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SEISMIC BEHAVIOR OF PILES SUPPORTING TALL BUILDINGS AND THE CONSIDERATION OF GROUND RESPONSE EFFECTS IN DESIGN

Yoshihiro Sugimura — Professor, Faculty of Engineering, Tohoku University, Sendai, Japan
K. Fujiwara & T. Ohgi — Mgr. & Engr., Architectural Engg. Div., FUJITA Corp., Tokyo, Japan
Madan B. Karkee — Manager, Development Division, GEOTOP Corporation, Tokyo, Japan

Abstract: The paper concerns analytical study of the seismic behavior of piles supporting tall reinforced concrete buildings at locations of deep soil deposit such as the Tokyo bay area. The nonlinear response of the soil-pile-structure system under extreme earthquake shaking was investigated by 2-D FEM analysis considering three simple soil conditions. It is seen that the response forces on piles from the soil-pile-building interaction system can be resolved into those from two constituting systems consisting of the building part and the foundation part, similar in concempt to the modal resolution in elastic analysis. Large response forces develop at interfaces of layered soil deposit reflecting strong influence of ground response effects, which is not adequately accounted for in the current pile design practice. A simple design approach based on the use of the design response spectra, and capable of accounting for the ground response effects, is proposed for practical application.

Introduction

The Hyogoken-Nambu earthquake of January 17, 1995 caused widwspread damage to different types of structures. Severe damage to pile foundations are also reported (Sugimura 1997 and Karkee et.al. 1997). The importance of the soil-pile-structure interaction to behavior of pile foundation was amply delineated in the 1978 Miyagiken-Oki earthquake (Sugimura 1981, 1987). Research efforts to study the interaction phenomenon led to the Japanese government design guide summarized by Sugimura (1988) and published by the Building Center of Japan (BCJ, 1984) as a guideline applicable to small to medium earthquakes. It was even before a design guideline for an extreme level earthquake was put forward that the Hyogoken-Nambu earthquake struck, again resulting in severe damage to pile foundations (Karkee et.al. 1997).

Three dimensional (3-D) analysis of space frame is noted to have made steady progress in Japan for better understanding of the seismic response of building superstructures. No similar progress can be noticed concerning interaction analysis of a soil-pile-building system, where even 2-D analysis examples are not very common. Quantitative information on the seismic resistance of piles considering nonlinear behavior during extreme seismic events is indeed very limited. Damage to piles during earthquakes may result from two distinct phenomenon: **ground shaking** and **ground failure** (Karkee et.al. 1997 & Karkee and Kishida 1997). While the ground failure aspects are not amenable to easy analytical solution, ground shaking effects are satisfactorily evaluated analytically. Under earthquake shaking, forces induced in piles can be considered to originate from two sources: **superstructure inertia** and **ground response**. The importance of making the distinctions between these two sources and the need to incorporate the resulting effects in the design of pile foundations is the topic of investigation of this paper.

In essence, enormous computation considering the characteristics of the building and the ground condition parametrically is necessary to draw general conclusions regarding the soil-pile-structure interaction during earthquake shaking. But there are still severe computer capacity limitations when it comes to such enormous computational effort. The present investigation is limited to a 35 storied reinforced concrete (RC) building having one basement floor. A trial design of the building is attempted by considering three simple variations in the ground condition. The ground conditions considered are: (a) homogeneous stiff soil condition (a-soil), (b) homogeneous soft soil condition (b-soil), and (c) two layered ground with upper soft and lower stiff layers (c-soil).

Typically 20 to 45 stories, RC buildings are common as condominiums in Japan. Natural period T_B[1] of these buildings range from 1.2s to 2.5s, the period range where spectral velocity ordinates of input motion are generally uniform. Generally, the buildings are supported on cast-in-place concrete piles with enlarged base, and the pile diameter is controlled by base shear action rather than by the axial force, resulting in similar pile diameter. In other words, these buildings are fairly close in terms of their dynamic model, and a 35 storied building is a representative case.

Method of Analysis

The analysis consists of three phases. The first phase involves estimation of the modal displacements and internal forces from elastic eigenvalue analysis. Rationality of resolving the main modes of the total system into the building part and the foundation part is confirmed from modal displacements. The total lateral ground reaction, consisting of the earth pressure on the basement wall and the subgrade reaction of soil adjacent to the piles, is termed as the *subgrade reaction*. The equilibrium relation between base shear, inertia of basement, subgrade reaction and shear force at pile head is investigated, providing a grasp of the load transfer mechanism depending on the soil types.

Second phase involves nonlinear response analysis considering the 2-D FEM model of 'total system' shown in Figure 1. The 1-D nonlinear analysis of the free-field was carried out for three input motions, and their relative characteristics were depicted by response spectra of surface motions.

Thirdly, the 2-D FEM analysis was carried out on the 'foundation system', obtained by removing the building from the 'total system'. The difference between the responses of the 'total system' and the 'foundation system' is referred to as the contribution of 'building system'. Thus the relative contributions of the two to the response of total system is quantitatively investigated. The correlation analysis shows that the total response considering the nonlinear behavior can be resolved into contributions of the building and the foundation, similar to the elastic case.

The Analytical Model

Analytical model of the system consisting of the building, pile foundation and the ground, utilized for the 2-dimensional FEM analysis, is shown in Figure 1. Major conditions assumed are as follows:

1. The width of the ground interacting with building is five times the building width. The FEM mesh size based on the shear wave velocity and the fundamental period of the ground.
2. The depth perpendicular to the paper to depict 3-D effect is not provided.
3. All the elements of soil-pile-building behave nonlinearly except for the base layer.

[1]According to Japanese code $T_B = 0.02H_B$ seconds, where H_B is the building height in meters

4. The viscous damping in the soil is of Raleigh type, consisting of 1% for both first and second modes. The viscous damping in the building is also 1% for first mode and is proportional to the initial stiffness. The internal damping in piles is negligible.
5. A 10cm gap exists under basement slab, such that building load is transmitted solely by the piles. (The assumption has significant implications to interpretation of the results.)
6. The input base layer is located at 50m below the ground. The lateral as well as the vertical boundaries are of viscous type.
7. The analytical algorithm is based on Newmark's β-method, and the integration interval is 400th of a second.
8. Dynamic characteristics and base shear capacity of building and the diameter of the pile are typical for tall RC buildings in Japan.

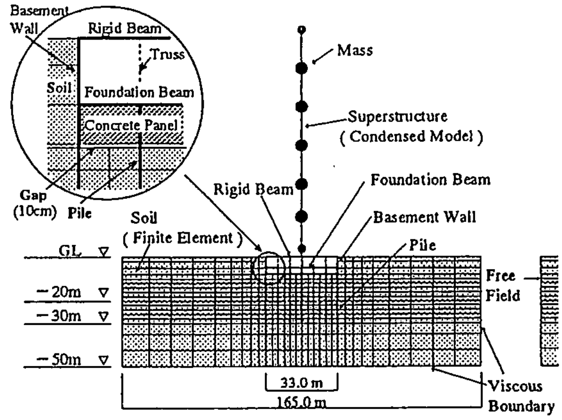


Figure 1: The 2-D FEM analytical model

Surficial Soil Conditions

The sandy gravel layer having SPT N-value greater than about 50 and lying at depths greater than 30m is a common bearing stratum in urban areas in Japan. The sandy gravel layer is often considered as the 'base layer' for the input of incident motion for

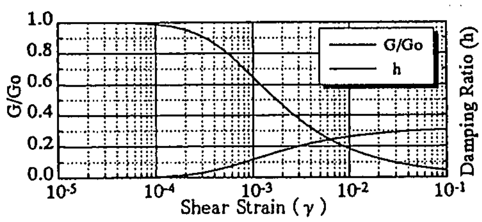


Figure 2: Ramberg-Osgood model for soil

Table 1: Details of the three soil profiles

Soil Type		a-soil	b-soil	c-soil
Soil Profile	GL-0m – GL-20m	sandy soil Vs = 250 m/sec ρ = 17.2 KN/m³	clayey soil Vs = 100 m/sec ρ = 15.7 KN/m³	clayey soil Vs = 100 m/sec ρ = 15.7 KN/m³
	GL-20m – GL-30m			sandy soil Vs = 250 m/sec ρ = 17.2 KN/m³
	GL-30m – GL-50m	sandy gravel Vs = 420 m/sec ρ = 20.6 KN/m³		
	GL-50m	viscous boundary		
Ground Period (T_G)	1st	0.49sec	1.20sec	0.85sec
	2nd	0.16sec	0.41sec	0.28sec

seismic response analysis. Accordingly, a sandy gravel base layer with shear wave velocity V_s 420m/s and density ρ $20.6kN/m^3$ is considered. Soil properties are shown in Table 1.The V_s of a-soil and b-soil are 250m/s and 100m/s respectively. The c-soil consists of clayey soil same as b-soil to a depth of 20m, underlain by sandy soil same as a-soil down to a depth of 30m below which lies the base layer. The density ρ and fundamental ground period T_G are also given in Table 1. Strain dependence of the shear modulus G and damping (h) of the soil was represented by the Ramberg-Osgood model shown in Figure 2. The standard shear strain $\gamma_{0.5}$, at which G becomes half the initial value G_0, is $1.5 \times 10^{-3} rad$ and the model parameters α and β are assumed to be 1.0 and 2.0 respectively for both sandy and clayey soils.

The Building Structure

The height H_B of the building is 103m, and the bottom of basement slab is 8m below the ground level and the elastic fundamental period T_B is about 2.0s. As shown in Figure 3, the building has a square plan with six 5.5m spans each way. The base shear capacity Q_y of the building is 63.99MN, which is 0.142 times the building weight. Figure 4 shows details of typical sections of beams and columns together with material specifications.

The building structure is represented by a bending-shear type stick model having floor masses approximately lumped at seven points along the height. The nonlinearity in shear resistance is considered, while the bending resistance is assumed to be elastic, as represented by the Takeda's hysteretic model (Takeda et.al., 1970) shown in Figure 5. The stiffness reduction coefficient is assumed to be 0.5. Dynamic properties and skeleton curve characteristics are given in Figure 6. The weight of basement portion is 21.8% of the weight of superstructure, which is 17.9% of the total building weight.

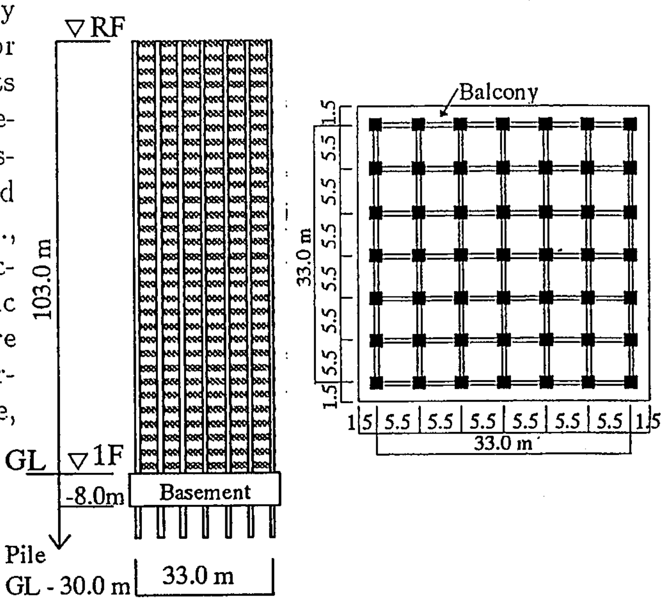


Figure 3: Plan and elevation of the building

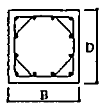
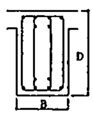
Member		Interior Column		Interior Beam		
Typical Section						
F	B×D (mm)	Bar	Hoop	B×D (mm)	upper bar lower bar	stirrup
30	800×800	12-D29	4-D13@100	500×800	4-D32	4-D13@150
25	800×800	12-D32	4-D13@100	500×800	4-D35	4-D13@150
12	850×850	12-D32	4-D16@100	550×850	4-D38+2-D32	4-D16@150
3	900×900	12-D35	4-D16@100	600×850	4-D38+2-D35	4-D16@120
2	900×900	12-D35	4-D16@100	600×950	4-D35+2-D32	4-D16@120
Material Specification		Reinforcement : D41~D35 SD390 D32~D10 SD345 Concrete : F_c30~ F_c45				

Figure 4: Typical RC sections of columns and beams

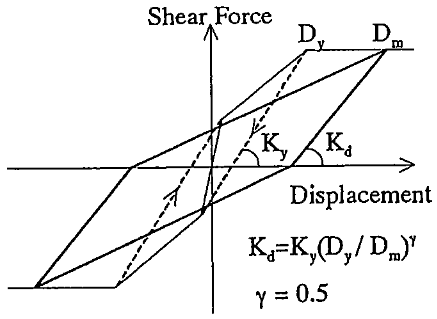


Figure 5: Takeda's hysteretic model

No	Height of mass GL+(m)	Weight (MN)	E I 10^{12} (MN-cm²)	G A 10^4 (MN)	α	Q_1 (MN)	Q_2 (MN)
1	103.2	36.1	0.955	2.12	0.117	9.2	15.9
2	85.8	72.3	0.955	2.12	0.226	8.9	31.7
3	68.4	74.8	1.182	3.20	0.231	11.4	43.6
4	50.7	77.4	1.182	3.20	0.269	11.5	52.9
5	33.0	72.5	1.482	3.92	0.272	13.8	59.4
6	18.3	67.6	1.482	3.92	0.277	14.1	62.8
7	3.5	41.1	1.482	6.41	0.208	18.0	64.0

EI : Flexural Stiffness
GA : Shear Stiffness

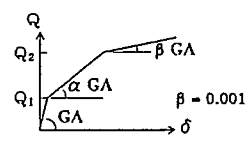


Figure 6: Dynamic characteristics of the structure

Piles Supporting the Building

Figure 7 shows the representative cross-section of the cast-in-place concrete pile (drilled shaft) of 2.0m diameter. The end bearing layer for the pile is 30m below the ground level. The concrete strength F_c is $30MPa$, and there are $40 - \phi 29$ rebars of SD345 grade, such that the steel area percentage P_g is about 0.82%. The area of rebar in the cast-in-place concrete piles is larger than 0.4% stipulated by the Japanese RC structure design code.

Figure 8 shows the relation between bending moment and curvature of piles under the action of the long term axial force. The bending resistance at cracking M_c is 6.93MN-m and the yield level bending resistance M_y is 14.08MN-m. The ultimate curvature of the section is5.6×10^{-5} (1/cm), which is made to correspond to a maximum bending compression strain of 0.3%. The restriction in strain stems from anticipation of low bending ductility in piles, whose shearing resistance cannot be strengthened sufficiently. Considering the pile head to be fixed, the maximum bending resistance computed by the subgrade reaction analysis is 6.25MN-m in a-soil, and 8.84MN-m in b and c-soils. The coefficients of horizontal subgrade reaction K_h is 196kPa for a-soil, and 49kPa for b and c-soils. The horizontal movement at pile head is 0.71cm in a-soil and 2.02cm in b and c-soils.

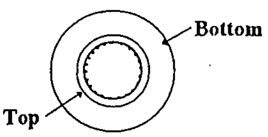
Typical Section		
Diameter of top		2000mm
Diameter of bottom		3000mm
Main Bar	upper part	40-D29
	lower part	32-D29
Hoop	upper part	O-D16 @150
	lower part	O-D16 @200
Material Specification	Reinforcement : SD345 Concrete : F_C30	

Figure 7: Typical cross-section of piles supporting the building

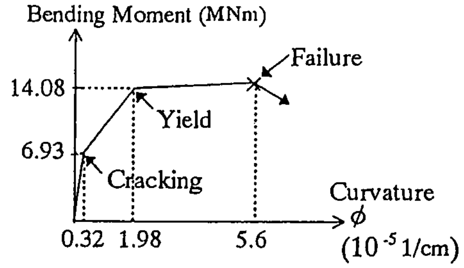


Figure 8: Bending moment resistance and curvature relationship for piles

Base Input Motion for the Analysis

Three input motions were considered for comparison to develop a grasp of their relative strengths. One of them is the well known El Centro NS record that is also recommended by the Building Center of Japan for the evaluation of the seismic resistance of tall buildings. The remaining two are Hyogoken-Nambu earthquake record of the Kobe Marine Observatory (Kobe JMA NS) and the artificial motion (Center L-2) recommended by the building center of Japan (BCJ, 1992). The El Centro motion is scaled to a maximum velocity of 50cm/s. The three input motions are given in Table 2. It is seen that three of the input motions are comparable except for the Kobe JMA NS which gives distinctly large spectral ordinates in the period range 0.3 to 3.0 seconds. Two of the input motions, El Centro NS and Kobe JMA NS, are used for the 2-D FEM analysis.

Table 2: Input earthquake motions

No.	Earthquake input motion	Duration (sec.)	Peak Acc. (cm/s^2)
1	El Centro NS	15.0	510.8
2	Kobe JMA NS	15.0	818.0
3	Center L-2	120	355.7

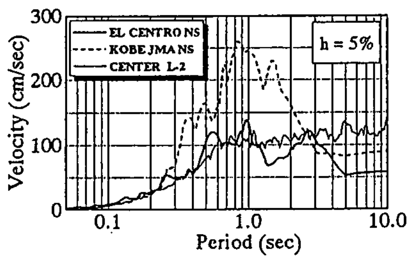


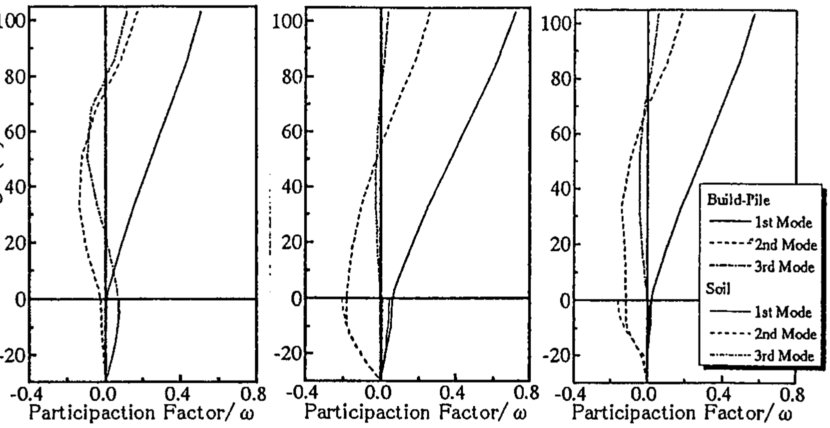
Figure 9: Response spectra of input motions

Elastic Eigenvalue Analysis of the System

Initially, elastic eigenvalue analysis was carried out on the 2-D FEM model of the soil-pile-building system. The results of the analysis were utilized to investigate the modal characteristics of the first three modes of shaking of the building structure in relation to the ground, depending on the type of surficial soil condition.

Figure 10 compares the modal displacements of the structure and pile with those of the ground.

The first mode of the total system is the first mode of the building structure in all the three soil types. In a-soil the first mode period of 2.05s of the total system is practically same as the building period of 2.0s considering fixity at base. Similarly, the third mode of the total system is the first mode of the ground, where the period of 0.46s is slightly shorter than T_G of 0.49s. The small difference in period indicates insignificant increase in the soil stiffness due to piles. In b-soil the first mode period of total system is 2.16s, about 10% longer than the fixed base period, while the second mode of the total system with a period of 1.07s corresponds to the first mode of the ground. From the free field fundamental ground period of 1.2s for b-soil, the ground stiffness seems to have increased by about a quarter due to the presence of piles.



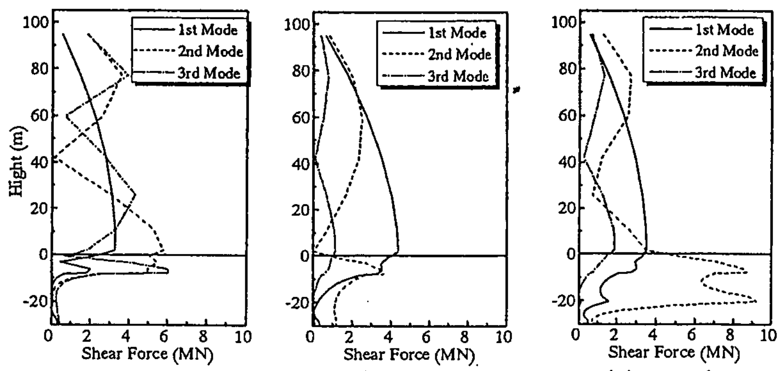
(a) a-soil (b) b-soil (c) c-soil

Figure 10: Comparison of first few displacement mode shapes

Figure 10(c) shows displacement mode shapes for c-soil, where the first mode period of the total system is 2.11s. Similar to b-soil, the second mode with period 0.8s, corresponds to the first mode of the ground. The stiffness of the soil is increased by about 16% in this case due to piles. While the difference in modal displacement between the free field ground and pile is clearly seen in b-soil, it is even more prominent in c-soil, showing a sudden change of slope at the two layer interface.

The modal response forces were derived from the modal displacement obtained by dividing the modal participation factors by the circular frequency. Figure 11 shows the distribution of modal shear force on the building. Similarly, Figure 12 and 13 show the distribution of the bending moment and the shear force respectively in the pile located at the center of the building.

In a-soil, when the shear force at pile head is set to 1.0, the inertia of basement is negligible at -0.009, while the base shear and subgrade reaction are -1.79 and 0.79 respectively in case of the first mode. The minus sign indicates that the subgrade reaction is opposite to the base shear, resulting in the reduction of the shear force at pile head. In case of the third mode in a-soil, when the shear force at pile head is set to be 1.0, the base shear becomes 0.31, the inertia of basement -0.68 and the subgrade reaction -0.63. In this case the inertia of basement and the subgrade reaction are acting on the same direction, resulting in a relatively smaller base shear.

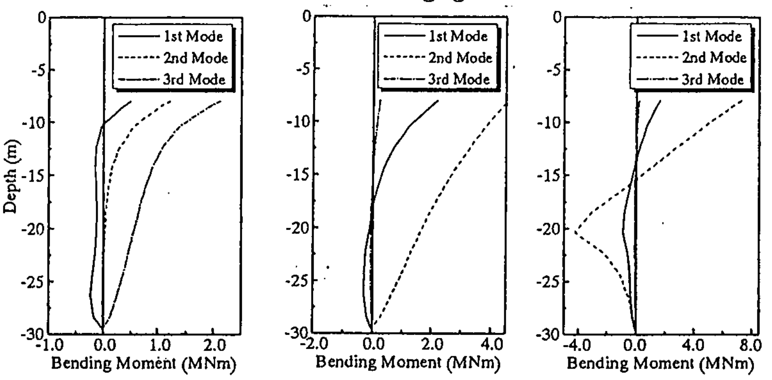


(a) a-soil (b) b-soil (c) c-soil

Figure 11: Distribution of modal shear in building and piles

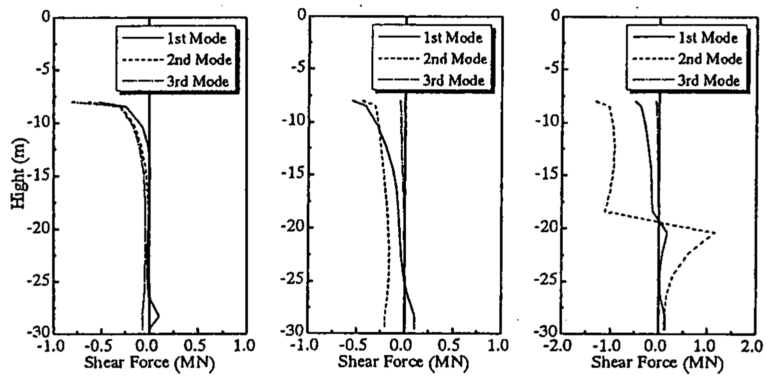
In b-soil, when the shear force at pile head is set to 1.0, the inertia of basement is -0.05, while the base shear and the subgrade reaction are -1.35 and 0.41 respectively in case of the first mode. The ground stiffness being small compared to a-soil, subgrade reaction is also relatively small. In case of the second mode, when the shear force at pile head is set to be 1.0, the inertia of basement is −0.47, the base shear is 0.01 and the subgrade reaction is −0.52. Here, the inertia and the subgrade reaction acting on the same direction has resulted in a negligible base shear.

In c-soil, when the shear force at pile head is set as 1.0, the inertia of basement works to be −0.03, base shear −1.30 and subgrade reaction 0.32 in case of the first mode. The subgrade reaction is smaller even compared to b-soil, probably because larger force being transmitted to pile by lower stiffer layer. In second mode, when the shear force at pile head is set as 1.0, the inertia of basement is −0.26, base shear is −0.40 and subgrade reaction is −0.34. This is the worst of the three soil type cases where the base shear, inertia of basement and subgrade reaction are all acting in the same direction. As can be noted in Figure 12, the bending moment at the two layer interface is very large, indicating a unique behavior of c-soil.



(a) a-soil (b) b-soil (c) c-soil

Figure 12: Modal bending moment distribution in central pile

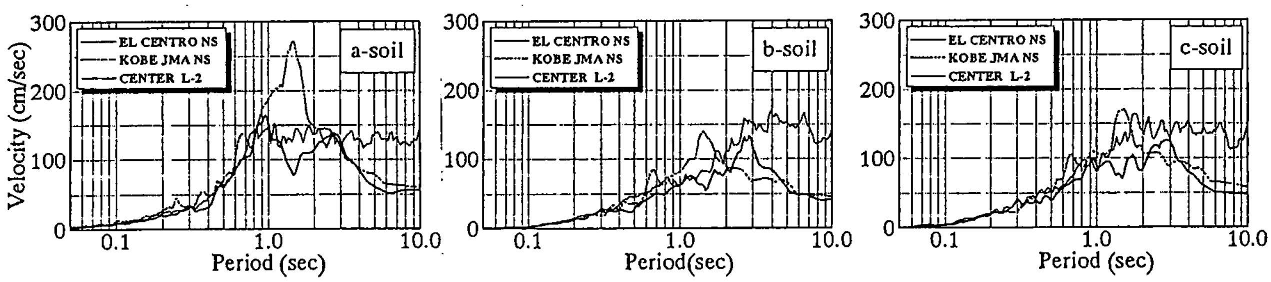


(a) a-soil (b) b-soil (c) c-soil

Figure 13: Modal shear force distribution in central pile

Response Analysis of Free Field Ground

The response of the free field ground was computed for a 1-D model using the input motions of Table 2. The response spectra of the surface response motion are compared in Figure 14. The Kobe JMA input motion shows large spectral ordinates in the period range 1-2s, particularly in case of a-soil. The Center L-2 input motion includes relatively large long period components and as a result the spectral ordinates are seen to remain practically uniform even beyond periods of 3.0s. El Centro NS is about intermediate between the other two.



(a) a-soil (b) b-soil (c) c-soil

Figure 14: Velocity response spectra of the computed surface response motions (h=5%)

The predominant period of shaking T_P was evaluated from the ratio of the smoothed Fourier spectra of surface response motion to that of the base input motion as shown in Figure 15. For a given soil condition T_P becomes increasingly longer than T_G as the level of excitation increases (Karkee, Sugimura and Tobita 1992) indicating increased nonlinearity in ground shaking. The extent of the period elongation depends on the soil type. Figure 16 shows the plot of T_P as a function of T_G. Kobe JMA input motion exhibits clearly larger T_P in all three soils indicating higher level of ground shaking. The guideline published by the Building Center of Japan (BCJ, 1992) recommends that T_G should be assumed to elongate by 2.2 times during extreme ground shaking.

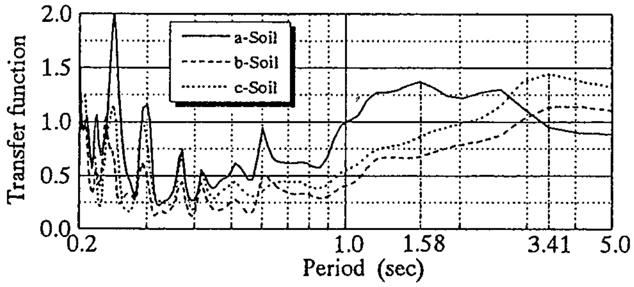


Figure 15: Ratio of smoothed Fourier spectra (Kobe JMA NS)

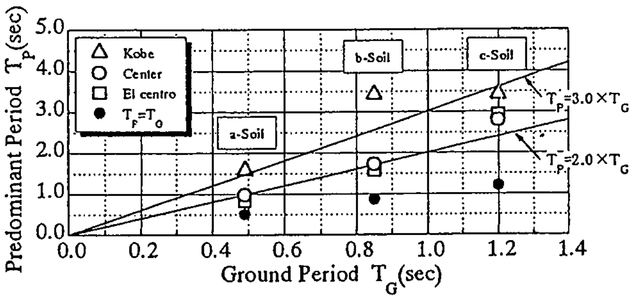


Figure 16: Elongation of ground period due to nonlinearity in ground shaking

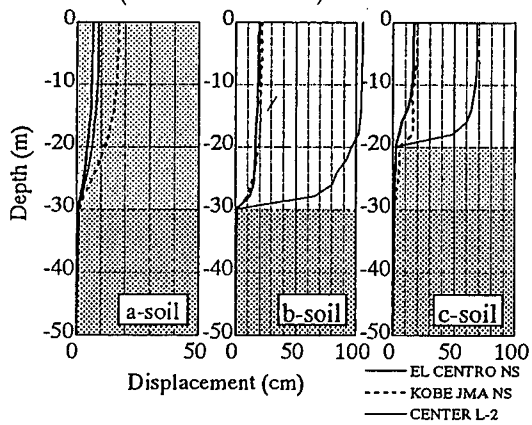


Figure 17: Maximum relative displacement

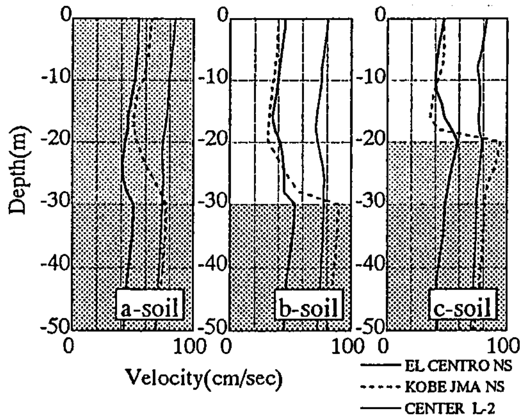


Figure 18: Maximum response velocity

Figure 17 shows the maximum response displacement of the free-field relative to the base layer. The maximum surface displacement relative to the base for a, b, and c-soils are 17.3cm (Kobe JMA), 105cm (Center L-2) and 68.5cm (Center L-2). Sharp rise in the maximum displacement can be noted near the pile toe in b-soil and at the interface of the two surface layers in c-soil, indicating stronger ground response effects at such locations. Figure 18 shows the maximum velocity distribution, where the maximum at surface are 84cm/s, 81cm/s and 83cm/s for a, b, and c-soils respectively, all for the Center L-2 input motion.

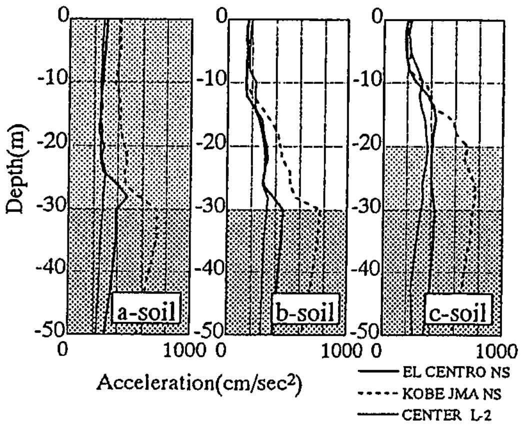


Figure 19: Maximum response acceleration

Similarly, Figure 19 the maximum acceleration distribution, where the values at surface are practically same for all the three input motions except in a-soil where Kobe JMA gives higher value of $415cm/s^2$. It is seen that the three input motions affect the ground response in different ways depending on whether we are interested in displacement, velocity or acceleration.

Seismic Response Analysis on 2-D FEM Model

The analysis on the 2-D FEM model was carried out for two of the input motions shown in Table 2: El Centro NS and Kobe JMA NS. In case of the El Centro NS input, the angle of maximum story drift was 1/107, 1/152 and 1/117 for a, b, and c-soils respectively. The corresponding maximum base shear was 64.60MN, 60.47MN and 60.18MN, indicating a base shear coefficient of 0.143, 0.133 and 0.133 respectively. All the base shear values are close to yield level, with the maximum value attained in case of a-soil. In case of the Kobe JMA NS input motion, the maximum story drift was 1/113, 1/213 and 1/118 for a, b, and c-soils respectively, and the corresponding maximum base shear was 56.78MN, 37.53MN and 54.72MN (base shear coefficients of 0.126, 0.083, and 0.121) respectively. From consideration of the maximum story drift response, it was seen that the superstructure has sufficient seismic resistance.

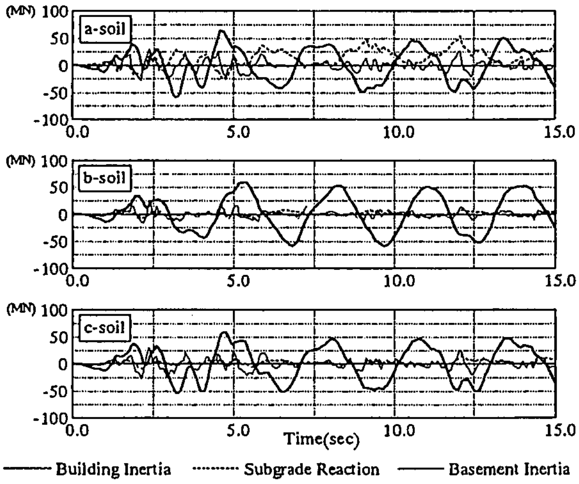


Figure 20: Time histories of the superstructure inertia, the basement inertia and the subgrade reaction (El Centro NS)

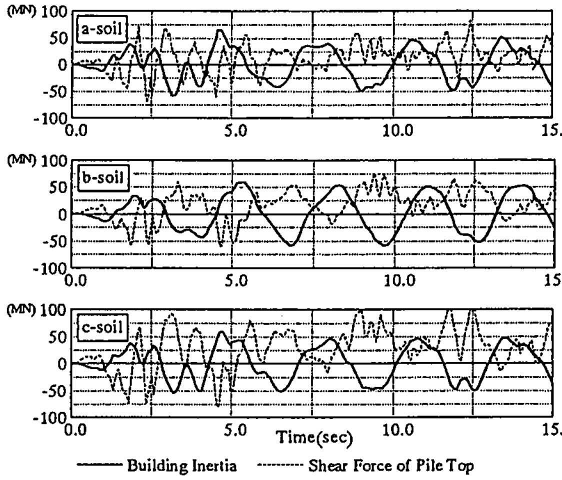


Figure 21: Time histories of the superstructure inertia and the shear force at pile head (El Centro NS)

Figures 20 and 21 shows the time histories of the inertia force and the forces due to soil response under the action of the El Centro NS input motion. From the computed base shear force time history it was noted that the predominant period of the building is about 2.9s in case of El Centro NS input, and 3.3s in case of Kobe JMA NS input irrespective of the type of soil. The elastic period of the fundamental mode being 2.1s, it was seen that the building period elongates by 1.4 and 1.6 times respectively, which is relatively smaller than the ground period elongation.

In Figure 20 the basement inertia has much shorter period of vibration than that of the superstructure, and the period of subgrade reaction is closer to that of the building inertia. Also, the basement inertia as well as the subgrade reaction is large in a-soil, while they are relatively small in soft soils (b and c-soils). Figure 21 shows that the shear force at pile head is strongly influenced by short period ground motion resulting in a time history having short as well as long period components, while the superstructure inertia consists primarily of long period components.

Figure 22 shows the maximum shear force response of the structure under the action of the two incident motions. In Japanese design practice, the ultimate horizontal force for which the piles are to be designed is specified as 1.5 times the sum of the base shear capacity of the building (62.48MN) and 0.15 times the weight of the basement portion (98.36MN), where 0.15 is the seismic coefficient for the basement. Thus the horizontal 'design shear force' works out to be 115.85MN.

The maximum shear force response at pile head for a, b, and c-soils are 86.38MN, 77.49MN, and 109.48MN respectively in case of the El Centro NS. The corresponding values are 137.07MN, 94.25MN and 132.76MN in case of Kobe JMA NS input motion. It should be noted that the maximum shear force response is larger at the two soil layer interface (GL-20m) than at the pile head in case of c-soil. The maximum shear force is seen to decrease from the pile toe and then increase again to the maximum value at pile head in both the a-soil and b-soil. Since the soil is assumed to be uniform over the pile embedment depth, the trend is attributable to the increased plasticity of soil near the pile toe. Of course this trend cannot be found in the elastic eigenvalue analysis discussed earlier.

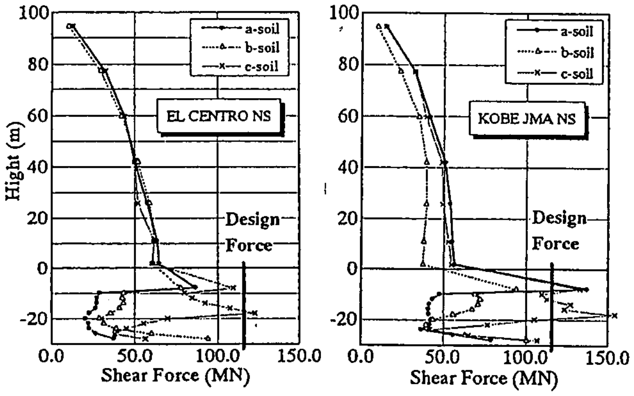


Figure 22: Maximum shear force response of the superstructure and the piles

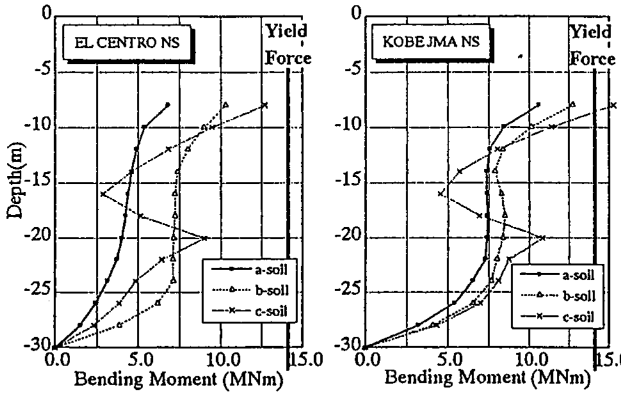


Figure 23: Maximum bending moment distribution in the piles

Figure 23 shows the maximum bending moment response of the piles, which can be noted to be strongly influenced by the soil layering. Large bending moment is seen at the two-layer interface (GL-20m) in c-soil, indicating the need to exercise caution in evaluating the bending moment at deeper portion of the pile rather than just the pile head. This can be of particular concern because the axial force in the pile at deeper portion would be smaller compared to axial load at pile head, because part of the axial load is borne by the shaft friction, resulting in smaller bending capacity of the concrete.

Building & Foundation Components of the Response

For the case of El Centro NS input motion, attempt was made to resolve the response of the total system into the building component and the foundation component and the nature and extent of contribution was investigated. For this the correlation of the inertial response of the structure with the forces from soil response was investigated. The coefficients of cross-correlation amongst building inertia, basement inertia, subgrade reaction and shear force at pile head are shown in Table 3. The correlation is not so clear in case of the total system, while it becomes more clearer when resolved into building and foundation systems.

Table 3: Coefficients of correlation between inertia forces and the forces from ground response

Total System ①	a-soil			b-soil			c-soil		
	(2)	(3)	(4)	(2)	(3)	(4)	(2)	(3)	(4)
(1)	-0.01	-0.74	-0.35	-0.03	-0.75	-0.64	-0.01	-0.26	-0.61
(2)	—	0.30	-0.84	—	0.26	-0.56	—	0.48	-0.58
(3)	—	—	0.15	—	—	0.28	—	—	-0.20
Foundation System ②	a-soil			b-soil			c-soil		
	(2)	(3)	(4)	(2)	(3)	(4)	(2)	(3)	(4)
(1)	-0.01	-0.03	0.01	-0.02	0.01	0.08	0.03	-0.02	-0.01
(2)	—	0.54	-0.85	—	0.55	-0.71	—	0.50	-0.73
(3)	—	—	-0.22	—	—	-0.61	—	—	-0.57
Building System ①−②	a-soil			b-soil			c-soil		
	(2)	(3)	(4)	(2)	(3)	(4)	(2)	(3)	(4)
(1)	0.29	-0.97	-0.92	0.42	-0.78	-0.99	0.11	-0.78	-0.98
(2)	—	-0.29	-0.32	—	-0.25	-0.45	—	-0.01	-0.29
(3)	—	—	0.86	—	—	0.81	—	—	0.73

Where, (1) Building Inertia (2) Basement Inertia
(3) Subgrade Reaction (4) Shear Force at Pile Head
Positive sign means two forces have same directions ,while negative sign means they are opposite to each other.

In the foundation system, the correlation of the superstructure inertia (represented by first floor shear force) does not seem to have any correlation with subgrade reaction, basement inertia and shear force at pile head, because the correlation coefficient of 0.01–0.08 in Table 3 can be considered to be practically zero. The correlation between basement inertia and shear force at pile head is high at 0.71–0.85 with the highest value in a-soil. However, the correlation coefficient between the basement inertia and the subgrade reaction is 0.50–0.54. Similar to what was observed in the elastic analysis, the first floor shear force and the subgrade reaction are opposite in direction, indicated by the minus sign in Table 3.

For the building system, correlation of the first floor shear force with the shear force at pile head, as well as with the subgrade reaction is high, with the coefficient of correlation ranging 0.92–0.98 and 0.78–0.97 respectively. Similar to what was observed in the elastic eigenvalue analysis, the subgrade reaction can be noted to be opposite to the base shear.

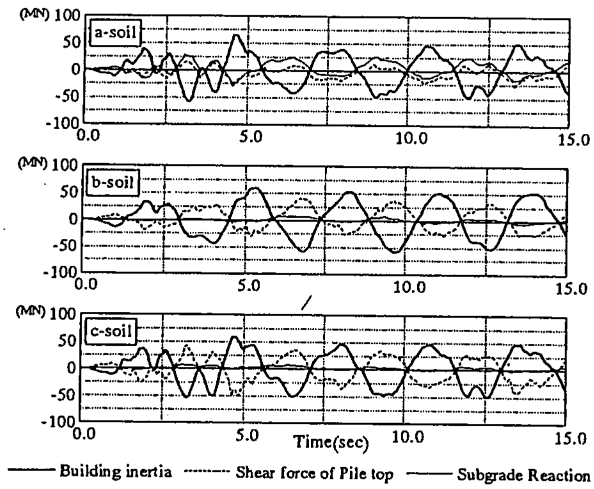


Figure 24: Response of the building system

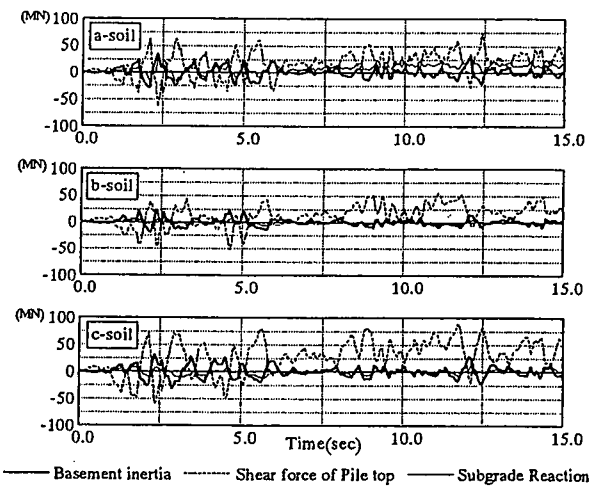


Figure 25: Time histories of foundation system

Figure 24 shows the time histories of first floor shear force, shear force at pile head and subgrade reaction for the building system. similarly, Figure 25 shows the time histories of the basement inertia, shear force at pile head and subgrade reaction of the foundation system. It is seen from the correlation analysis that, although there is significant nonlinearity in soil and building due to severe shaking during extreme seismic events, it is all the same possible to decompose the total system into main modes of vibration of the superstructure part and the foundation (soil and pile) part, similar in principle to the elastic analysis.

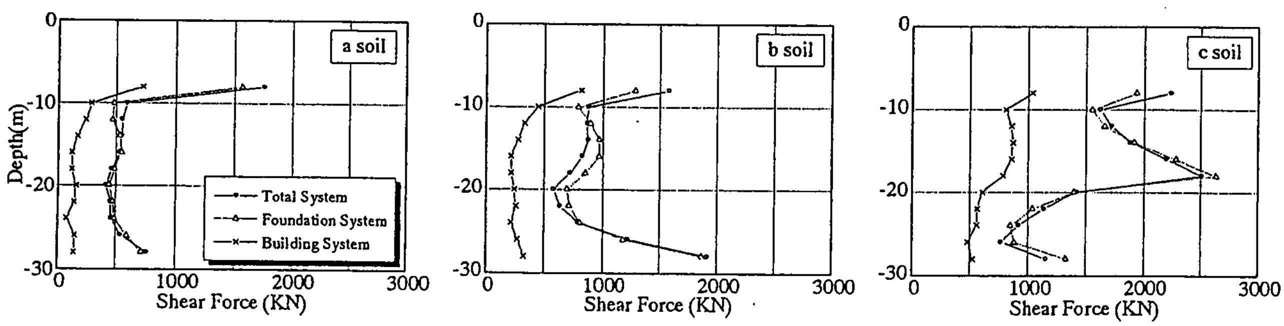


Figure 26: Maximum shear force responses of pile for the total and the component systems

Figure 26 shows the maximum shear force in pile along the depth. If the maximum shear force at pile head for the total system is normalized to be 1.0, that for the foundation system works out to be 0.89, 0.81 and 0.87 for a, b, and c-soils respectively. At deeper portion, the maximum shear force response for the total system and the foundation system are seen to be practically coincident. The result shows that the shear force acting along the pile length during earthquake shaking can be completely represented by the response of the foundation portion for all practical purposes.

If the inertial response of building has no correlation with ground response to earthquake shaking, and if the maximum value of one is smaller than that of the other, the one with smaller value tends to have little effect on the response of the total system. For example, if the maximum value of the ground response is 1.0 and that of the building 0.5, the maximum of the total response, considering the two to be mutually uncorrelated, may be given by the square root of the sum of the squares ($\sqrt{1.0^2 + 0.5^2}$) as 1.12, which is only slightly greater than 1.0. In this manner, the maximum shear force at pile head for the building system is 0.41, 0.53 and 0.47 for a, b, and c-soils respectively relative to the total system. From the above rule we get 0.98, 0.97, and 0.99 (1.30, 1.34 and 1.34 by simple summation)[2] respectively for the foundation system, which is close to 1.0, indicating domination of the ground response.

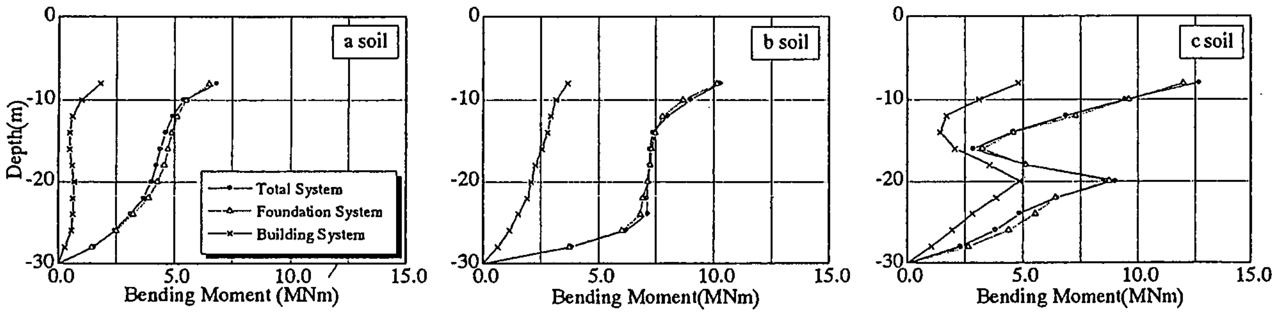


Figure 27: Maximum bending moment responses of pile for total and component systems

Figure 27 shows the maximum bending moment in pile along the depth. Considering the maximum bending moment at pile head for the total system to be 1.0, that for the foundation system works out to be 0.95, 0.98 and 0.94 for a, b, and c-soils respectively. Similar to the maximum shear force distribution, the maximum bending moment distribution for the total system and for the foundation system are practically coincident. The result may be expected, because bending distribution is obtained from integration of the shear force distribution.

Proposed Simple Design Approach

The current practice in Japan uses allowable stress method to design piles supporting buildings, in which the inertia of the building and the basement is to be resisted by the subgrade reaction, the forces exerted on pile due to soil response being completely neglected (BCJ, 1984). Noting the importance of ground response, a design approach based on the use of design spectra is proposed.

In Japan the response spectra of the different levels of base input motions are recommended by the Building Center of Japan (BCJ, 1992). If needed, the designer can estimate the response spectrum of the surface motion using suitable free field response analysis. The peak acceleration and peak displacement distribution in the structure can be determined from the design response spectra and the eigenvalue analysis. The design involves the following steps:

(1) **Superstructure inertia** is accounted for by estimating the base shear for the 'building system', from which the forces exerted on the pile is evaluated. This is done by the equivalent linearization method that is in current practice. The maximum value of the base shear should not exceed the base shear capacity of the structure. The ground provides the horizontal resistance.

[2]Values for foundation system are 0.89, 0.81 and 0.87 respectively for a, b and c-soils as given in page 11.

(2) **Basement inertia** and the subgrade reaction is evaluated from the 'foundation system', from which the forces exerted on the pile are estimated. Although both the basement inertia and the subgrade reaction are actions on the pile, limit on the maximum ground resistance can formulated.

(3) **Modal superposition** of the vibration modes of the building and the ground based on the ratio of T_B to T_G (referred to as the period ratio) can be utilized to arrive at the response of total system. The concept is illustrated in Figure 28. The superposition may be a simple sum or a weighted sum. The weight for the weighted sum may be estimated based on the ratio of the shear force at pile head or the pile head bending moment for the building system to those of the foundation system.

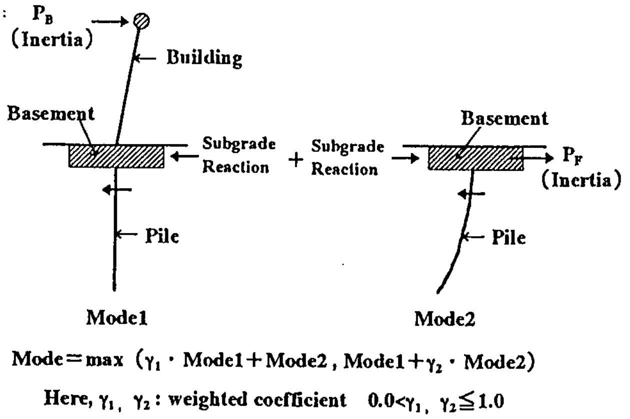


Figure 28: Illustration of superposition concept

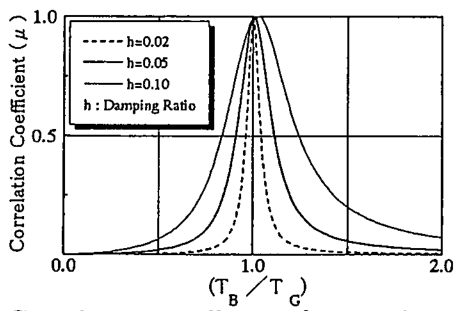


Figure 29: Correlation coefficient & period ratio

Figure 29 shows the typical variation of correlation coefficient (μ) with the period ratio (T_B/T_G) for different damping ratios (h). When the period ratio is around 1.0 (for instance, $0.65 < T_B/T_G < 1.5$) the superposition may be based on simple summation. For a period ratio greater than 1.5, it should be based on weighted summation, considering the need for a probabilistic modal superposition method owing to the period elongation of the building and the ground during extreme earthquake shaking. As for the case of period ratio less than 0.65, the first modes of the building and the ground are expected to be in phase and a simple summation method would be suitable. However, longer T_G than T_B would effectively result in decreased inertia of the building. This effect is considered to be reflected in the design spectra.

(4) **The proposed analytical model** of the foundation system is shown schematically in Figure 30 for the case of an ideal case consisting of a homogeneous soil deposit. The model is suitable for evaluation of forces and displacements in the pile considering nonlinearity. Strength as well as ductility aspects are vital in pile design, and the analytical model proposed is capable of accounting for both the aspects.

(5) **The ground response effect** under the action of the earthquake shaking may be evaluated by a simple equivalent static method. The bending moment acting on the pile M_B due to the inertia of the building mass may be given by Equation 1. The subgrade reaction on the basement wall P_S can be obtained from earth pressure analysis and the coefficient of subgrade reaction K_h is given by Equation 2.

$$M_B = \frac{P_B}{2\beta} \quad (1)$$

$$\beta = \sqrt[4]{\frac{K_h B}{4E_P I_P}} \quad (2)$$

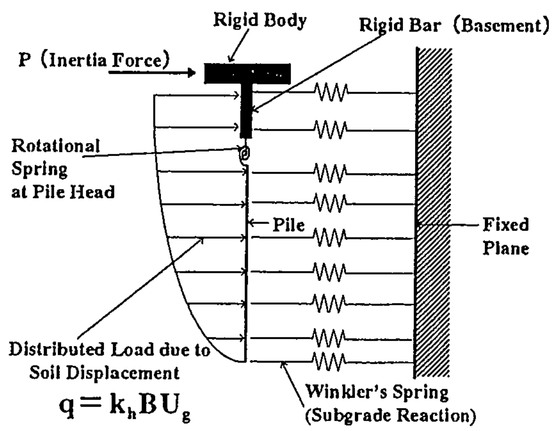


Figure 30: Analytical model for pile design

Here, P_B is the difference between building inertia force and P_S, B is pile diameter, E_P is pile Young's modulus, and I_P is moment of inertia of the pile cross-section. General solution based on the analytical model proposed in Figure 30 may be utilized for the analysis and design of piles.

A simple solution for the bending moment M_S due to the ground displacement, considering the ground to be homogeneous, may be obtained as a special case from Equation 3 (Hasegawa and Nakai, 1992). For the evaluation of the basement inertia (P_F) and the P_S, the pile and soil may be assumed to move together and computed solely from the difference in horizontal displacement between the basement and the adjacent soil. This way P_S may be given by Equations 7. P_F may be obtained from the acceleration response spectral ordinate a_F at period zero, which is in fact the peak acceleration of input motion.

$$M_S = \left(\frac{\pi}{2}\right)^2 E_P \left(\frac{I_P}{H}\right)\left(\frac{U_P}{H}\right) \tag{3}$$

$$U_g = a_F \left(\frac{T_P}{2\pi}\right)^2 \tag{4}$$

$$T_P = 2.2 \times T_G \tag{5}$$

$$U_P = \frac{U_g}{1 + \frac{1}{64}\left(\frac{\pi}{\beta H}\right)^4} \tag{6}$$

$$P_S = K_h L H_o \times U_g \left[1.0 - \cos\left(\frac{\pi H_o}{2(H + H_o)}\right)\right] \tag{7}$$

$$P_F = W_F \times \left(\frac{a_F}{g}\right) \tag{8}$$

$$M_F = \frac{P_S + P_F}{2\beta} \tag{9}$$

Here, U_P is horizontal displacement of pile head, H is pile length, U_g is horizontal displacement of free-field, L is width of building foundation, H_o is depth to pile head from the ground surface, W_F is weight of foundation portion, and g is acceleration due to gravity. Equation 5 indicates that T_G elongates by 2.2 times due to nonlinearity under extreme earthquake excitation (BCJ, 1992). If the ground is homogeneous, the above solutions can be utilized to evaluate the comparative extent of building inertia and ground movement affecting the design of piles. Parameters βH, B/H and U_P/H are of particular importance.

Conclusions

Results of elastic eigenvalue analysis indicate the diminance of the fundamental modes of the building and the ground in pile design. In fundamental mode of the building, the subgrade reaction is opposite to the inertia of the building, and the basement inertia is negligible in all the three soil types considered in this investigation. On the other hand, the basement inertia and subgrade reaction are in the same direction in fundamental mode of the ground. Inertia of building is not negligible but acts opposite to the shear force at pile head in a-soil, while it is practically negligible in b-soil giving rise to easy resolution into building and foundation systems. Similar to a-soil, the base shear is not negligible in the fundamental mode of c-soil, and in addition, the building inertia, basement inertia and subgrade reaction all act together to give a worst force combination for piles.

From the 2-D FEM analysis, it was seen that the maximum base shear response during extreme shaking reaches the ultimate capacity of the structure. The Japanese design code criteria for the maximum story drift (1 in 100), however, is adequately met. Overall, the structure is considered to have adequate seismic capacity. Similar the results of eigenvalue analysis, basement inertia and subgrade reaction add up to a large shear force at pile head response in c-soil. In addition, the maximum shear force occurs at the interface of the two soil layers, indicating that piles for tall RC buildings in layered ground condition require special precaution in design. The pile head bending moment increases from a to b to c-soils in that order. The pile bending moment capacity exceeds in case of the Kobe JMA NS input, but there was leeway in sectional radius of curvature.

The correlation among the time histories of external forces such as inertia of building and basement, and those of response forces such as subgrade reaction and shear force at pile head, was analyzed. The results confirm that the response of the total soil-pile-structure system can be resolved into building and foundation components, similar to the elastic analysis. The response forces acting on the pile based on foundation system is practically same as that based on total system, irrespective of the soil types, indicating significance of ground response effects.

Considering the results of the investigation reported here, a simple design approach for the design of piles is proposed. The proposed design approach is expected to be applicable to practical design applications.

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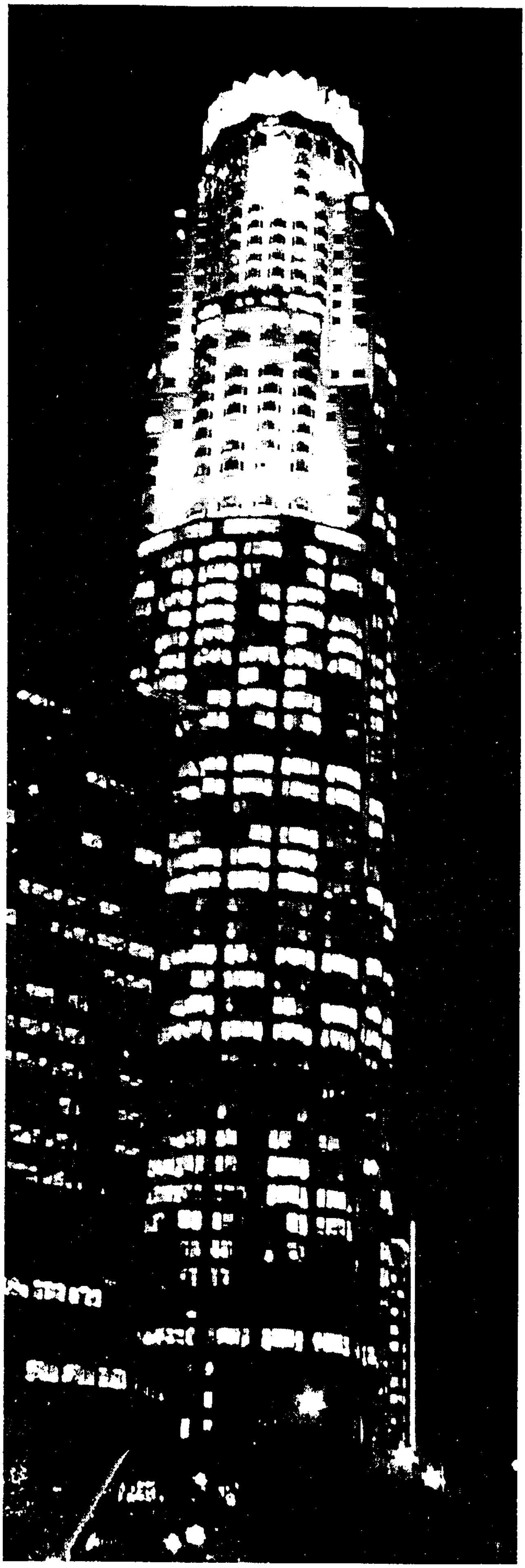

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