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Dynamic design procedure for buildings in waterfront area

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ABSTRACT: Many highrise buildings are presently being planned on soft soil sites in waterfront areas. This paper proposes a rational dynamic design procedure for these buildings, taking into account the predicted ground motions, local site effects of soft soils, soil-structure interaction, soil non-linearity and liquefaction. The procedure has been applied to a 33-story reinforced concrete condominium supported by wall foundations and piles. Resulting stresses and strains are compared with those of the conventional design for earthquake waves applied to the first floor without considering different soil conditions. The design shear force in the superstructure was about the same as those for the conventional design. The vertical stress distribution along the wall foundations differed from those for the conventional design due to the interaction between the wall foundations, piles and surrounding soils.

1 INTRODUCTION

Highrise buildings have previously been built on rigid foundations in stiff soils, and their dynamic analyses have been conventionally performed using several earthquake waves observed under similar soil conditions with adjusting the maximum velocity amplitude.

However, many highrise buildings are presently being planned on soft soil sites in waterfront areas, e.g. the Tokyo metropolitan coastal area. In this area, up to about 25 meters of fill or alluvial soft soil overlie the diluvial stiff soil.

In seismic design of these buildings, the following three characteristics should be taken into account (Ohta 1991).

1. Local site effects of soft soils, that is, high amplification of ground motions, longer predominant period components and incidence of surface waves.
2. Soil-structure interaction.
3. Soil nonlinearity and liquefaction.

A dynamic design procedure is proposed in which these characteristics are properly evaluated. The procedure is readily applicable to the site with different soil conditions.

2 DESIGN CRITERIA

The building shall satisfy the following two design criteria.

1. The stresses and story drift angles caused by a level 1 earthquake shall not exceed the allowable stresses and 1/200 respectively. A level 1 earthquake is one which may be expected to occur more than once during the life of the building.
2. The ductility factors of each story and story drift angles caused by a level 2 earthquake shall not exceed 2.0 and 1/100, respectively. A level 2 earthquake is the strongest to have occurred at the site in the past or expected to occur in the future.

Level 1 and 2 earthquakes are recommended as standard waves by the Building Center of Japan. They are the observed waves of El Centro NS, Taft EW, Hachinohe and others with adjusted maximum velocity amplitude of 25 kine and 50 kine for level 1 and 2 earthquakes, respectively.

In addition to the conventional standard waves, newly developed earthquakes have been adopted for the design of buildings in waterfront areas. This paper mainly presents the seismic design for a level 2 earthquake, using the strong ground motion generated at seismic bedrock.

3 DESIGN PROCEDURE

The flow of the dynamic design procedure is as follows.

1. Perform preliminary structural design based on static analyses using equivalent lateral loads and dynamic analyses using the

standard earthquakes. This is the same as the conventional design method.

2. Select the most severe earthquakes (level 2) that have been occurred or expected to occur, by referring to historical earthquake data, past earthquake damage and seismic activity at the site. Then, generate earthquake waves in the seismic bedrock based on fault models and earthquake records near the site.

3. Estimate input waves at the bearing stratum from bedrock motions, based on one dimensional wave propagation theory.

4. Carry out nonlinear earthquake response analyses using interaction models composed of the superstructure, basement, wall foundation and piles, and the surrounding soils.

5. Check the stress and deformation of the structure, and if the resulting stresses and strains differ from the design criteria, feed them back into the structural design. Final design of the building is obtained after several iterations.

6. After the building has been constructed, perform vibration tests and earthquake observations to verify and improve the seismic design method.

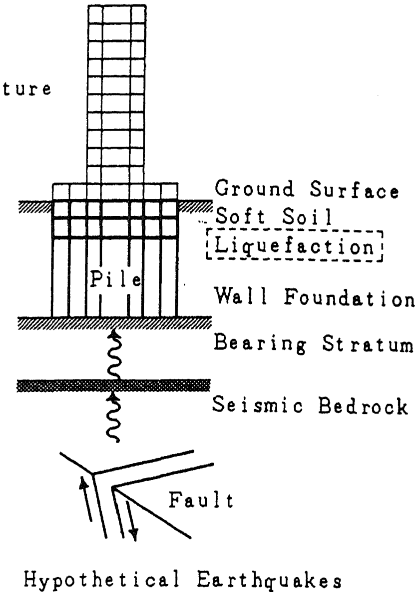

Fig.1 Typical structure in waterfront area

4 APPLICATION

4.1 Outline of the building

The above design procedure has been applied to the seismic design of a 33-story reinforced concrete condominium with two basement levels, supported by a combination of wall foundations and piles. Fig. 2 shows a section of the building, whose maximum height is 104m and typical story height is 2.95m. The typical floor plan is square with a 30m side length and has 5 spans in X-direction and 6 spans in Y-direction as shown in Fig.3. The structure consists of ductile moment resisting frames.

The foundation structure is the strong and rigid, consisting of 6m-deep foundation girders disposed in the shape of parallel crosses between the bases of columns. The 1.2m-thick wall foundations are disposed under the exterior sides, while piles with expanded bottoms are disposed right under the interior columns as shown in Fig.4. The wall foundations have vertical slits and are divided into several elements.

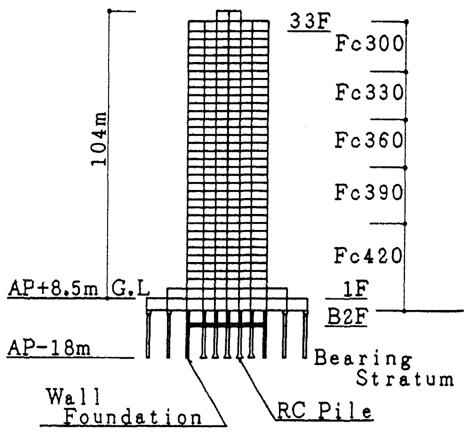

Fig.2 Section of the building

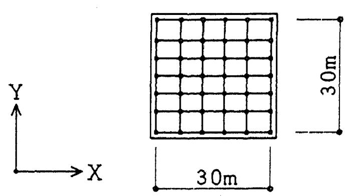

Fig.3 Typical floor plan

4.2 Outline of soil profile

The site is located in the Tokyo metropolitan coastal area, which is a typical soft soil area. The soil profile is shown in Fig.5. The strata consist of alternate 12m-deep sandy and cohesive alluvial soft soils. The wall foundations and piles are supported on the 6.5m-thick diluvial Tokyo gravel layer, whose N-value is at least 50.

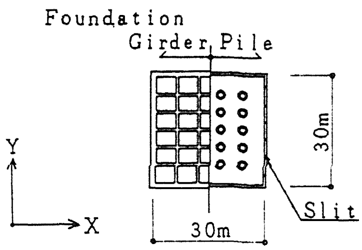

Fig.4 Foundation girder, wall foundation and pile

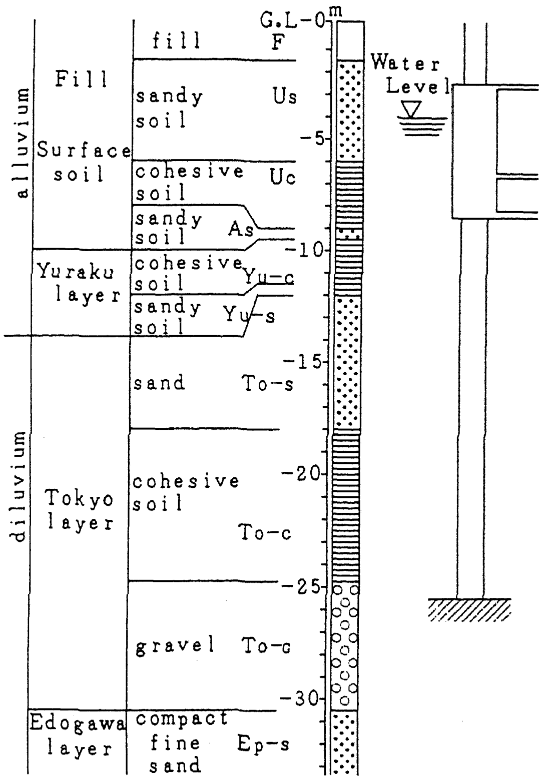

Fig.5 Soil profile

4.3 Generation of ground motions

Earthquake motions are evaluated as incident waves on the outcrop of seismic bedrock (stiff diluvial deposit). For this site, the seismic bedrock and the bearing stratum of the foundations (Tokyo gravel layer) coincide.

Two kinds of analytical method based on the fault model are adopted for three hypothetical earthquakes (1885 Ansei-Edo, 1923 Kanto, Hypothetic Tokai). One is a semi-empirical method develped to estimate strong ground motions during large

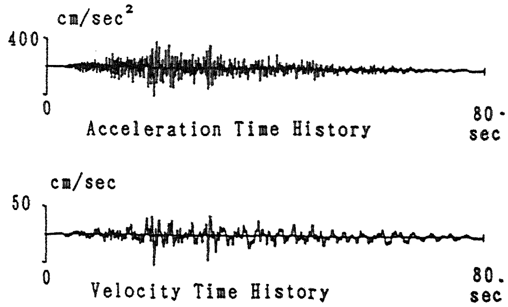

Fig.6 Time history of hypothetic Kanto earthquake

earthquake, using the records of small earthquakes as the empirical Green's function. The other consists of the superposition of normal modes in multilayered elastic media, taking fault parameters into account.

The evaluated strong motion of the hypothetic Kanto earthquake (Niwa 1992) is employed as the input ground motion for level 2, as it is most destructive against the object building. Fig.6 and Fig.9 shows the time history and response spectrum of the hypothetic Kanto earthquake, respectively. The maximum acceleration of the earthquake is 407 gal and that of velocity is 53 kine.

4.4 Prediction of liquefaction

Liquefaction was predicted by the effective stress analysis method and the total stress analysis method. The results of both analyses indicate that liquefaction takes place in the Us, As and Yu-s layers during level 1 and 2 earthquakes. Stiffness reduction due to liquefaction was considered in the earthquake response analysis.

4.5 Soil-foundation interaction model

The soil-foundation interaction model is composed of lumped masses with the bending and shearing rigidities of the wall foundations, springs with the equivalent linear properties of surrounding soils and the free field soils as shown in Fig.7.

1. Spring constants of soil

The free field motions and equivalent stress dependent shear modulus and damping factors of the soils overlying the seismic bedrock was estimated by the wave propagation theory (SHAKE).
The spring constants of the soils between adjoining lumped masses of wall foundation,

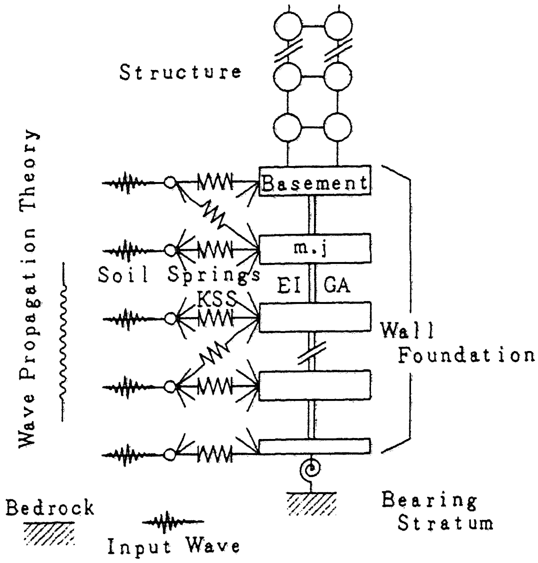

Fig.7 Soil-structure interaction model

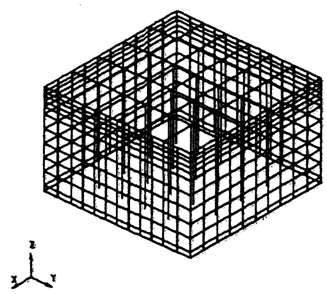

Fig.8 Mesh layout of 3-D FEM model

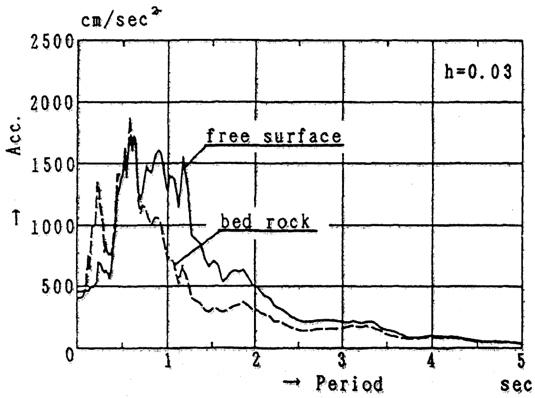

Fig.9 Response spectra at seismic bed rock and ground surface

piles and free field soils were estimated from the 3-dimensional static Thin-layered Element Method analysis under lateral loads. Similarly, the spring constant for the rocking motions of the foundation are estimated under vertical load.

2. Modeling of wall foundations and piles

Modeling of wall foundations and piles was established from the results of a 3-dimensional finite-element-method analysis (Masuda 1991). Shell elements are used for the wall foundation with vertical slits and slabs and beam elements are used for the foundation girders and piles. The mesh layout of this 3D FEM analysis is shown in Fig.8. The equivalent bending-shearing model is evaluated based on the relation between unit applied force and displacement.

Fig.9 shows the response spectra of 3% damping factor at the seismic bedrock and ground surface. The surface spectrum includes the site effects and it can be compared directly with those of the standard waves.

4.6 Vibration model of the building

The final structural members and materials were determined after several repetitions of level 1 and 2 earthquake responses.

1. Structural members

Typical cross sections and reinforcing arrangements of the columns and beams are shown in Fig.10. The sizes of the columns and beams and the rebar arrangement were unified as much as possible to simplify construction and avoid sudden changes in stiffness.
The nominal compressive strength of 300-420 kg per square centimeter was used as shown in Fig.2. The longitudinal rebars were large deformed bars with maximum diameters of 41mm. Center bars were installed in the corner and exterior columns to resist the large overturning moments. The shear reinforcements in the columns consists of spiral and square hoops, and that in the beams consists of stirrup ties.

2. Vibration model

The vibration model of the building comprises 34 lumped masses, one mass being cocentrated at each floor. Each frame is substituted by a bending-shearing element whose bending flexibility due to axial deformation in the columns is assumed to be elastic and whose shearing flexibility due to frame deformation is assumed to be non-linear. The non-linear relations of frame deformations are idealized based on the results of static analysis under monotonous incremental lateral loads. Each element is linked at its floor level according to the rigid floor assumption.

4.7 Earthquake response analysis

The total soil-structure interaction model is established by combining the building model with the soil-foundation interaction model. The free field motions are applied at the exterior ends of the soil springs. The internal damping factor is assumed to be 3% for the first mode.

Table 1 shows the periods of the total interaction model and the building model fixed at the base. The first natural periods of the interaction models are about 15% longer than those of the fixed base models due to the sway and rocking effects.

As the results of earthquake response analysis are similar in X and Y-directions, the result is shown only for X-direction.

1. Maximum response of building

Fig.11 shows the maximum story drifts in X-direction together with those for conventional standard waves of level 2 earthquakes. The story drift is less than 1/100 and the ductility factor is relatively small.

Fig.12 shows the maximum story shear forces in X-direction. The shear forces against the hypothetical Kanto earthquake are a little smaller than those of the standard waves at the middle stories and a little larger in the lower stories. As a whole, the shear force is about the same as for the conventional design.

2. Maximum response of foundations

Fig.13 shows the maximum shear forces distribution in the wall foundations. This distribution suggests a complicated interaction between foundation and soils.
As the maximum shear forces do not occur simultaneouly, the shear force distributions are checked step by step and two patterns are found. In one, the maximum shear force occurs at the upper part of the foundation walls and in the other, the maximum shear force occurs at a lower part. The former pattern can be anticipated by the conventional method, whereas the latter pattern is obtained only by this kind of interaction model.

Similarly, the bending moment distributions

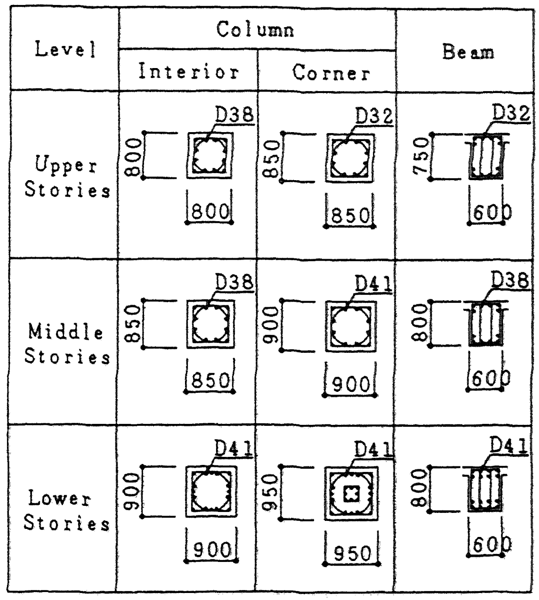

Fig.10 Cross sections of columns and beams

Table 1 Natural period (sec.)

Order	Interaction		Fixed base	
	X dir.	Y dir.	X dir.	Y dir.
1	2.12	2.09	1.86	1.81
2	0.66	0.66	0.60	0.58
3	0.40	0.32	0.33	0.32

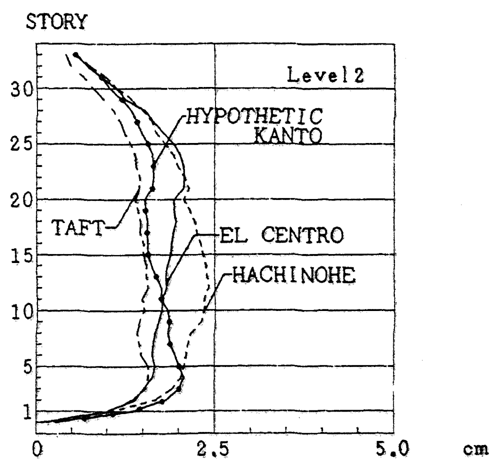

Fig.11 Maximum story drifts

are calculated step by step. Fig.14 shows the maximum bending moments of the one element in the wall foundations. The time steps of the solid and dotted lines are the same as those of the lines for shear forces in Fig.13.

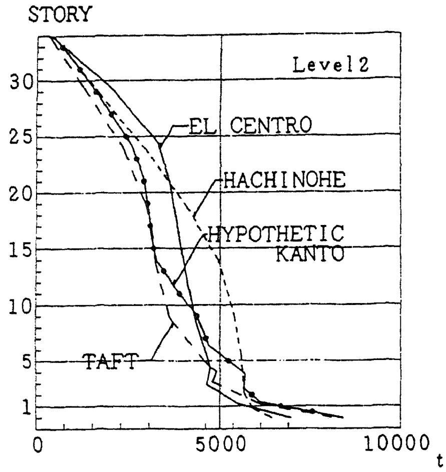

Fig.12 Maximum story shear forces

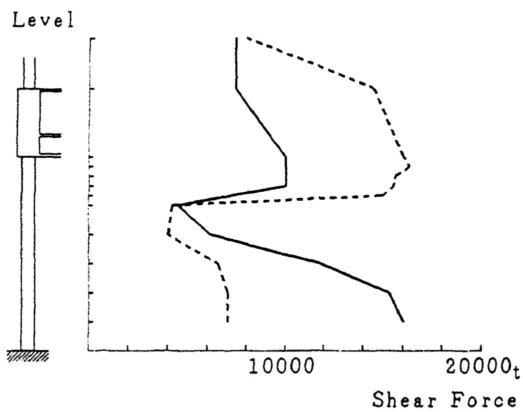

Fig.13 Maximum shear forces of foundations

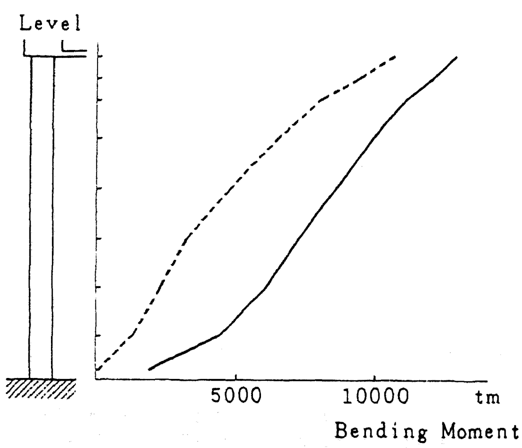

Fig.14 Maximum bending moments of foundations

5 CONCLUSION

Waterfront areas are typical examples of soft soil sites. A rational and practical dynamic design procedure has been proposed. Through the application of this procedure, the following conclusions are obtained.

1. The design shear force in the building is about the same as for conventional method.
2. The vertical stress distribution along the wall foundations differs from that of the conventional method due to the interaction between the foundation and the surrounding soils.
3. It is possible to design a substructre that is safe even under liquefaction.

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REFERENCES

Masuda,K. et al.1991. Dynamic soil- structure interaction analysis considering diaphragm wall and pile group. Int. Cof. on Computational Eng. (ICES '91)

Niwa,M.et al. 1992. Strong ground motion for aseismic design of highrise building in Tokyo metropolitan coastal area by using earthquake fault model. Proc. 10WCEE, Madrid, Spain.

Ohta,T.et al. 1991. Advanced seismic design of highrise buildings on soft alluvial soils. Annual report of Kajima institute of construction technology, Vol.39,125-132.