11 WCEE
Copyright © 1996 Elsevier Science Ltd
Paper No. 1452. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

SEISMIC RESPONSE ANALYSIS FOR A COLLAPSED UNDERGROUND SUBWAY STRUCTURE WITH INTERMEDIATE COLUMNS

TAKASHI MATSUDA, NORIKAZU TANAKA

Technical Research Institute of Obaysahi Corporation,
4-640,Shimokiyoto, Kiyose, Tokyo 204, JAPAN

ABSTRACT

Underground subway structures in Kobe suffered severe damage during the 1995 Hyogoken-Nambu earthquake. Major damage occurred at RC intermediate columns. To clarify the damage factors, two-dimensional seismic response analysis was conducted. Target underground structure is two story RC box culvert with intermediate columns. The results of response section forces induced by assumed earthquake waves were compared with a conventional criteria for concrete structures in Japan. From the result of response displacement, deformation of intermediate columns is affected both deformations of upper and lower slab, which makes larger magnitude and complex deformation. This large deformation can be thought a cause of the damage. The primary cause of the damage is large shear stress at intermediate columns. It is evaluated that the maximum shear stress attain to almost twice of the shear strength, although axial stresses have enough margin against the design compressive strength in this cross section.

KEYWORDS

damage analysis; Underground structure; intermediate columns of RC; seismic response analysis; equivalent linear method; ultimate strength of concrete

1. INTRODUCTION

During the 1995 Hyogoken-Nambu earthquake, underground subway structures suffered severe damage (Kobe city, 1995). The primary damage of subway structure occurred at intermediate columns of concrete box culvert as shown in Fig.-1. It was thought that large amplitude of relative displacement of underground structure induced by ground motion affected the collapse of intermediate columns. In order to represent seismic behavior of the collapsed underground structure (the part of Kamisawa station, Kobe city), the authors conducted a seismic response analysis using two-dimensional equivalent linear method. First stage of the investigation was a simulation analysis for the collapsed structure. Simulated stresses of the structure were evaluated by ultimate strength based on the Japanese standard for concrete structures. Furthermore, comparative analysis that the intensity of input motion and the thickness of stratum were taken as analytical parameters was performed to clarify a factor related to the collapse of underground structure. Analysis conditions was so limited that predicted soil parameter and input motion were adopted in the analysis. Computed stresses including static loading condition were used for the evaluation of seismic stability.

2. ANALYSIS PROCEDURE

2.1 Outline of the damage

Fig.-1 shows the target section of two-story concrete box culvert with asymmetric intermediate columns. Severe damage occurred at intermediate columns of the upper deck while shear cracks occurred at the lower columns and slight cracks appeared at the upper slab and side walls. Interval distance in the longitudinal direction of intermediate columns is 5m. Main reinforcement of the columns with the width of 0.7m by 1.4m in cross section consists of 32mm deformed bar installed every 10cm interval with double placement. There

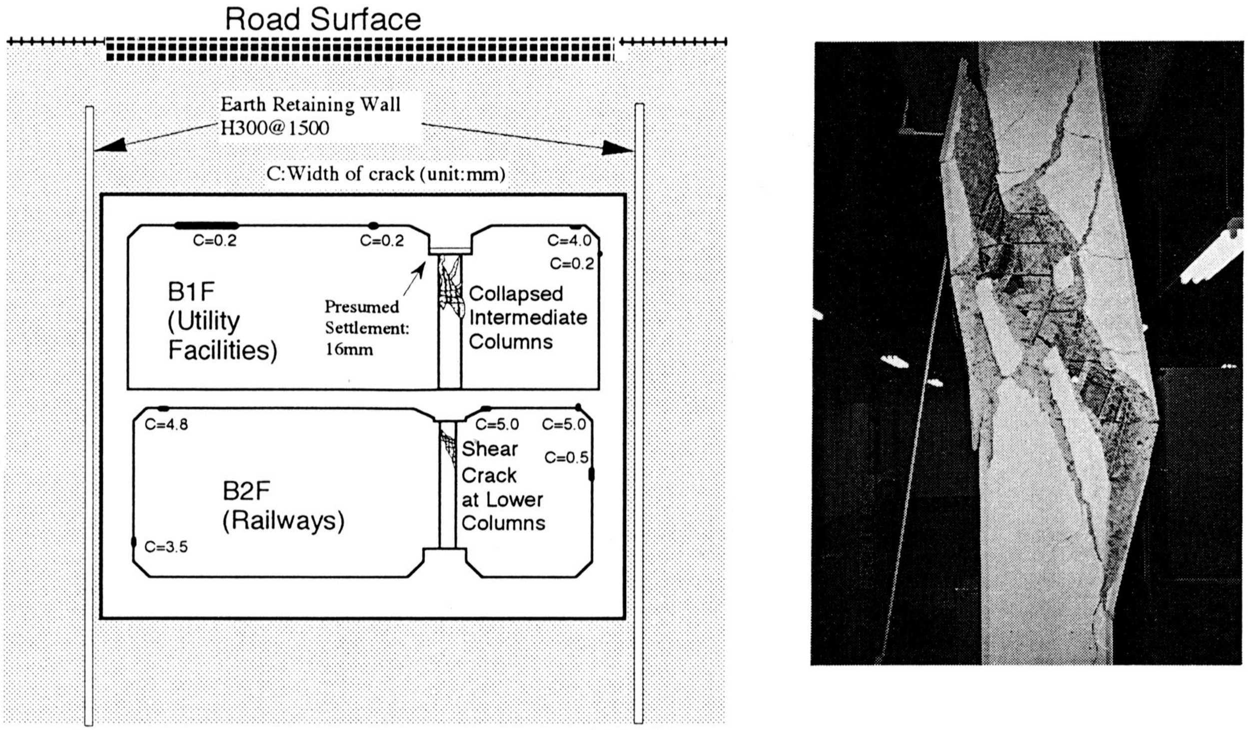


Fig.-1 Damaged cross section (Kamisawa Station in Kobe, C-section)

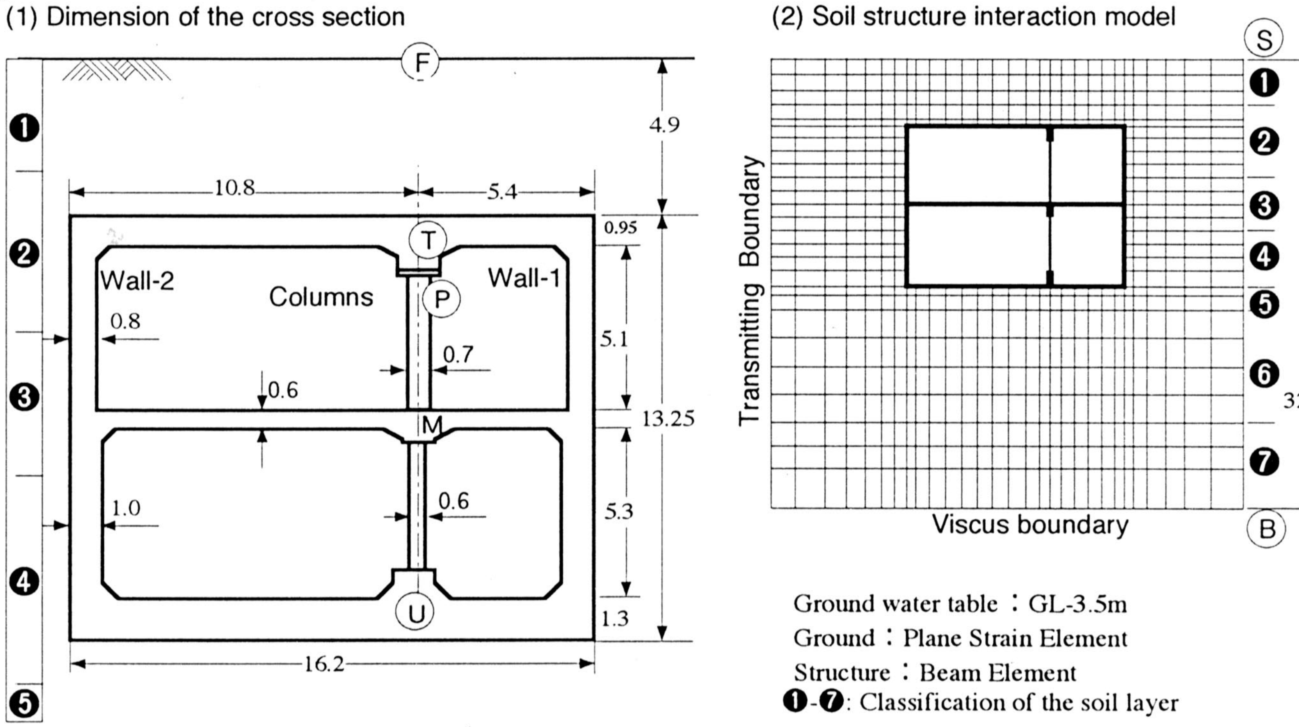


Fig.-2 FEM model and soil classification

was no severe subsidence on the surface road, while presumed settlement of the upper slab was 16mm.

2.2 FEM model

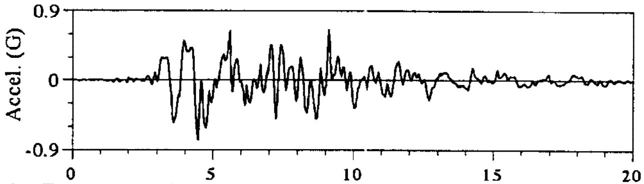
Two-dimensional equivalent linear analysis code for nonlinear seismic response was adopted for the simulation against the damage. Fig.-2 shows FEM mesh with classification of the soil layers. The model of surrounding soil layers were determined by using the soil investigation that was conducted at the construction of this railway system. The soil properties are listed in Table-1. The positive soil properties for the response analysis were unit weight and N-value. S-wave velocity (Vs) and initial shear modulus are evaluated by using the relationship between N-value and Vs ($Vs=97N^{0.323}$). Lower boundary represented by viscus boundary elements of soil layers was set at GL-32.7m. The subsurface soil layer consists of alternating diluvium sandy soil, clayey soil and the Osaka Unit as the engineering basement layer where the incident wave was set up.

Table-1 Soil properties

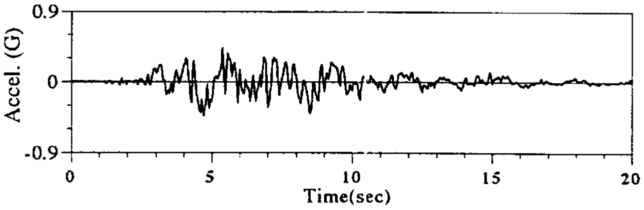
Layer	Depth GL-(m)	Soil Type	Unit Weight (tf/m^3)	S-wave Velocity (m/sec)	Initial Shear Modulus (tf/m^2)	Reference Strain (10^{-4})	Maximum Damping Ratio
❶	3.5	Gravel	1.8	150	4590	4.4	0.25
❷	9.0	Sandy Silt	2.0	200	8160	6.4	0.25
❸	13.0	Sandy Clay	2.0	230	10790	19.4	0.15
❹	17.5	Silt	1.7	250	10840	14.1	0.25
❺	19.0	Gravel	2.0	260	13790	11.1	0.25
❻	27.5	Gravel	2.1	270	15620	12.6	0.25
❼	32.7	Sandy Clay	1.7	250	10840	19.4	0.15

The nonlinearity of soil layers in the horizontal excitation were taken into account by using H-D model. The parameters of H-D model such as reference strain and maximum damping ratio were determined by the results of the previous research (Irikura *et al.*, 1995). In the vertical excitation, soil's properties were assumed as linear condition due to the result of the simulation analysis for the recorded vertical Port Island wave in Kobe. The structure model consists of linear beam elements with the damping ratio of 5%.

(a) Surface Layer (S)



(b) Basement Layer (B)



(c) Transfer Function (S/B)

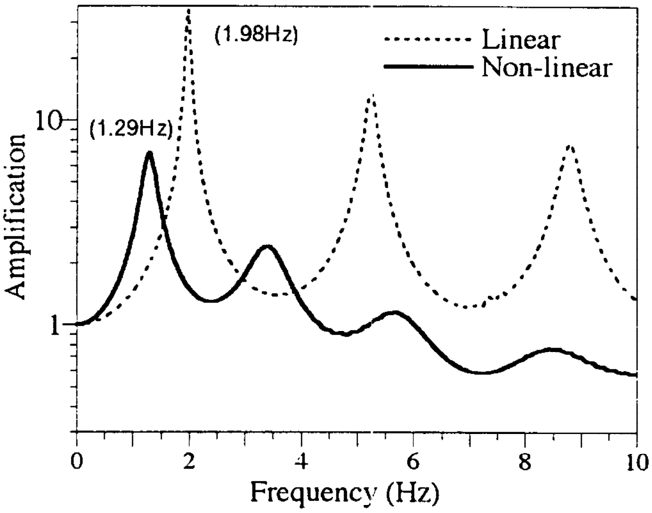


Fig.-3 Amplification characteristics and input motion

2.3 Input motion

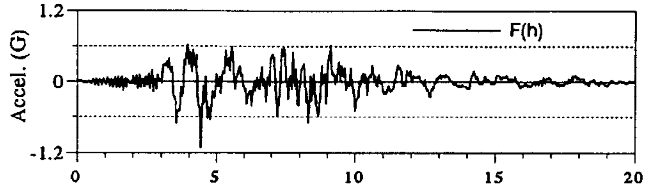
There was no recorded earthquake waves around the collapsed underground structure, so that the input motions for the analyses were set up by using recorded earthquake waves in Kobe(Port Island). This recorded waves were measured at GL-79m in basement layer(Vs=320m/sec). The incident waves were computed by the multi-reflection theory with equivalent linear procedure. The horizontal input motion analysis and the vertical input motion analysis were conducted separately.

Fig.-3(c) shows the transfer function between analytical basement layer (designated as B) and the surface (S). The natural frequency under small strain level indicates 1.98Hz, which is close to the predominant frequency of the mincrotremor observed at the adjacent area. The natural frequency declines to 1.29Hz under large strain level due to the nonlinearity of the soil. Fig.-3(a) and (b) present response acceleration waves of the basement layer and surface layer, respectively. The maximum acceleration at the basement layer (B) indicates 0.43G and that of surface (F) attains to 0.78G. The large amplitude at the surface (F) appears between 3 and 10 second.

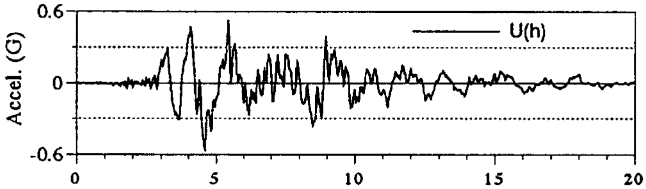
2.4 Analysis cases

The present investigation consists of two analyses designated as simulation analysis and parametric analysis. Simulation analysis provides the primary causes of the collapse, and parametric analysis gives various factors

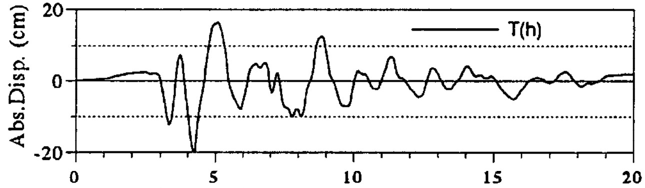
(1) Horizontal acceleration at F



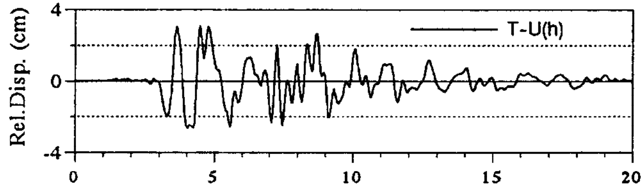
(2) Horizontal acceleration at U



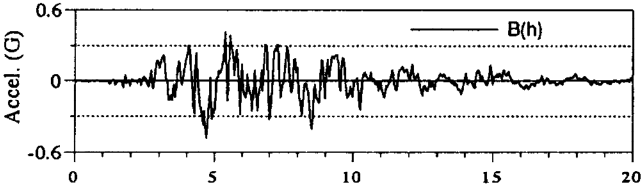
(3) Horizontal displacement at T(Absolute)



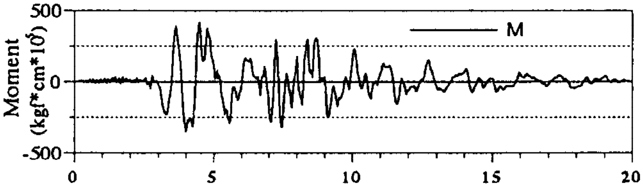
(4) Horizontal displacement between T-U(Relative)



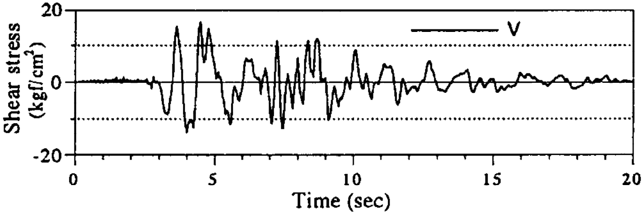
(5) Horizontal acceleration at B (Basement layer)



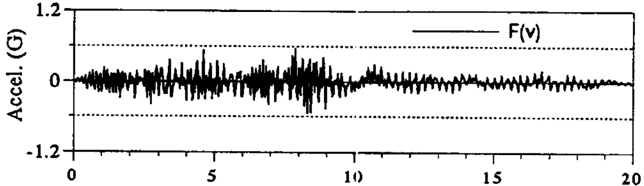
(6) Moment at P



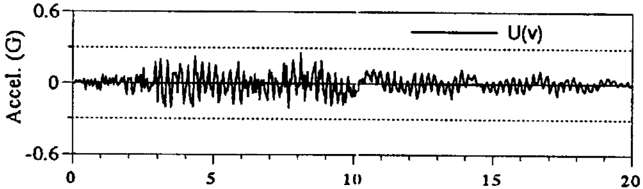
(7) Shear stress at P



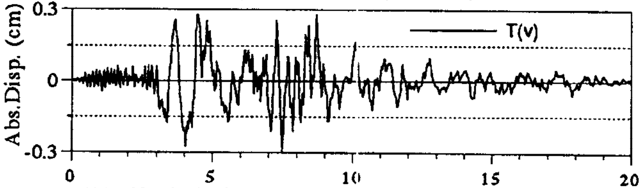
(8) Vertical acceleration at F



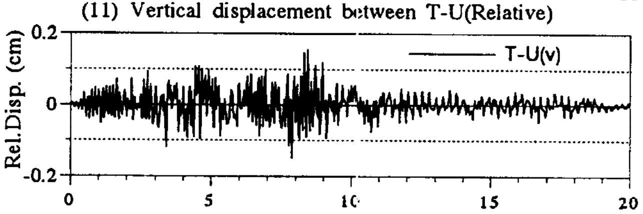
(9) Vertical acceleration at U



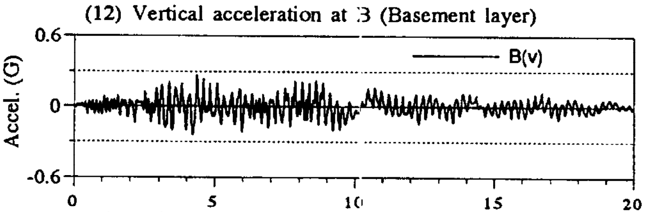
(10) Vertical displacement at T(Absolute)



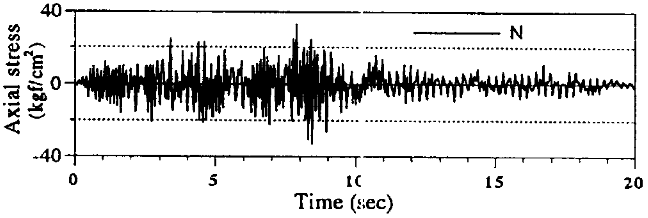
(11) Vertical displacement between T-U(Relative)

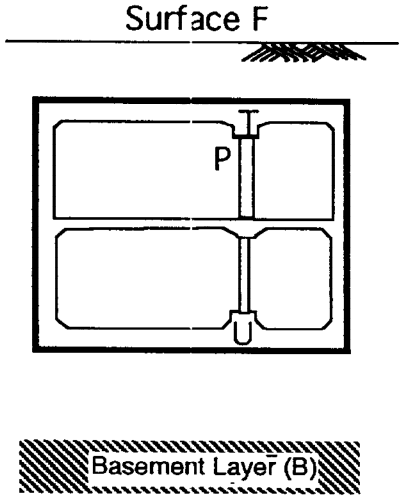


(12) Vertical acceleration at B (Basement layer)



(13) Axial stress at P





F : Surface of Ground
T : Upper Slab
P : Intermediate Columns
U : Bottom Slab
B : Basement Layer
h : Horizontal Input Motion
v : Vertical Input Motion

M : Bending Moment (kgf*cm)
V : Shear Stress (kgf/cm^2)
N : Axial Stress (kgf/cm^2)

Fig.-4 Response waves

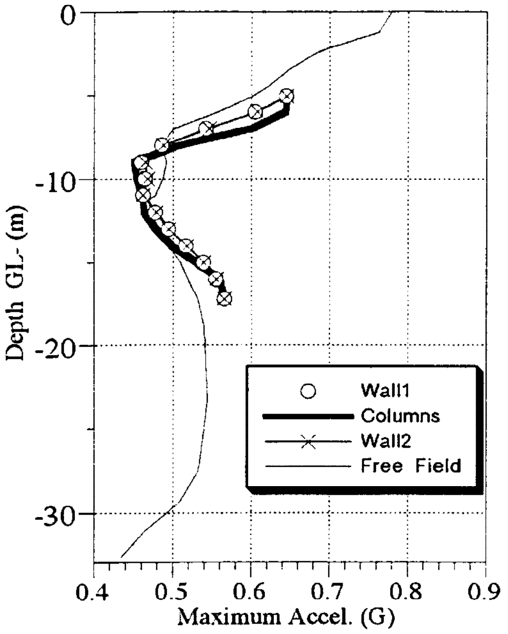


Fig.-5 Distribution of maximum acceleration

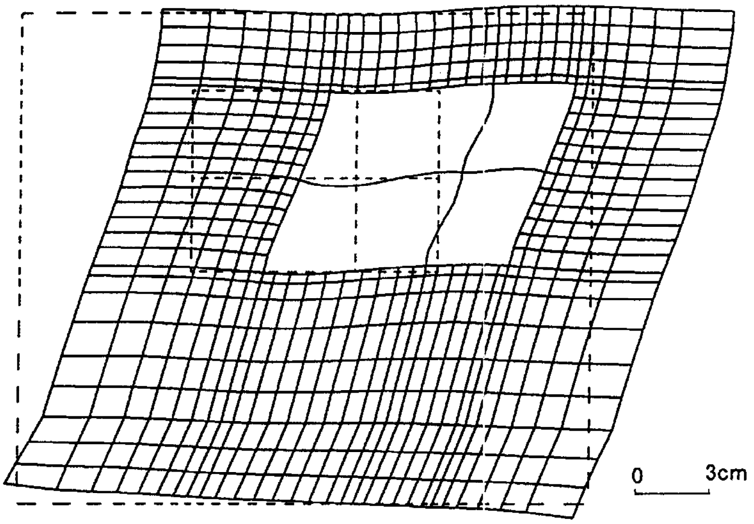


Fig.-6 Distribution of maximum displacement

for the damage. The results of simulation analysis were compared with the real collapse phenomena. Parametric analysis considered the intensity of input motion and the depth of sedimentary layer under the structure as parameters.

3. SIMULATION ANALYSIS FOR THE DAMAGE

3.1 Response acceleration and displacement

Response time histories of accelerations, displacements and stresses in the intermediate columns are shown in Fig.-4. Fig-4(5) and (12) present accelerations of the basement layer. Horizontal response acceleration at the ground surface (F in Fig.-4(1)) indicates large amplitudes of about 0.7G between 3 and 10 second. Vertical acceleration at (F in Fig.-4(8)) shows large amplitudes of about 0.4G as well before the time which the peak of horizontal acceleration appears. The maximum displacement amplitude of the ground surface (F, Fig.-4(1)) is 20cm in the horizontal excitation. Fig.-4(4) presents the relative displacement between (T) and (U), which is equal to differential deformation of the cross section. This relative displacement attains to 3.1cm which makes the average shear strain of 2.3×10^{-3} calculated from the height of 13.25m. It should be noted that shear strain magnitude of 2.3×10^{-3} is close to the ultimate strain of concrete members, which was related to the collapse. Time histories of bending moment and shear stress at the intermediate columns shown in Fig.-4(6),(7) coincides with the relative displacement of the structure (Fig.-4(4)) in the horizontal excitation. That of axial stress coincides with the relative displacement in the vertical displacement shown in Fig.-4(11) as well. From the relationship among these time histories, it is recognized that the section force of the underground structure is primarily affected by relative displacement.

Fig-5 shows the distributions of the maximum accelerations. Thin solid line indicates that of free field, and it is shown that the responses of the structure indicate slightly larger than free field. Fig.-6 illustrates the distribution of the maximum displacement. The deformation of walls coincides with the adjacent ground deformation, which affects the deformations of upper and lower slabs. The deformation of intermediate columns is affected both deformation of upper and lower slab, which makes larger magnitude and complex deformation. This large deformation can be thought a cause of the damage.

3.2 Section force and damage factor

Fig.-7 shows the maximum section forces of bending moment, shear stress and axial stress under static condition, horizontal excitation and vertical excitation at wall-2, respectively. Bending moment and shear stress under horizontal excitation predominate, while axial stress under static condition predominates over that under seismic excitation.

Fig.-8 indicates the maximum section forces at walls and intermediate columns, respectively. The maximum section forces consist of those under static condition, horizontal excitation and vertical excitation, these maximum values were calculated by absolute maximum value of seismic condition adding to that of static condition. It is shown that the distribution at the upper part of intermediate columns is significantly larger than other part. Besides this, bending moment and shear stress at the lower part of the walls indicate large sectional forces, which affected occurrence of the cracks at these portion.

The damage factor is defined by the ratio of section forces against the ultimate strength, herein. When the damage factor exceeds one, any damage will be suspected. We adopted the Japanese civil engineering standard for concrete structures (Japan Civil Engineering Society., 1993) (designated as only the standard, hereafter) as evaluation procedure for ultimate strength of cross section. The concrete compressive strength was assigned design value of 210kgf/cm^2. According to the standard, yield bending moment and ultimate shear strength are related to axial forces. It was assumed that axial forces for the evaluation of ultimate strength consists of the static values. The distributions of the damage factors at walls and columns are shown in Fig.-9. It should be noted that the damage factors of shear stress at upper part of intermediate columns entirely exceed 1 and almost attain to 2, which can verify the collapse of this part. Besides this, the damage factors of bending moment and shear stress at the lower part of wall slightly exceed 1, which can reveal the cracks of this portion. However, there was no large damage factor around the lower part of intermediate columns, so that it can not explain the shear crack here. From Fig.-4(6) and (7), the section forces resulting the collapse of the intermediate columns should occur at the moment of around 4.5 second. There are large amplitudes of the relative displacement after 5 second, which subjected to the collapsed structure. It is thought that the crack at intermediate columns of B2F should occur in the stress redistribution condition due to the collapse of intermediate columns at B1F. However, this procedure can not be directly assessed by the present analysis method. On the other hand, the damage factors of axial stress are less than 1, which means that axial stresses has rather margin. However, it is thought that seismic axial stress increment affected shear and bending moment strength and the behavior of the collapsed columns due to large shear stresses.

4. PARAMETRIC ANALYSIS

4.1 Effect of seismic intensity

Two cases of which intensity of the input motion were reduced to 83% (designated as Case-83%) and 70% (ditto Case-70%) were computed to clarify the effect of magnitude. The results of these analysis cases will

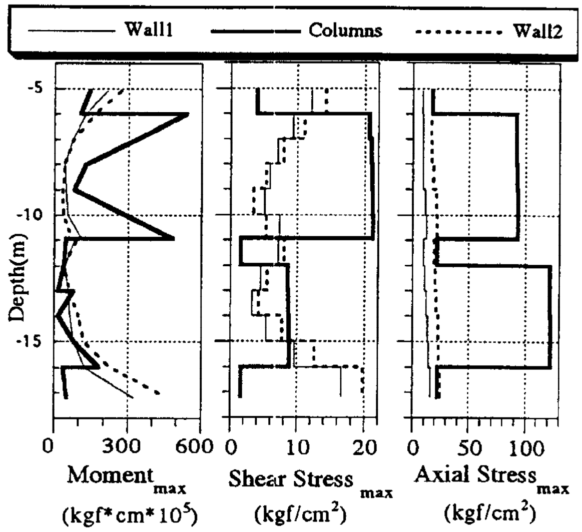


Fig.-7 Component of section forces

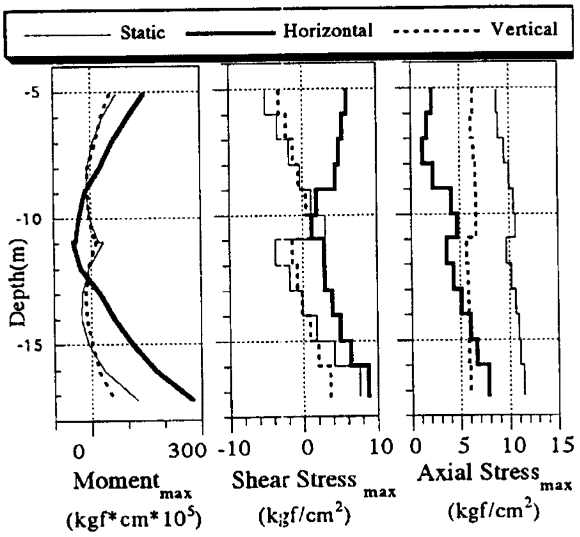


Fig.-8 Maximum section forcedistribution

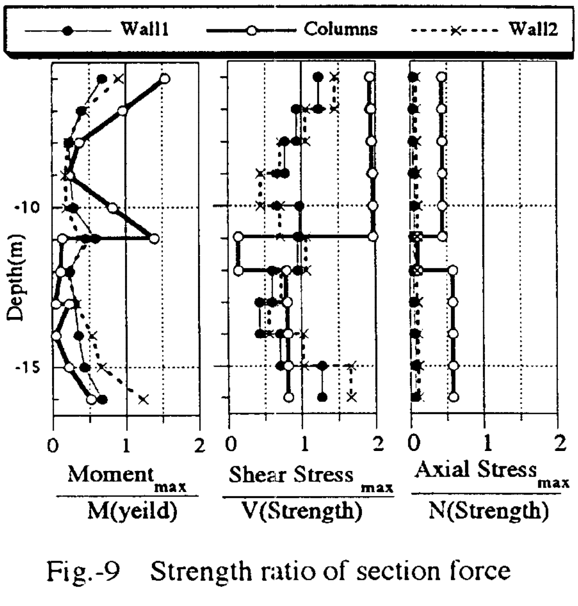


Fig.-9 Strength ratio of section force

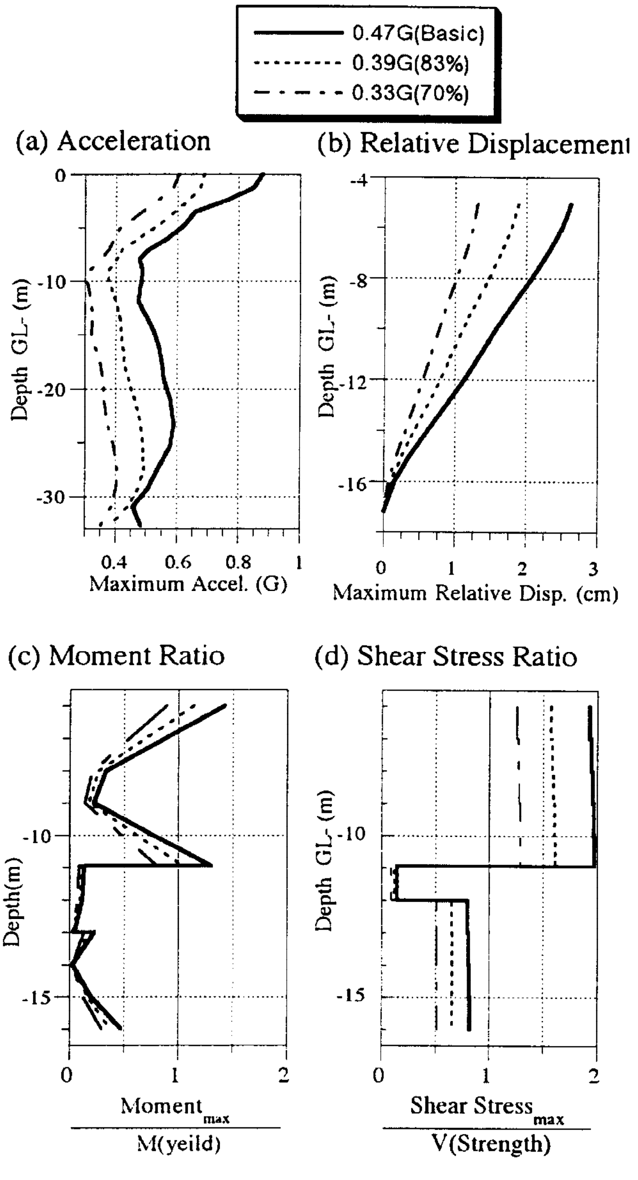


Fig.-10 Effect of intensity of input motion

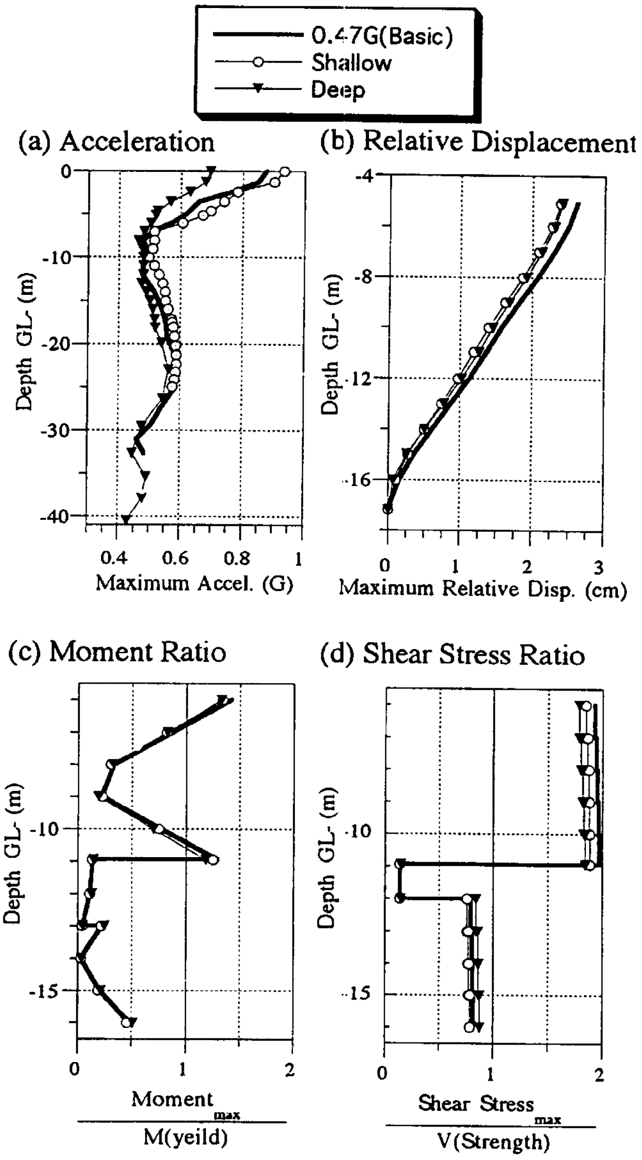


Fig.-11 Effect of amplification factor

be compared with previous simulation analysis designated as basic case.

Fig.-10(a) and (b) indicates the maximum acceleration and relative displacement distribution at the intermediate columns, respectively. When the intensity of input motion decrease, response acceleration and displacement naturally become smaller. In Case-70%, the maximum relative displacement decreases to Basic case of 50% due to the nonlinearity of soils.

Fig.-10(c) and (d) indicate the damage factors of bending moment and shear stress, respectively. As well as acceleration and relative displacement, the damage factors decrease in proportion to intensity of input motion. In Case-70%, although the relative displacement decreases to 50% of Basic case, section force decreases to 70% approximately. This means that section forces are affected by not only relative displacement but also magnitude of response acceleration.

4.2 Effect of amplification factor of ground

To investigate the effect of amplification characteristics of ground, two case of which the depth under the structure was taken as a parameter. Shallow case has a half depth Basic case, and Deep case has twice of that.

Fig.-11(a) and (b) indicates the maximum acceleration and relative displacement distribution at the intermediate columns, respectively. The maximum accelerations of Shallow case are the largest among these cases, although the maximum relative displacements of Base case are the largest among these cases. However, the difference among these maximum accelerations at the depth under GL-7m is not so large.

Fig.-11(c) and (d) indicate the damage factors of bending moment and shear stress, respectively. Slight difference among three cases in shear stress ratio is found. It is thought that slight difference of relative displacement affects the shear stress under the condition of the almost same acceleration amplitude in the ground.

5. CONCLUSIONS

The results obtained from the presented analysis are as follows

1. The primary cause of the damage observed at the underground subway structures is large shear stress at intermediate columns. It is evaluated that the maximum shear stress attained to almost twice of the strength. Axial stress has enough margin against the design strength in this cross section.
2. The deformation of intermediate columns is quite complex and large comparing to walls, which lead to the collapse.
3. Shear stress and bending moment mainly arise due to horizontal excitation, axial stress has a relation to static condition.
4. Large stresses concentrate on intermediate columns of upper deck, which coincides with the damage observation. However, it is difficult to represent a second damage such as the cracks at B2F.

It should be noted that two-dimensional equivalent linear response analysis method incorporated with a conventional criteria of concrete structure has an applicability to estimate for a seismic behavior and strength capacity of underground structures. However, to represent precise responses of whole structure system including post collapse of elements, it is required to develop a three-dimensional seismic response analysis code with taking account of soil and concrete nonlinearities.

ACKNOWLEDGMENTS

The authors would like to thank the authority of Kobe city for coopeation and giving useful data concerning to the damage and design materials.

REFERENCES

Kobe city (1995). Earthquake damage of the underground subway system due to the Great Hanshin disaster

Irikura,K. , Aguirre,J. (1995). Preliminary analysis of non-linear site effects at Port Island vertical array station during the 1995 Hyogoken-Nanmbu Earthquake, JNDS Journal, Vol.16, pp.49-58

Japan Civil Engineering Society (1991). Standard specification for design and construction for concrete structures