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A STUDY ON SEISMIC RESPONSE EVALUATION BY MONOLITHIC ANALYSIS OF BUILDING AND ITS FOUNDATION

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SUMMARY

In order more exactly to verify structure safety of the whole building including a foundation structure, the monolithic analysis model to have unified superstructure, ground, and pile foundation was proposed. A static nonlinear push over analysis was done by means of a monolithic analysis model. Then, force - displacement relationship, rigidity, and strength was compared with a conventional foundation fixing-model. Force - displacement in relation to, a growth of a natural period accompanied with a deformation of a pile, and a local non-linearity of a ground in the periphery of a pile is included in it by a monolithic analysis model. Consequently, a difference was seen by a modified performance of a frame and collapse mode with a foundation fixing-model and monolithic analysis model. Force - displacement relation was applied to an equivalence single-degree-of freedom system model including a foundation that it calculated with a monolithic analysis model. By this, a method to estimate the response point to have considered a dynamic interaction effect at seismic time was proposed. In addition, a response point by a capacity spectrum method was compared as enforcing a time history seismic response analysis by a sway-rocking model. A this result, response point by a monolithic analysis model became a safely neighboring evaluation.

INTRODUCTION

For this paper, an estimation method of a new seismic response by a monolithic analysis was presented. Further, an effect of a monolithic analysis by an exercise was shown. In the course of a conventional foundation fixing model design where the superstructure, ground and foundation are separated, it is difficult to grasp correctly the interval stress transmit among the superstructure and foundation structures, as well as the crack of materials and the occurrence state of yield hinge. On the contrary with a monolithic analysis model, it can verify stress transmit at a joint of every material of superstructure, ground, pile foundation and every material against that. By making a static push over analysis with a monolithic analysis model, in order to be able to obtain exact force - displacement relation, it became

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more exact static design is possible than before. It dose not only becomes a useful static design for a result of this monolithic analysis, but there is a merit to have a dynamic interaction effect at seismic time harvested by a design. To put it concretely, for the extension of a natural period accompanied with deformation of a foundation structure change of damping by a local nonlinearity of ground in the periphery of pile, further, it was evaluated. If a monolithic analysis is done, it can verify seismic performance of the whole of a superstructure and a foundation structure. Further, a set up of criteria and a rational design are possible.

Earthquake response analysis method according to the response spectrum method using the monolithic analysis model

Setting of an earthquake motion at ground surfaces:

Acceleration response spectrum at ground surfaces: $S_a(T)$shall have the following relationship by using the standard acceleration response spectrum in the released engineering base position. The spectrum has the following relationship and represented by expressions using $S_0(T)$:

$S_{ad}(T) = G_{sd}(T) \times Z \times S_{0d}(T)$ Expression (1)

$S_{as}(T) = G_{ss}(T) \times Z \times S_{0s}(T) \times F_h$ Expression (2)

where,

$S_{ad}(T)$, $S_{as}(T)$: Acceleration response spectrum on ground surfaces at damage limited design and at safety design

$G_{sd}(T)$, $G_{ss}(T)$: Acceleration amplification rate on surface-layer ground at damage limited design and at safety design

$S_{0d}(T)$, $S_{0s}(T)$: Standard acceleration response spectrum at damage limited design and at safety design

F_h :Acceleration reduction coefficient caused by attenuation at safety design

By shifting the surface-layer ground to an equivalent shearing type multi-mass model, the predominant period of surface ground and acceleration amplification rate: $G_s(T)$ considering ground non-linearity was calculated. The calculation procedures are shown in Figure 1. The primary and secondary predominant period of considering non-linearity of surface-layer ground at safety design for the surface-layer grounds

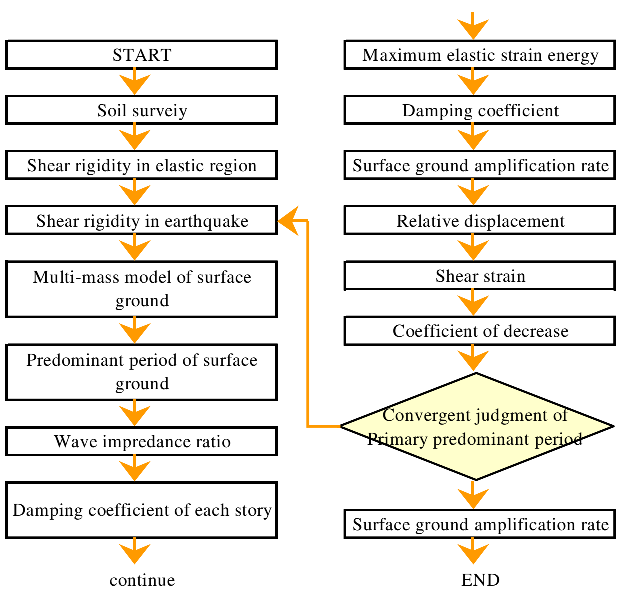


Figure1:Calculation procedure of surface ground amplification rate

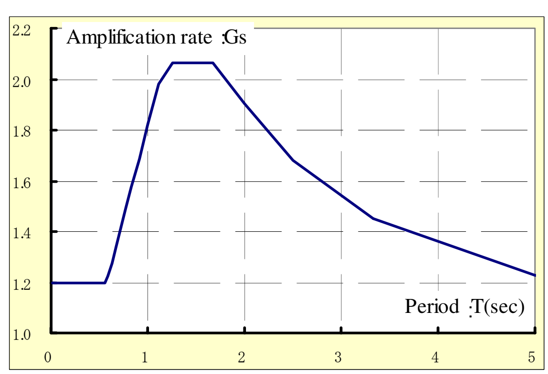


Figure2:Acceleration amplification rate to have nonlinear of surface ground Gs

were $T_1 = 1.46$sec,$T_2 = 0.49$sec. Also the amplification rates respective predominant period are: $G_{s1} = 2.07$, $G_{s2} = 0.94$. The amplification rate considering non-linearity of surface-layer ground at safety design: $G_{ss}(T)$ is shown in Figure 2.

Contraction towards equivalent single-degree-of-freedom (ESDOF) Building Model

As shown in Figure 3, by using the force - displacement relationship depending on the static nonlinear monotonically increasing load analysis of monolithic analysis model, the objective buildings will make contraction towards ESDOF unit model. When the spring constant in superstructures and the applicable spring constant of sway and rocking in the monolithic analysis model are defined as K_B, K_{sw}, K_{ro}, respectively, as a whole building, the spring constant: K_e.can be expressed as the sum of inverse number in the below expression. This is because that the displacement of the entire building is depending on the installation of the applicable springs arranged in series.

$$1 / K_e = 1 / K_B + 1 / K_{sw} + H^2 / K_{ro} \qquad \text{Expression (3)}$$

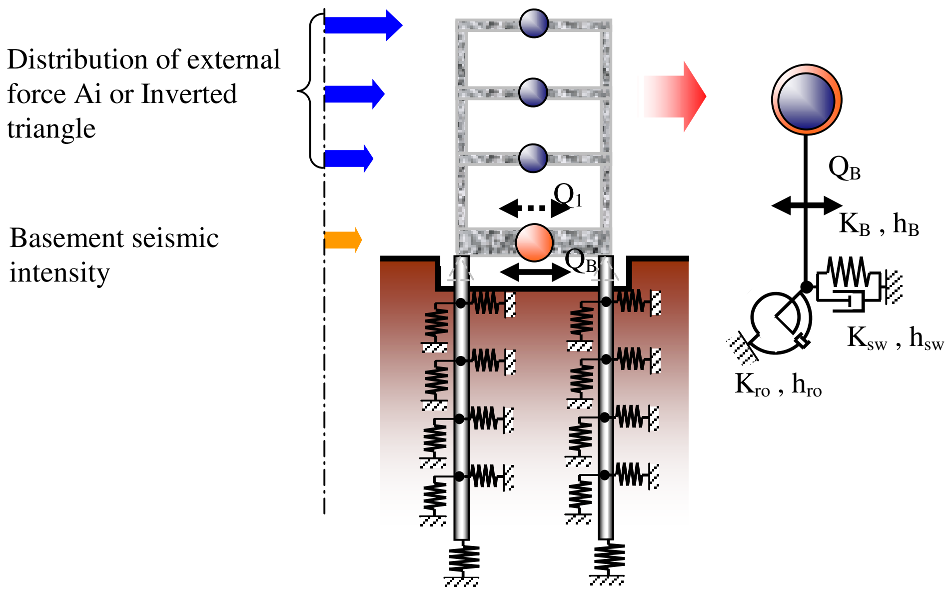


Figure3:Conversion to equivalent single-degree-of-freedom(ESDOF)system of monolithic model

As far as the sway springs and rocking springs of monolithic analysis model are concerned, they are able to calculate from the relationship between the shearing force and horizontal displacement in the pile head position, as well as from the relationship between the difference of directional displacement in the pile head axis in the compressed section and pulling section and the length of span in the rigid direction. :

$$K_B = Q_B / \delta_B , K_{sw} = Q_{sw} / \delta_{sw} , K_{ro} = Q_{ro} / \delta_{ro} \qquad \text{Expression (4)}$$

where,

δ_B: Horizontal displacement in superstructures

δ_{sw}: Horizontal displacement of monolithic analysis model in pile head position

δ_{ro}: Horizontal displacement of monolithic analysis model in pile head position depending on rocking angle : $\delta_{ro} = H \times \theta$, This is related with the following items as shown below :

Q: Shearing force in equivalent ESDOF unit contraction model : $Q = Q_B = Q_{sw} = Q_{ro}$

δ : Horizontal displacement in equivalent ESDOF unit contraction model : $\delta = \delta_B + \delta_{sw} + \delta_{ro}$

H: Typical height measured from the foundation bed in equivalent ESDOF unit contraction model θ : Turning angle generated by inversion moment : $\theta = ({}_{pc}\delta_z - {}_{pt}\delta_z) / L_y$

${}_{pc}\delta_z$,${}_{pt}\delta_z$: Pile head axis displacement in the compressed section and pulling section.

L_y: Length of span in rigid direction

Equivalent circular vibration in compound considering interaction: ω can be expressed by the below expression:

$$\omega_e^2 = (1 / \omega_B^2 + 1 / \omega_{sw}^2 + 1 / \omega_{ro}^2)^{-1} \qquad \text{Expression (5)}$$

where,

ω_B: means natural circular frequency against superstructure displacement: $\omega_B^2 = K_B / M_u$

ω_{sw}: means natural circular frequency against sway displacement: $\omega_{sw}^2 = K_{sw} / M_u$

ω_{ro}: means natural circular frequency against rocking displacement: $\omega_{ro}^2 = K_{ro} / (M_u H^2)$

M_u: means effective mass of a building including foundation

Calculation of Equivalent Viscous Dumping Constant.

The equivalent viscous dumping constant of sway and rocking depending on compound frequency was calculated by a cone model [1)] The cone-model-used shearing rigidity was specified as equivalent shearing rigidity obtained from the calculation of acceleration amplification rate of surface layer ground. In this case, however, as far as the radiation damping is concerned, it was determined to evaluate separated by using later described method.

The equivalent viscous dumping constant: h_{sw}' corresponding with the sway displacement of pile foundation may practically correspond with equivalent viscous dumping constant of direct foundation and pile foundation. Therefore, it was calculated as the direct foundation.

The equivalent viscous dumping constant: h_{ro}' corresponding with the rocking displacement of pile foundation was procured by multiplying 0.5 to the rotation equivalent viscous dumping constant calculated as the direct foundation [2)] .

The equivalent viscous dumping constant of composition : h_e was calculated by Expression (4) by using superstructure, sway and rocking spring constant according to the monolithic analysis model as shown below :

$$h_e = \Sigma h_j ' W_j / \Sigma W_j \qquad \text{Expression (6)}$$

where,

W_j: This is energy and $W_j = 1/2\ Q_j^2/K_j$

C_B, h_B' is the equivalent viscous dumping coefficient and equivalent viscous dumping constant of superstructure in composing equivalent circular frequency: ω_e.. And then, $h_B' = C_B / (2\ \omega_B M_u)$

C_{sw}, h_{sw}' is the equivalent viscous dumping coefficient and equivalent viscous dumping constant of sway at composing equivalent circular frequency: ω_e. And then, $h_{sw}' = C_{sw} / (2\ \omega_{sw} M_u)$

C_{ro}, h_{ro}' is the equivalent viscous dumping coefficient and equivalent viscous dumping constant of rocking in composing equivalent circular frequency: ω_e. And then , $h_{ro}' = C_{ro} / (2\ \omega_{ro} M_u H^2)$

The relationship between the sway displacement and rocking displacement can be described as a double rope rigidity as shown below;

$$\bar{K} = \bar{F} / \bar{U} = F / u\ e^{i\phi} = F / u (\cos \phi + i \sin \phi) = K + i K' \qquad \text{Expression (7)}$$

where,

$\bar{K}$: Dynamic impedance

$\bar{F}$: Dynamic external force

$\bar{U}$: Dynamic response displacement

F : Excitation amplitude

u : Displacement amplitude

ϕ : Phase difference of respond displacement against exciting force

i : Imaginary number

Real number part of dynamic impedance: K, and Imaginary number part: K' are respective ground spring constant and the constant relating to damping. The equivalent viscous dumping constant in the dynamic impedance represented by the double-rope rigidity may be expressed by using the following expression:

$h' = \sin(0.5 \tan^{-1}(K'/K))$ Expression (8)

If the imaginary number part of dynamic impedance is formed as proportional to the circular frequency: ω, then the equivalent viscous dumping constant may be expressed by using the following expression:

$C = K'/\omega$ Expression (9)

Accordingly, the dynamic impedance of sway and rocking may be expressed by using the following expressions:

$\bar{K}_{sw} = K_{sw} + i\,K'_{sw} = K_{sw}(1 + 2\,i\,h_{sw}') = K_{sw} + i\,C_{sw}\,\omega$ Expression (10)

$\bar{K}_{ro} = K_{ro} + i\,K'_{ro} = K_{ro}(1 + 2\,i\,h_{ro}') = K_{ro} + i\,C_{ro}\,\omega$ Expression (11)

where, h_{sw}', h_{ro}' are the equivalent viscous dumping constant against the sway displacement and rocking displacement counted from the double-rope rigidity in the respective composing frequency. For the imaginary number part of dynamic impedance (Constant relating to attenuation), it has features that the values are variable bordering the primary predominant frequency of ground if the surface-layer ground is provided with predominant frequency. Therefore, the imaginary number part the dynamic impedance was set up as shown in Figure 4.

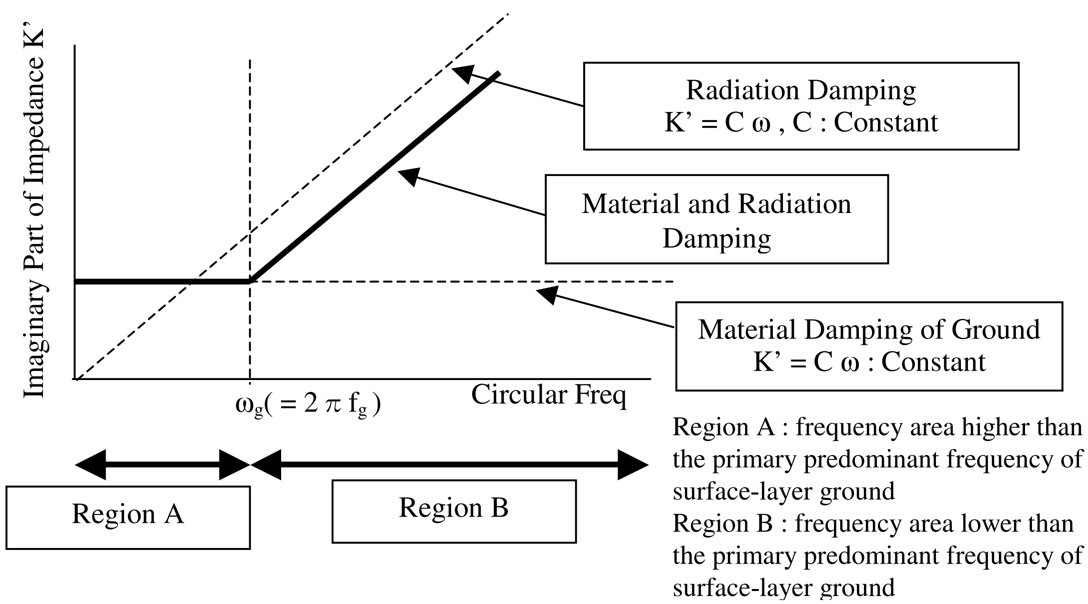


Figure4: Simplified Evaluation for Imaginary Part of Impedance

The imaginary number part the dynamic impedance at frequency area lower than the primary predominant frequency of surface-layer ground shall be a certain value. On the contrary, in the high frequency area higher than the primary predominant frequency of surface-layer ground, the imaginary number part the dynamic impedance will increase in proportion the frequency.

Setting of Limit Value:

Results obtained from the static non-linear increasing loading analysis in the monolithic analysis model, smaller value shall be defined as the limited value selecting from the below items.

1. Time reaching to the limit layer to layer deformation angle of superstructure in the monolithic analysis model.
2. Time reaching to the collapse mechanism of lower structure in the monolithic analysis model.

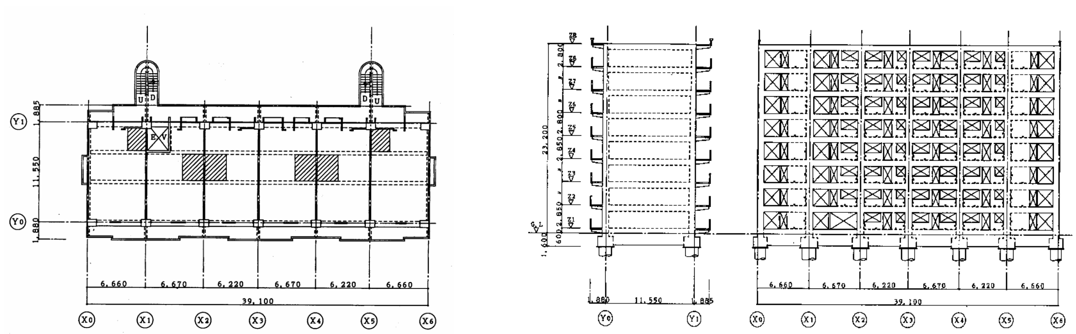
Examination on Results:

According to the response spectrum method, against the equivalent 1-freedom unit contraction model, the damage limited design considering the dynamic interaction, as well as the response vale in the safety design shall be calculated. Then make sure that these response values are less the limited value in the damage limited design and safety design,

Modeling of Monolithic Analysis

Superstructures (Buildings)

The study was made on a collective housing of reinforced concrete construction, with 8 stories above the ground as the target building of analysis. The building of 6-span rigid frame construction in the rigid direction and of 1-span multi-story bearing wall construction in the span direction. The outline of the building is shown in the following figure:



Typical Plan Framing (span direction) Framing (ridge direction)

Figure5: Outline of model building

Ground and Pile Foundation

The specifications of the ground and pile used for the analysis are as shown in Figure6 below. The natural period of the ground is 0.757 second. Moreover, the pile foundation is a cast-in-place concrete pile, and piles of which diameter is ϕ 1400-1800 are being used. The amount of bar arrangement in the pile head section is as follows: Ratio of reinforcement; 1.0-1.9%, in the middle section; 0.6-1.0%,and in the lower section; 0.4%.

Conditions of External Force

The external force distribution form of the superstructures in the static non-linear increasing load analysis according to the monolithic analysis model shall be defined as an Ai distribution. Moreover, any external force acting on the foundation structures (pile head section) was set so that the base shear in the superstructures will become: Foundation earthquake intensity: k = 0.1, where C_B = 0.2. On this occasion, as far as the ground deformation during earthquake is concerned, the external force shall not be considered.

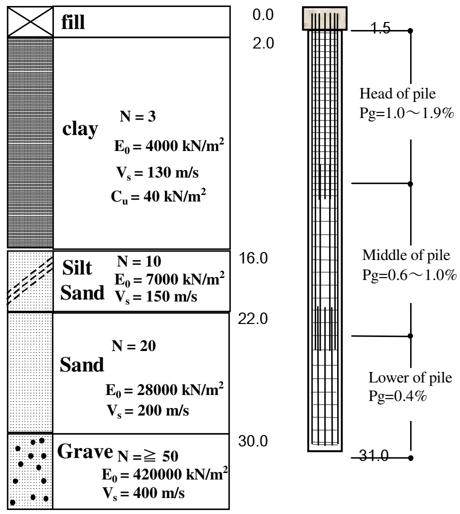


Figure6: Ground condition and model of pile

Structural Model

The static non-linear increasing load analysis was conducted in the respective ridge direction and rigid direction by using a solid frame model consisting of the one-piece superstructures and foundation structures. Figure7 shows the modeling of monolithic analysis.

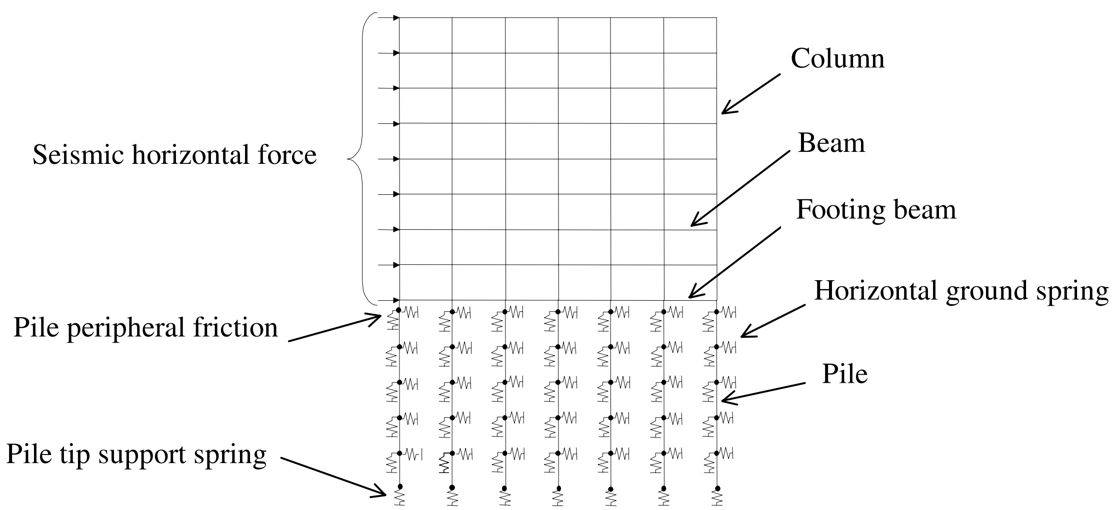


Figure7: Monolithic Model

Modeling of Structural Members

Modeling was made as follows:

- Column members were modeled in the column head and column leg section.
- Beam members were modeled in both ends as shown in Figure8
- In this case, the bending moment (M) and rotating angle (θ) make a model of Tri-linear type curve.

As far as the shear walls are concerned, as shown in Figure9, the applicable beams in each story were modeled as rigid beams and only modeled structural columns which can bear the axial tension only (column heads and column legs are supported by pins). In other words, wall members were modeled as virtual columns where the wall members can bear bending and shearing forces.

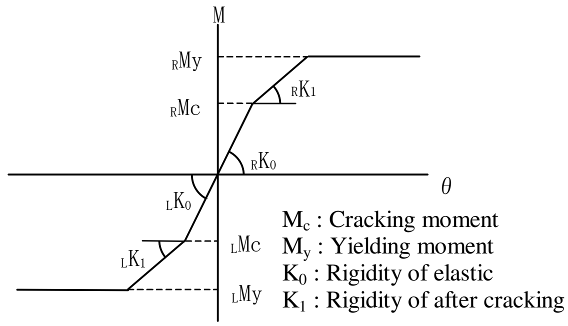


Figure8: Model of Beam

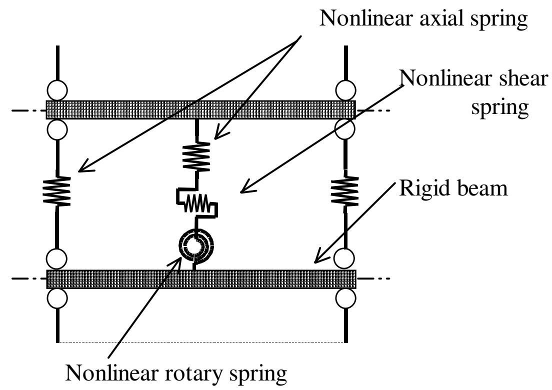


Figure9 Model of shear wall
(Kabeyasawa Model)

Modeling of Piles

Modeling was made as follows:

Pile members were modeled, as shown in Figure10.The relationship between the Bending moment (M) and Curvature (ϕ) (M- ϕ relation) made a model as Tri-linear type skeleton curve. In the course of modeling, the pile head axis force of superstructure required horizontal yield strength (at equivalent to required D_s) was used. Then based on the (M- ϕ relation) calculated from the cross-section analysis, the member characteristics of body structure was set up so as to obtain the expression: (Area -1) + (Area -3) = (Area -2). Moreover, the relationship between the Concrete stress-Strain was specified by using the Bi-linear type skeleton curve where, the material strength of reinforcement was specified 1.1 times larger than the design standard strength according to "e Function method". Moreover, the dissecting length of pile members was standardized as 1/2 of the pile diameter; 800 mm up to near GL-16m and 2000 mm up to near GL-16 to 22m, and then, the more deep section was specified as 4000 mm. In addition, the curvature distribution in the dissecting zone was specified constant. It was assumed that the pile and pile cap was jointed rigidly.

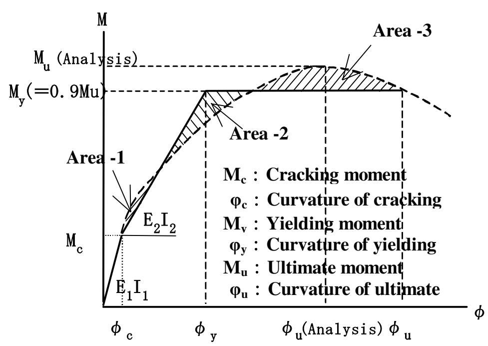


Figure10: Model of pile (M- ϕ relationship)

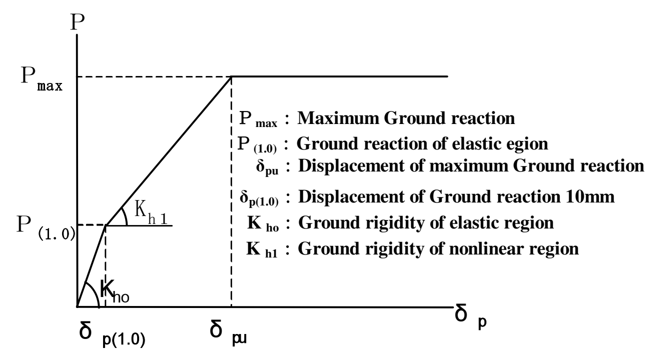


Figure11: Horizontal ground spring

Modeling of Ground around Piles

The horizontal resistance around piles was, as shown in Figure11 above, modeled as Tri-linear type skeleton curve where, the maximum ground reaction of "B.B.Broms" was specified as the upper limit value. In this case, however, the initial rigidity was set to "$K_{ho} = 0.63\ E_o\ D^{-3/4}$"[3]. Then the second rigidity was set to 1/3 of the initial rigidity. The friction resistance of pile peripheral was, as shown in Figure12, modeled as Bi-linear type skeleton curve where, the maximum peripheral friction force was specified as the upper limit value.

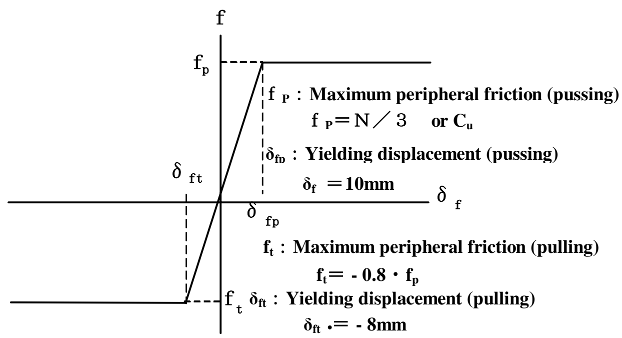


Figure12: Pile peripheral friction spring

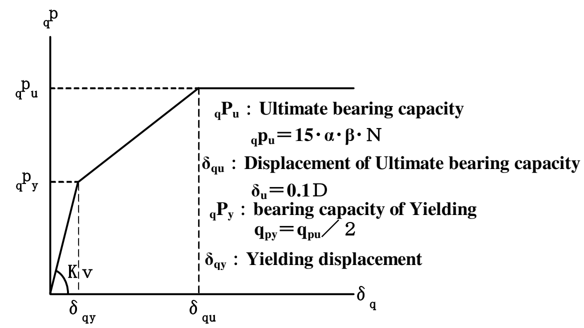


Figure13: Pile tip support spring

The resistance of the pile tip was, as shown in Figure13 above, modeled as Tri-linear type skeleton curve where, the maximum bearing force at the pile tip was specified as the upper limit value was specified as the upper limit value. But, the initial rigidity was specified as "$K_v = 2E_o/\{\pi D(1-\nu^2)/2\}$[4]", and the second rigidity used the values calculated from "${}_qP_u$, ${}_qP_y$, δ_{qu}, δ_{qy}".

External Force Conditions

The external force distribution form of superstructure in the analysis of static nonlinear monotonically increasing load analysis according to the monolithic analysis model was specified as "Ai distribution". Moreover, external forces acting on the foundation structure (in pile head section) were set up so that the superstructure base shear becomes as [Foundation earthquake intensity becomes: k = 0.1], where $C_B = 0.2$. On this occasion, as far as the ground deformation at earthquake is concerned, it shall not be considered.

Analytical Results of Static Nonlinear Monotonically Increasing Load Analysis according to the Monolithic Analysis Model

Ridge direction

Figure14 shows a Force - Displacement Curve in the ridge direction according to the Monolithic Analysis Model. The Force - Displacement Curve in one-story part in the Monolithic Analysis Model is featured by smaller value of the initial rigidity and of second rigidity in comparison with those of the foundation fixing model. In other words, it is presumable that in case of the Monolithic Analysis Model, the influence of a bending moment generated in the pile head section is being exerting upon the footing beams and upon the one-story column leg section.

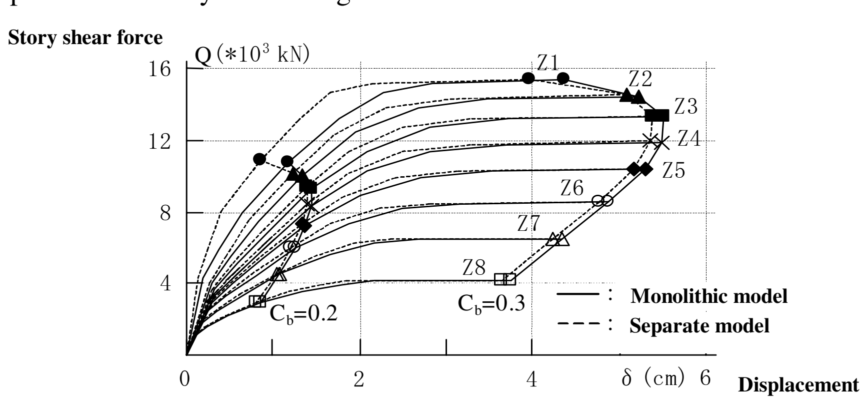


Figure14: Force-displacement curves in ridge direction

Span Direction:

Figure15 shows a Force - Displacement Curve in the span direction in the span direction according to the Monolithic Analysis Model. The pile head deformation is the value counted from the pile tip.

Figure16 shows a hinge occurrence status of X3 ways. At C_B=0.42, an yield hinge occurred on the column head in the pulling out section. At C_B=0.46-0.49, also an yield hinge occurred in the ground. In frames other than X3 ways, the yield hinge occurred in the pile head position among C_B=0.37(X1 ways)-0.44(X6 ways), as well as an yield hinge occurred in the ground among. C_B=0.41(X1 ways)-0.54(X6 ways). Immediately after passing C_B=0.54, and just before an yield hinge occurs in the compressed section, the status became unstable, and then, the analysis was terminated. Accordingly, at this period, the applicable layer to layer shearing force was defined as the ultimate limit, in the monolithic analysis model.

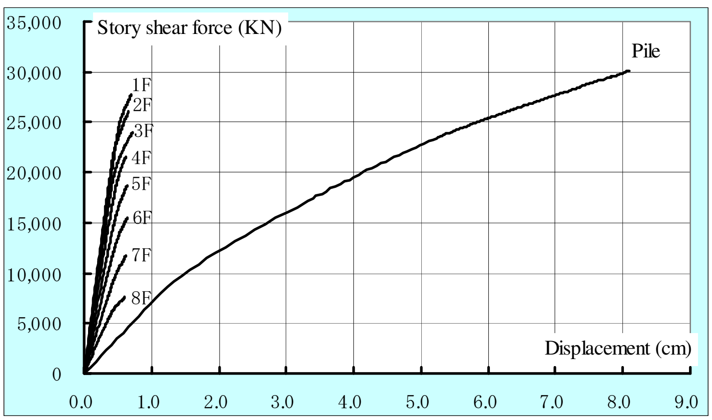


Figure15: Force-displacement curves in span direction

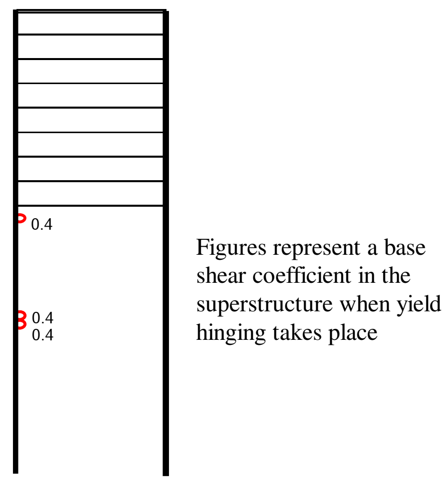


Figure16: Hinging in pile foundation monolithic model

Results of Earthquake Response Analysis depending upon the Response Spectrum Method

Contraction toward ESDOF

Figure17 shows the force-displacement relationship by making contraction toward ESDOF by using the results of the static non-linear increasing loading analysis in the monolithic analysis model. The displacement occurred at this time is the addition of horizontal displacement in the superstructure and the sway displacement of the pile foundation, as well as horizontal displacement by rocking angle. These are shown in the figure as a whole.

Figure18 shows the horizontal foundation spring constant, turning foundation spring constant and superstructure spring constant of the pile foundation obtained from the Expression (4).

Referring to Figure17 and Figure18, the displacement of the ESDOF in the rigid direction depending on the monolithic analysis model will be governed by the horizontal displacement component in the lower structure. On the contrary, it is known that the horizontal displacement in the superstructure has less governing force.

Calculation of Equivalent Viscous Dumping Constant:

The Equivalent Viscous Dumping Constant of sway and rocking according to the compound system frequency procured from the cone model method is shown in Table1. The Equivalent Viscous Dumping Constant of the monolithic analysis model was as follows: At damage limited design: h_{ed} = 20.9%, At safety design: h_{es} = 24.3%

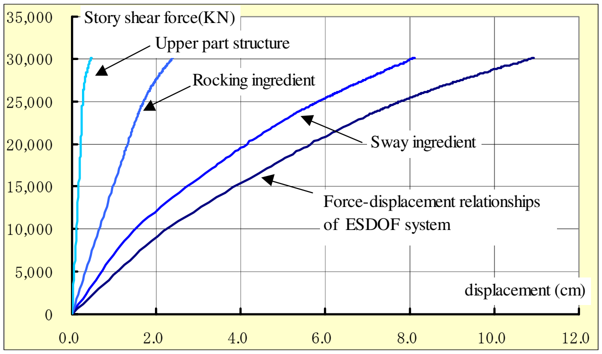


Figure17: Force-displacement relationships of ESDOF system

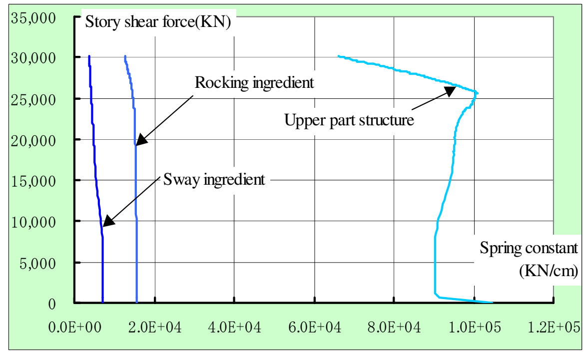


Figure18: Ingredient relation of every spring constant

Table1: equivalent viscous damping factor

Damage Controll Limit State	damping factor : h'	equivalent single-degree-of-freedom system shear force at monolithic model :Q(KN)	displacement : δ (cm)	rigidity :K	energy :W	equivalent viscous damping factor :he
upper part structure	1.2%	12017.1	0.130	9.227E+04 (KN/cm)	7.826E+02	20.9%
sway spring constant	28.6%		1.958	6.138E+03 (KN/cm)	1.176E+04	
rocking spring constant	5.0%		0.779	4.908E+10 (KN ·cm/rad)	4.683E+03	

Life Safety Limit State	damping factor : h'	equivalent single-degree-of-freedom system shear force at monolithic model :Q(KN)	displacement : δ (cm)	rigidity :K	energy :W	equivalent viscous damping factor :he
upper part structure	1.4%	25310.7	0.252	1.004E+05 (KN/cm)	3.189E+03	24.3%
sway spring constant	30.0%		5.960	4.247E+03 (KN/cm)	7.542E+04	
rocking spring constant	8.2%		1.727	4.665E+10 (KN ·cm/rad)	2.185E+04	

Setting of damage limited design:

The limited value of the damage limited design in the monolithic analysis model was specified as same as that of the foundation fixing model. The limited value of the safety design was specified at the time when any yield hinge may occur in the compressed section.

Examination of Results:

As shown in Table2, the damage limited design considering the dynamic interaction against the ESDOF contraction model according to the response spectrum is described, as well as the results of calculated response value in the safety design is described. In all cases, it was confirmed that the response values are specified less than the limited values.

Time History Response Analysis according to the Sway and Rocking Model

In order to compare the response value with the ESDOF contraction model of the monolithic analysis model, the sway and rocking model (hereinafter called SR Model) was set up by using the force-displacement relationship depending upon the monolithic analysis model. Then Time History Response Analysis was thus conducted.

Analytical Model:

The superstructure was modeled as 8 mass points equivalent bending shearing moment. Then the skeleton curve in layer to layer spring was procured by the force-displacement relationship depending on the static non-linear increasing loading analysis in the monolithic analysis model. The mass of foundation was specified to only the 1-story flooring section of a building and then, the mass of the foundation and piles was not considered. The restoration characteristics of the layer to layer springs were specified as origin-oriented-type, and then, the damping characteristics were specified as the instantaneous rigidity proportional type ($h_1 = 0.03$).

For the sway spring representing the interaction effects, it was matched with the initial rigidity of static non-linear increasing loading analysis based on the force-displacement relationship in the horizontal direction in the pile head position according to the monolithic analysis model.

Then the second rigidity was specified as the Bi-linear passing through two points of 4cm and 7cm of horizontal displacement. The restoration characteristics of the sway spring were three types; represented by Normal Bi-linear type, Home position orientation type and inverse type.

Then comparative analysis was made. The damping constant in the sway was varied as 30%, 20% and 10% respectively after considering the lowering of shearing rigidity cause by foundation shearing deformation and underground radiation damping, etc. For the rocking spring was featured by elastic element, and the rigidity was specified as the moment in the pile head position in the monolithic analysis model. Next, the difference between the pile head axis direction in the compressed section and pulling section was divided by the span length. Then the procured turning angle was used for the calculation. The damping constant in rocking was specified as 1/2 of the sway.

For the input earthquake wave, 10 waves of mimic earthquake motions were composed with phase-variable based on acceleration response spectrum on the release engineering foundation stipulated in the Building Code. Then the response acceleration wave forms in the foundation base level at the applicable earthquake motions (GL-1.6 m) were calculated.

Analytical Results:

The difference among the maximum response values according to the earthquake motions was observed in the fluctuations of about 15% from the averaged value of the layer to layer shearing force. When the restoration characteristics of the sway spring was varied, difference was observed max. 20% in the horizontal displacement, and max 10% in the layer to layer shearing force.

According to the Normal Bi-linear type, due to the large damping effects by history. Accordingly the difference of the damping constant of the horizontal spring, the difference of the layer to layer shearing force was very small, but the as far as the horizontal displacement is concerned the difference became large as 2.9-5.9cm.

Comparative Examination on Time History Response Analysis according to the Response Spectrum Method and SR Model using Foundation Fixing Model and Monolithic Analysis Model in the Response Spectrum Method

Comparison with Foundation Fixing Model and Monolithic Analysis Model in the Response Spectrum Method

The results of the response spectrum method depending on the foundation fixing model and monolithic analysis model were shown in Table2.

According to the foundation fixing model, the foundation weight is not considered. On the contrary, according to the monolithic analysis model the foundation is considered. Then the static non-linear increasing loading analysis is being conducted. Accordingly, the full mass became 1.17 times larger.

Table2: Analysis Result by Capacity Spectrum Meyhod

analysis model				separate model	monolithic model
building-ground interaction				out of consideration	in consideration
All masses			: M (ton)	5,290	6,175
Damage Controll Limit State	Limit value	strength at damage limit of buildings	: Q_{ud} (KN)	22,148	22,148
		base shar coefficient	: C_{Bud}	0.43	0.43
		Limit displacement	: δ_{ud} (cm)	0.52	0.52
	Response value	response strength	: Q_d (KN)	9,500	11,103
		base shar coefficient	: C_{Bd}	0.18	0.21
		response displacement	: δ_d (cm)	0.22	2.87
		effective mass	: M_{ud} (ton)	3,903	6,016
		effective mass ratio	: M_{ud} / M	0.75	0.97
		natural period	: T_d (sec)	0.75	0.75
		damping factor of building	: h_d	5.0%	20.9%
		acceleration decrease rate→ β ($\geqq$ 0.75	: η (*1)	1.00	0.48 → 0.75
Life Safety Limit State	Limit value	strength at life safety limit	: Q_{us} (KN)	58,142	28,067
		base shar coefficient	: C_{Bus}	1.12	0.54
		Limit displacement	: δ_{us} (cm)	6.57	11.17
	Response value	response strength	: Q_s (KN)	34,821	23,381
		base shar coefficient	: C_{Bs}	0.67	0.45
		response displacement	: δ_s (cm)	1.70	7.94
		effective mass	: M_{us} (ton)	4,375	6,102
		effective mass ratio	: M_{us} / M	0.83	0.99
		natural period	: T_s (sec)	0.29	0.87
		damping factor	: h_s	7.5%	24.3%
		acceleration decrease rate	: Fh (*2)	0.86	0.44

(*1) η : The reduction factor of the acceleration by the coupling effect at the natural response period at the damage limit

(*2) Fh : The reduction rate of acceleration according to the damping of the earthquake motion

The response value in the damage limited design was compared with the base shear coefficient: 0.18 of the foundation fixing model. In case of the monolithic analysis model, it was 0.21, and was 1.2 times larger. For the limited value in the safety design, the base shear coefficient: 1.12 of foundation fixing model was observed. On the contrary, according to the monolithic analysis model, it was 0.54; practical it showed half value. This is due to the fact that the limited value of the foundation fixing model in the safe design is featured by that the anti-earthquake walls are at the bending yield time. But, in case of the monolithic analysis model, the time is specified at the occurrence of yielding hinge at the pile head in the compressed section. The response value in the safe design, in comparison with the base shear coefficient : 0.67of foundation fixing model、 in case of the monolithic analysis model it was 0.45, and reduced by 70%. In other words, the attenuation constant became large depending upon the effect of dynamic interaction.

Comparative Examination on Time History Response Analysis according to the Response Spectrum Method and SR Mode by using the monolithic analysis model

Figure19 indicates the overlapping of the time history response analysis results according to the SR model on the limited value and response value at the safe designing in the yielding force in the ESDOF yielding force curve and required response spectrum, as well as in the foundation fixing model and the monolithic analysis model. In this case, however, the response value depending on the SR model are all averaged value of the input earthquake wave consisting of 10 waves. The response value depending on the monolithic analysis model were compared with the results of the time history response analysis depending on the SR model. In all cases, the evaluation has been made in the safe side.

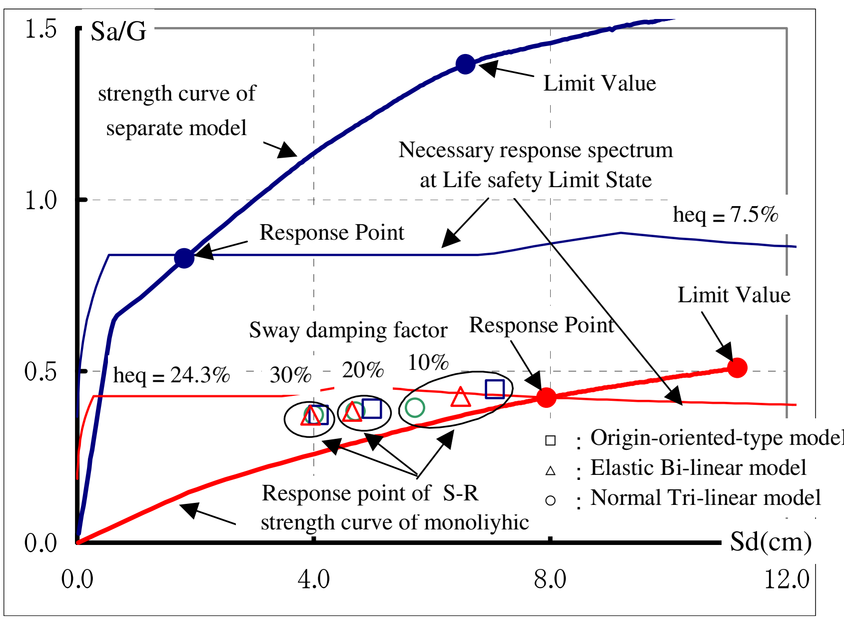


Figure19: Comparison of analysis result by the Capacity Spectrum Meyhod and response analysis with S-R model

CONCLUTIONS

According to the examination results obtained this time, caused by the difference of stress occurrence between the monolithic analysis model and conventional foundation fixing model, miscellaneous types troubles represented by cracking in structural members, occurrence of yield hinge and collapse form, as well as force-displacement relationship have occurred. In particular, in the rigid direction, for the foundation fixing model, the anti-earthquake walls were yielded, and in the monolithic analysis mode, the piles were yielded. As a result, a great difference was observed on the evaluation of collapsing form of both the walls and piles.

Through the monolithic analysis model, if force-displacement relationship in which interaction effects are contained was procured. In the event that if the response spectrum method was applied, both the damage limited design and safety design, it was resulted that the damping constant has become large, at least 4 times larger than that of the foundation fixing model (the interaction effects are not considered). Moreover, when the response value of the base shear conversion is compared with the procured value, at the damage limited design, the monolithic analysis model has increased by approx. 17% in comparison with the foundation fixing model. But, at the safety design, it was resulted that approx.43% has been decreased.

In comparison with the response values at the safety design according to time history response analysis depending upon the response spectrum method and the SR method, the response value depending upon the response spectrum method has become the evaluation in the safety side in comparison with the response value of the SR model.

In the future, a parametric study shall be exercised paying special attention to the scale, type of building, as well as to the difference of ground characteristics. At the same time, by checking up the actual behaviors, represented by earthquake observation records and the relating data, we have been able to grasp the general tendency. Based on such an idea, we would line to propose the rational designing method and design criteria of the whole building including foundation structure.

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