Tp=1s)



(b) JMA Ishinomaki (TR and LN directions)

Fig.10 - Psued-Velocity Spectra of Input Bedrock Ground Motions

4.3 Frequency response characteristics of the building

Transfer functions of building responses at top, middle and bottom to bedrock motion for different pile stiffness in the transverse (y) direction and those in the longitudinal(x) direction are shown in Fig.11. The left figures are transfer functions for TR direction and the right figures are those for LN direction. The top figures are transfer functions for 100% pile stiffness, middle figures are for 50% and bottom figures are for 25%. It is recognized that the amplification factor at the peak frequency, namely the natural frequency shown in Table 2, increases with decrease of pile stiffness in TR direction but almost no change in LN direction.

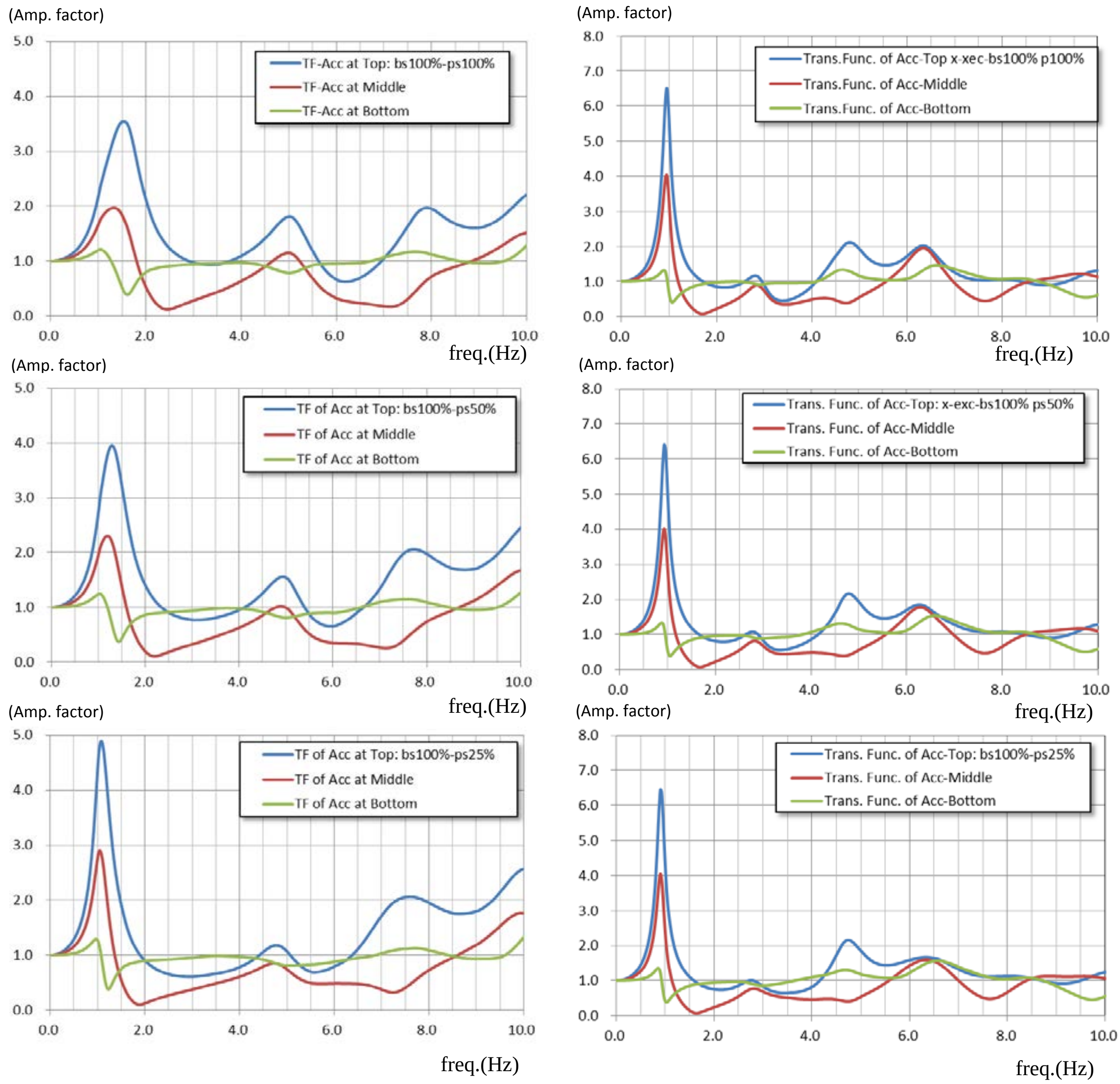


Fig.11 - Transfer functions of building responses at top, middle and bottom to bedrock motion for different pile stiffnesses in the transverse (y) direction and those in the longitudinal(x) direction

4.4 Maximum response values of the building and example of time history response

(1) Response characteristics for pule-like ground motions

The maximum response values in the transverse direction for Ricker wavelet and Fling-Step pulse with 1s dominant period and with maximum acceleration of 500gal are shown in Table 3 for the different building stiffness (original and 50%) and different pile stiffness (100%, 50%, 25%). In this table, the ratio of induced moment at foundation to the critical moment defined as Eq. (1) is shown together with the corresponding critical bedrock acceleration for uplifting. It is found form this table that the building response values in cluding induced moment at foundat incease with decrease of pile stiffness, and that the response valus for the fling-Step pulse are lager than those of Ricker wavelet. In case of original pile stiffness, that critical acceleration level to cause uptifting is about 180gal for Ricker wavet and less than 160gal for Fling-Step pulse. In case of 25% pile stiffness th critical acceleration is about 130 gal for Ricker wavelet and about 100 gal for Fling-Step pulse.

Table 3 - Maximum Responses for Ricker Wavelet and Fling-Step Pulse with 1s dominant period

Building Stiffness	Pile Stiffness	Ricker Wavelet(Tp=1.0s-500gal)					Fling-Step Pulse(Tp=1.0s-500gal)				
		Top Disp (cm)	Vertical Base-Edge Disp (cm)	Induced Moment at Founda-tion(t･cm)	Ratio to Critical Uplifting	Critical Bed-rock Acc. For Uplift (gal)	Top Disp (cm)	Vertical Base-Edge Disp (cm)	Induced Moment at Founda-tion(t･cm)	Ratio to Critical Uplifting	Critical Bed-rock Acc. For Uplift (gal)
Original	100%	13.4	0.615	1.43E+07	2.73	183	15.5	0.697	1.64E+07	3.14	159
	50%	18.3	1.110	1.58E+07	3.02	166	23.6	1.420	2.05E+07	3.92	128
	25%	29.9	2.290	2.00E+07	3.82	131	38.3	2.930	2.55E+07	4.88	103
50%	100%	21.6	0.641	1.48E+07	2.83	177	28.7	0.853	1.99E+07	3.80	131
	50%	27.6	1.160	1.68E+07	3.21	156	37.1	1.560	2.25E+07	4.30	116
	25%	39.6	2.280	1.98E+07	3.79	132	50.9	2.940	2.55E+07	4.88	103

Fig.12 shows the effect of pile stiffness reduction to building response at top in the TR directionn in the cases of 50% building stiffness for Ricker wavelet with Tp=1s and Max.acc. 500 gal (cm/s^2). The natual frequency is shown in Table 2(a). It is recognized that the displacement response value inceases with decrease of pile stiffness, and it is recognised from pulse-like input motion that the damping facter becomes smaller with decrease of pile stiffness. Fig.13 shows the comparison of the building response of the building with 50% builfig stiffnes and 25 pile stiffness at top in the TR directionn for Ricker wavelet and Flig-Step pulse. As is understood from Fig.9(a) that the response valus for the fling-Step pulse are lager than those of Ricker wavelet.

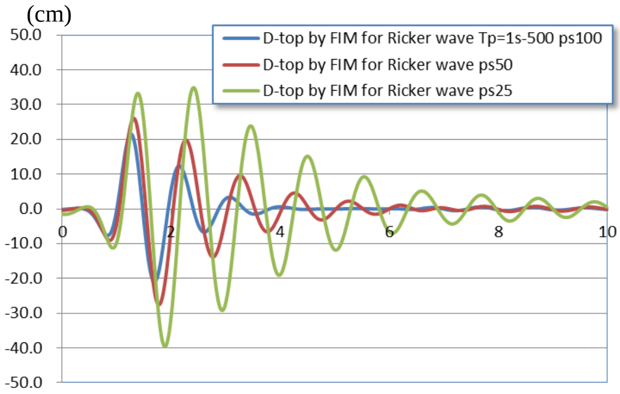


Fig.12 - Effect of pile stiffness to building response at top due to Ricker wavelet with Tp=1s and 500gal (Building stiffness:50%)

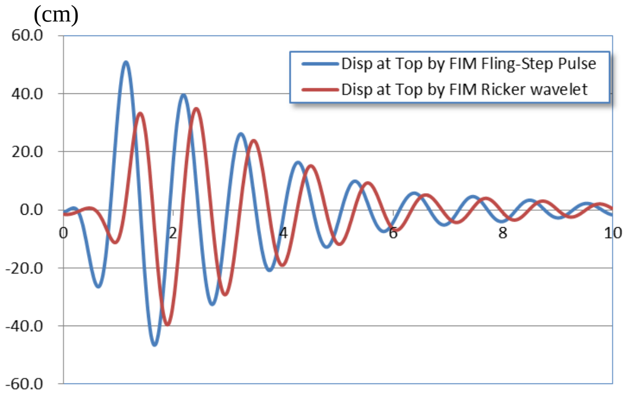


Fig.13 - Comparison of building response at top for Ricker waveletand Fling-Step pulse (Building stiffness:50%, Pile stiffness25%)

(2) Response characteristics to earthquake motions

The earthquake response values to discuss the tilting damage of the objective building using the observation records (JMA Ishinomaki) near the site are shown as Table 4 and Table 5. It is noted that the earthquake response analysis is performed for the distinctive two phases of the long-dudation records, phase-1 for the time section of 40 seconds from 30sec to 70sec, and phase-2 for also 40 seconds from 75sec to 115sec of the original records. In the paper, earthquake response analysis for the phase 2 is shown because the response values are larger than those of the phase 1.

Table 4 shows the maximun responses in the transverse (TR) direction and in the longitudinal (LN) direction. It is recognized that the induced moment response values in TR direction are smaller than the critical uplifting moment, and that the induced vertical displacement at the foundation corner is larger in LN direction. Table 5 shows the maxmaum response values due to the two horizontal directions earthquake motions. The response time histories for TR direction and LN direction are combined as LN+TR and LN-TR and the obtained maximum values are compared to the critical value denoted by Eq.(2). The response values of two direction motions to those of TR direction are also shown as the orthogonal effect. It is found that the ratio to the critical uplifting exceeds 1.0, and that the smaller pile stifness case value is lager. These results are consistent with the tilting damage feature of the building, not to TR direction but to inclined azimuth[3].

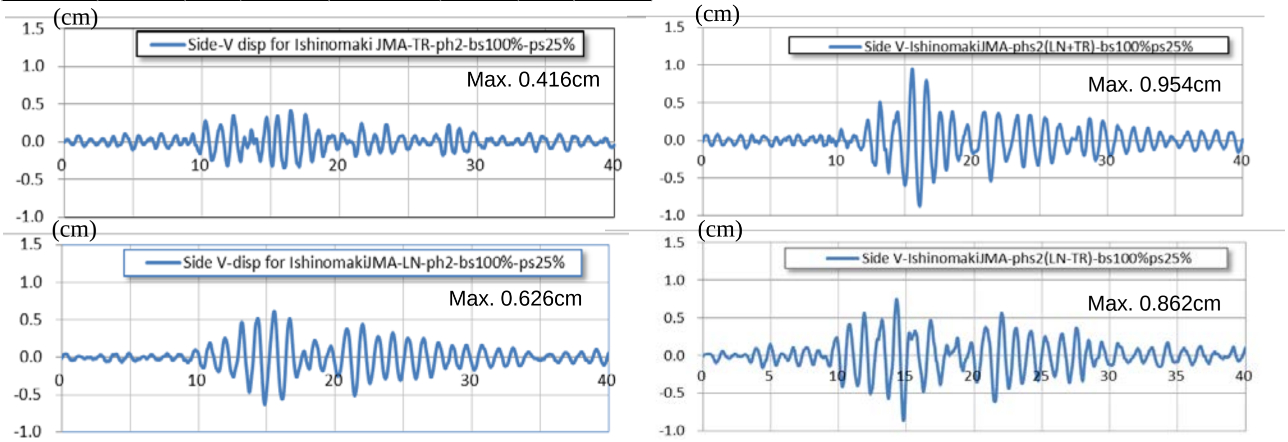
Fig.14 shows the time history response of vertical displacement at foundation corner due to rocking motion for (a) One direction input motion, and for (b) two horizontal directions' input motions. It is noted that the vertical displacement of the foundation corner is directly related to axitial force of the column and pile. It is found that the vertical displacements due to the coupling motions become larger than two times as orthogonal effect compared to those due to only TR direction input.

Table 4 – Max. Responses for the Earthquake Motions

Building Stiffness	Pile Stiffness	JMA Ishinomaki (TR: NE-SW)				JMA Ishinomaki (LN: NW-SE)	
		Top Disp (cm)	Vertical Side Disp (cm)	Induced Moment at Foundation (t・cm)	Ratio to Critical Uplifting	Top Disp (cm)	Vertical Side Disp (cm)
Original	100%	2.92	0.135	3.16E+06	0.60	15.5	0.170
	50%	3.48	0.223	3.20E+06	0.61	23.6	0.310
	25%	5.47	0.416	3.62E+06	0.69	38.3	0.626
50%	100%	3.78	0.109	2.61E+06	0.50	28.7	0.220
	50%	5.12	0.217	3.16E+06	0.60	37.1	0.353
	25%	8.45	0.487	4.26E+06	0.81	50.9	0.606

Table 5 – Max. Coupling Corner Response for Two Horizontal Direction Earthquake Motions

Pile Stiffness	Building Stiffness		Max.Vetical Corner Disp (cm)		Coupling Resp /TR Resp	Ratio to Critical Uplift
	LN	TR	LN+TR	LN-TR		
100%	100%	100%	0.219	0.239	1.77	1.10
		50%	0.257	0.227	2.36	1.18
50%	100%	100%	0.429	0.371	1.92	1.22
		50%	0.473	0.438	2.18	1.34
25%	100%	100%	0.954	0.862	2.29	1.52
		50%	0.940	0.949	1.95	1.51



(a) One direction input motion
(Top: TR direction, Bottom: LN direction)

(b) Two horizontal directions' input motions
(Top: LN+TR, Bottom: LN-TR)

Fig.14 - Comparison of vertical displacement at foundation corner due to rocking motion for TR-direction input and two horizontal directions' input (Building stiffness:100%, Pile stiffness25%)

5. Conclusions

In this paper, analytical investigation is performed focused on the tilting earthquake damage of building with high aspect ratio. In the analysis, the SSI model based on the TLEM for pile foundation combined with the building's structural data is applied to one of the damaged 14-story pile-foundation buildings during the 2011 Tohoku earthquake. Earthquake observation records during the 2011 Tohoku earthquake obtained at the JMA station near the site are used together with pulse-like motions as Ricker pulse and Fling-Step pulse with dominant period of 1s. The earthquake response characteristics of the building focused on moment response applied to the foundation compared to the threshold value of uplifting are qualitatively and quantitatively investigated by considering changes of building stiffness and piles' elastic stiffness. The moment response values are discussed for seismic countermeasures for the tilting problem and highlighting those considering resonance phenomena. The obtained findings are as follows.

1) Regarding impedance characteristics of the 12 piles' foundation, the group pile effect is clearly recognized. Effect of pile stiffness reduction is very small for horizontal impedance but sensitive for the rocking impedance directly related to rocking motion of the building.

2) Regarding FIM characteristics of the 12 piles' foundation, FIM increases with decreasing pile stiffness but much smaller compared to free surface motion.

3) As for the dynamic response characteristics in the transverse direction due to the pulse-like ground motion, it is found f that the building response values including induced moment at foundation increases with decrease of pile stiffness, and that the response values for the fling-Step pulse are larger than those of Ricker wavelet. The critical acceleration level to cause uplifting is about 180gal for Ricker wavelet and less than 160gal for Fling-Step pulse in case of original pile stiffness. In case of 25% pile stiffness, the critical acceleration is about 130 gal for Ricker wavelet and about 100 gal for Fling-Step pulse.

4) Earthquake response characteristics using the observation data, it is recognized that the induced moment response values in TR direction are smaller than the critical uplifting moment. But by considering the response values of two direction motions in order to take in to account orthogonal effect, the ratio to the critical uplifting exceeds 1.0, and that the smaller pile stiffness case value is lager. These results are consistent with the tilting damage feature of the building.

It is recommented that the tension resistant capacity enough fot the building with high aspect ratio considering the resonant vibration and orthogonal effect, if not, the similar tilting damage could be occurred.

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