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Paper No. 2151. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

DAMAGE TO DAIKAI SUBWAY STATION DURING THE 1995 HYOGOKEN-NUNBU EARTHQUAKE AND ITS INVESTIGATION

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ABSTRACT

A detailed reconnaissance survey was conducted at the Daikai subway station which is the first subway structure that completely collapsed due to an earthquake. A complete collapse occurred at the location of more than half of the center columns, which resulted in the failure and collapse of the ceiling slab, and settlement of the subsoils over the station by more than 2.5 m at maximum. Many diagonal cracks were also observed on the walls in the transverse direction. Judging from the damage pattern, a strong horizontal force was imposed on the structure from the surrounding subsoils. Investigation of the damage mechanism was made through analytical approach. It is concluded that ceiling slab failed by the lack of the load carrying capacity against shear at center column.

KEYWORDS

earthquake, subway station, underground structure, earthquake damage, damage investigation, dynamic analysis, static analysis, nonlinear analysis

INTRODUCTION

The Hyogoken-nanbu earthquake of January 17, 1995 caused severe damage to various structures. Among these, damage to the subway was one of the amazing event, because underground structures have been considered to be relatively safe from earthquake effects compared to structures above the ground, and no significant damage has been reported so far. Figure 1 shows the location of the damaged subway and damage patterns. The general damage pattern is damage to columns. As shown in Fig. 1, the Japan Meteorological Agency seismic intensity in this area was evaluated to be 7 or more, which is equivalent to MM seismic intensity scale of 10 or more.

In many design specifications for the underground line-shaped structures, aseismic design is not usually considered in the transverse direction. The reason for this is that the underground structures are assumed to follow the deformation of the ground during an earthquake and the apparent unit weight of the structure is much smaller than that of the subsoils. Earthquake resistance design of underground structures can be reviewed and evaluated from a study of this damage.

The authors implemented a detailed reconnaissance survey on the damage to the Daikai station in order to assist the investigation of the damage mechanism. Then, the damage mechanism found by the survey was verified through the analytical approach. Two analyses, which consist of dynamic response analysis for soil-structure system and static nonlinear analysis for structure system, were conducted. The results of the survey and the analysis are reported in this paper.

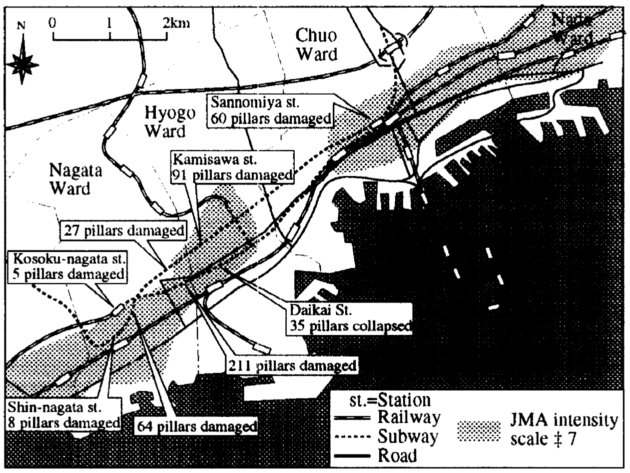


Fig. 1 Location of damaged subways and damage patterns.

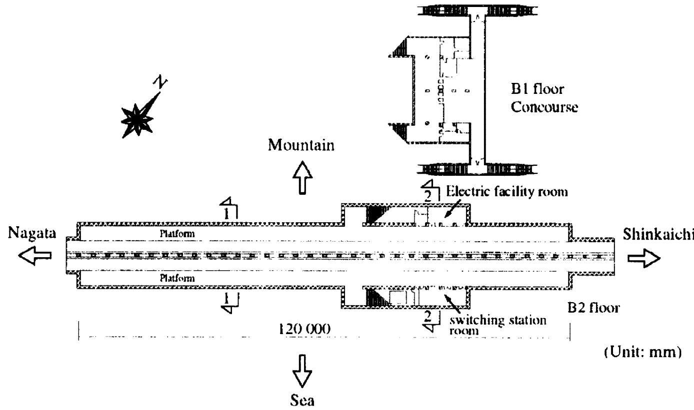


Fig. 2 Plan of Daikai station.

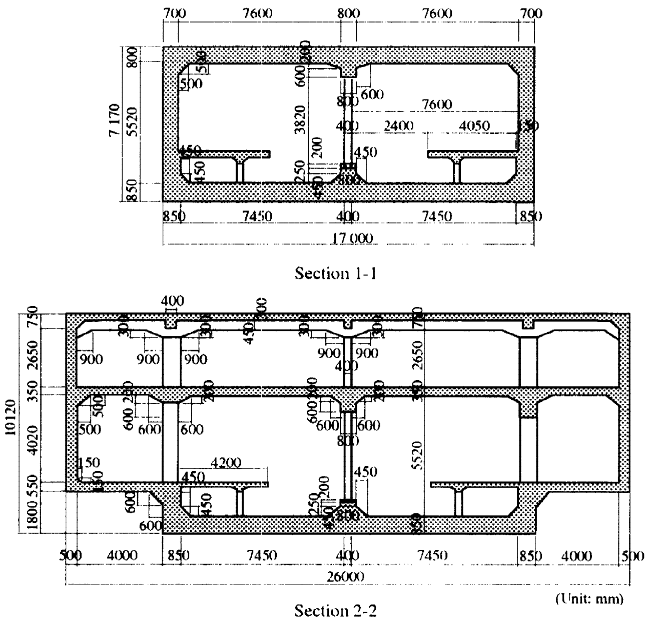


Fig. 3 Typical section of Daikai Station.

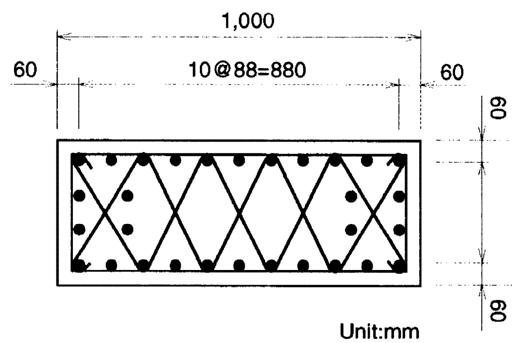
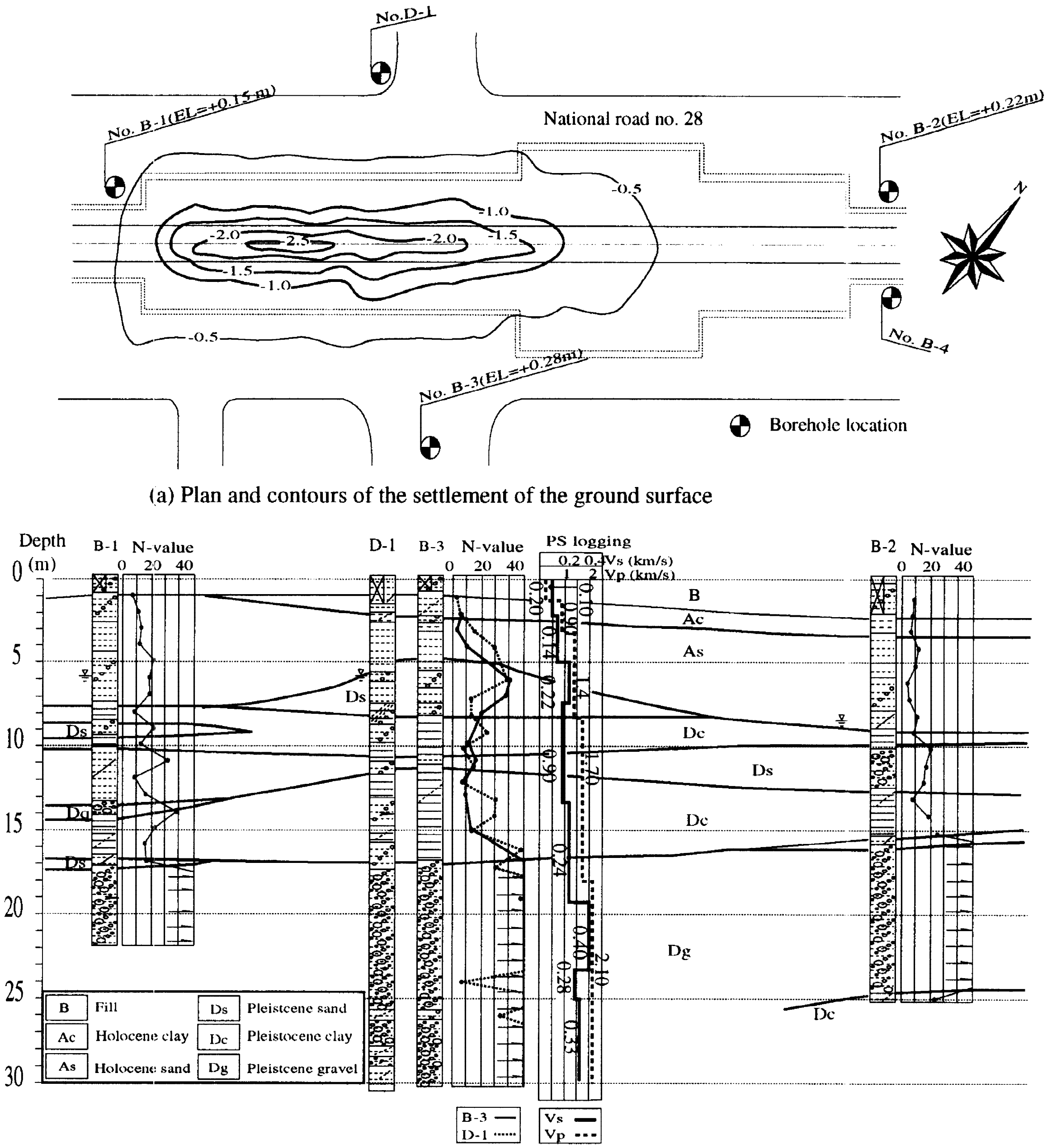


Fig. 4 Reinforcing steel arrangement drawings with respect to center column

DESIGN AND CONSTRUCTION OF DAIKAI STATION

The Daikai station was constructed right under the National road no. 28 in 1964 using the cut-and-cover method. Figures 2 and 3 show a plan and cross-section of the station. The station is two story reinforced concrete underground structure; B2 floor consists of platforms and rail lines and the B1 floor is a concourse with a ticket barrier. The thickness of the overburden soil is about 4.8 m at section 1-1 and 1.9 m at section 2-2 respectively. The B2 floor is mainly a box type frame structure with columns at the center, measuring 17 m wide and 7.17 m high in the outside dimension, and it is 120 m long in the longitudinal direction. The center column is 3.82m high and has a cross-section of 0.4 m × 1.0 m, and the distance between columns is 3.5 m.

The frame was designed based on a consideration of the weight of the overburden soil, lateral earth pressure, and weight of the frame under ordinary loading conditions, but the earthquake load was not taken into account, which was the normal method being used at that time of the design. Round steel bars with diameters from 16 to 25 mm were used as reinforcing for the walls and slabs, and 32 mm diameter bars were used in the center column. A transverse hoop (9 mm diameter) was placed at every 350 mm in the center column. Figure 4 shows the reinforcing steel arrangement of the center column. The allowable axial force was 4439 kN, which was slightly larger than design axial force of 4410kN. Two tests were made after the earthquake in order to evaluate the strength of the concrete. A strength of 37240 kN/m^2 with standard deviation of 2646 kN/m^2 was obtained by Schmidt hammer tests. Average compression strengths of the 8 cylindrical specimens taken from the center column was 39690 kN/m^2.



(a) Plan and contours of the settlement of the ground surface

(b) Soil profiles

Fig. 5 Settlement of the ground surface (numeral in (a)) and soil profile based on the borehole investigation after the earthquake. Boring No.4 is fill whose SPT-N value is shown in Fig. 6.

GEOLOGICAL SETTING AT DAIKAI STATION

The soil profiles by the investigation made after the earthquake in February, 1995 is shown in Fig. 5. The depth of the water table was between 6 and 8 meters. Referring to another source (Kobe city, 1980), the depth of the base (SPT-N value > 50) was deep on the west side and it consists of silty or clayey surface soil. It becomes shallow toward east; the depth at the Daikai station site is about 15 m and that at the Shinkaichi station is less than 10 m. In addition, sand becomes more predominant toward east.

Standard penetration and cone penetration tests were made near the structure in order to determine the properties of the fill material. Decomposed granite soil was used as the fill material. Fig. 6 compares SPT N-values with N value computed from the cone penetration tests. The N value of the fill is about 10 at all depths except near the bottom. Figure 7 shows strain dependent characteristics of undisturbed samples of the fill material obtained from dynamic deformation tests. The test was made under the isotropic initial stress states with confining pressures of 49 and 98kN/m^2, and an anisotropic initial stress state with principal stresses of 49 and 29.4 kN/m^2.

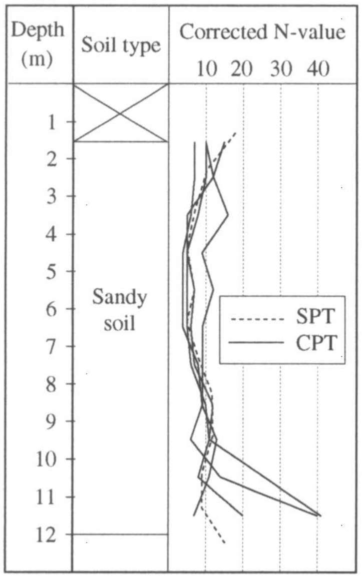


Fig. 6 SPT-N value of fill materials. SPT was conducted at No.4 site in Fig. 5.

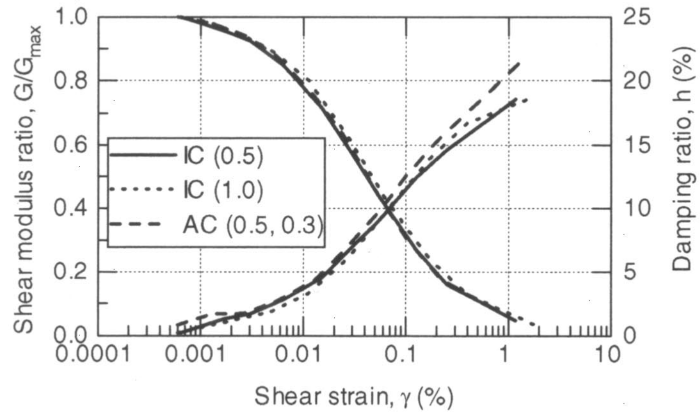


Fig. 7 Strain dependent characteristics of shear modulus and damping ratio obtained from dynamic deformation test on undisturbed samples, where IC denotes isotropically consolidated sample, AC denotes unisotropically consolidated sample, and numbers in the parenthesis are initial effective confining pressure in kgf/cm^2).

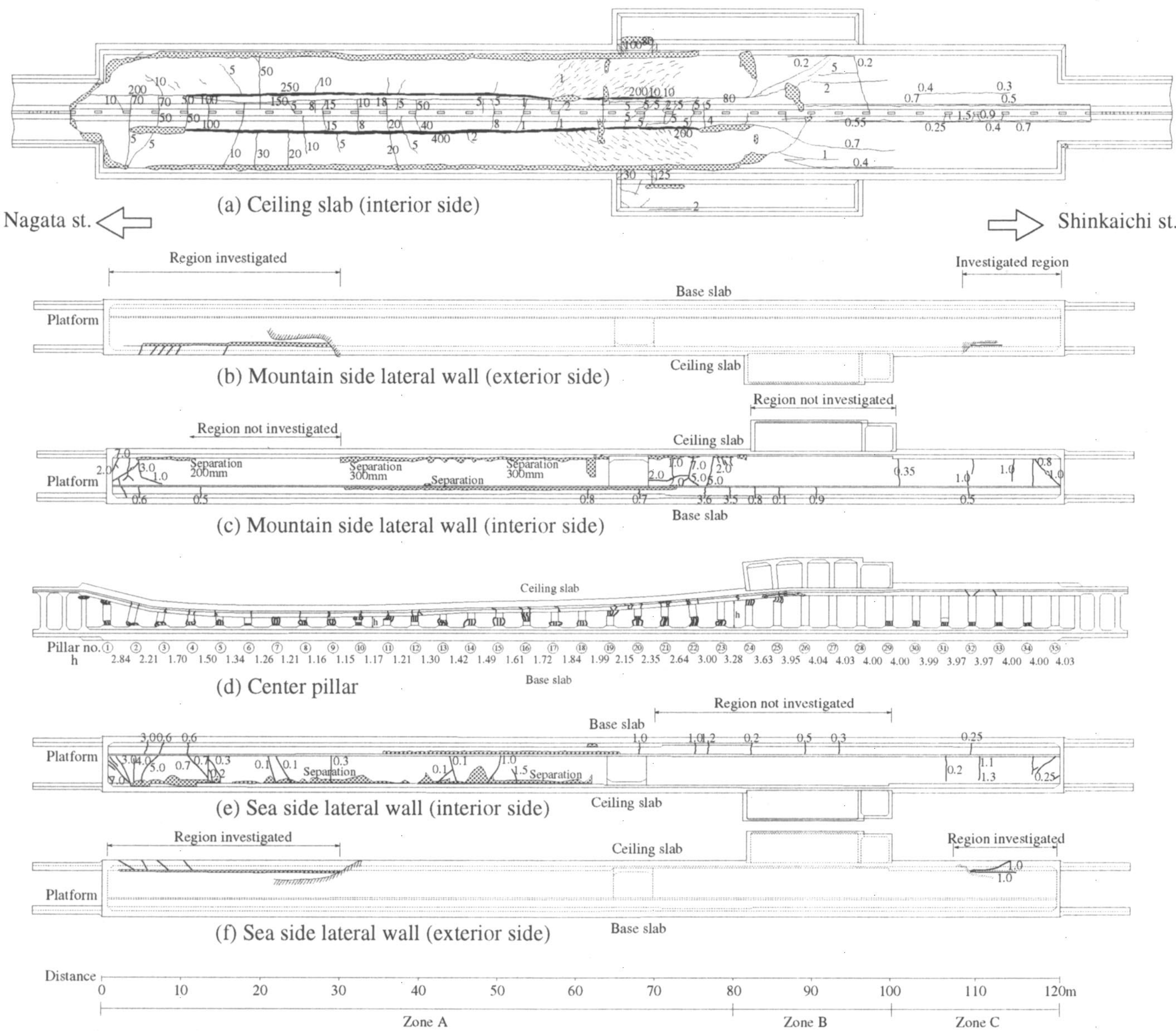


Fig. 8 Schematic figure indicating the damage. Here, *h* denotes measured clear height, circled number denotes column number. Numeral denotes crack width in mm.

Figure 8 shows schematic diagrams of the damage in the longitudinal direction. The ground of the National road no. 28 for a distance 90 m long and 23 m wide gradually settled up to more than 2.5 m maximum. The contour lines of the settlement of the road surface are shown in Fig. 5, which was measured on January 28. Referring to Figs.2 and 8, the station can be divided into 3 zones in the longitudinal direction along the station depending on the structural system: zones composed of columns 1-23, columns 24-29, and columns 30-35, which are designated zone A, B and C, respectively, hereafter. Zones A and C are a one story box frame structure whereas zone B has utility rooms adjacent to the platform as well as the B2 floor (concourse).

Damage was the most severe at zone A, in the Nagata side zone. Almost all of the center columns completely collapsed and the ceiling slab fell down. As a result, the original box frame structure distorted to an M-shaped section as shown in Fig. 9(a). Typical damage to the center columns is shown in Photo 1. The ceiling slab kinks and cracks 150 to 250 mm wide appeared in the longitudinal direction about 2.15 to 2.40 m from the center line of the columns. In addition, the separation of cover concrete was observed over almost the entire area near the haunch and the intersection between the lateral wall and ceiling slab. In zone B, as shown in Fig. 9(b), the collapse of the column occurred in the upper portion and reinforcing steel buckled into a symmetrical shape for columns 24 and 25. The upper longitudinal beam connecting the center columns was bent at a point between columns 25 and 26. The small separation of the corner concrete of the center columns is observed at the mountain side of upper portion and at sea side of lower portion, in columns 26, 27 and 28. Although the structural system in zone C was the same as that for zone A, damage was less in zone C compared with that in zone A. Figure 9(c) shows the damage to column 31 where the lower part of the center column is damaged and ceiling slab settled about 5cm.

In the lateral wall, separation of cover concrete was observed near both the top and bottom haunches. According to the investigation of the exterior surface, wide cracks in the longitudinal direction were observed along the intersection with the haunch. Under the platform, a significant separation of cover concrete was observed on the both side lateral walls. There are several walls in the transverse direction: both ends, electric facility room, switching station room, etc. Diagonal cracks typically shown in Fig. 9(d) and Photo 2 were observed in all the walls in the transverse direction.

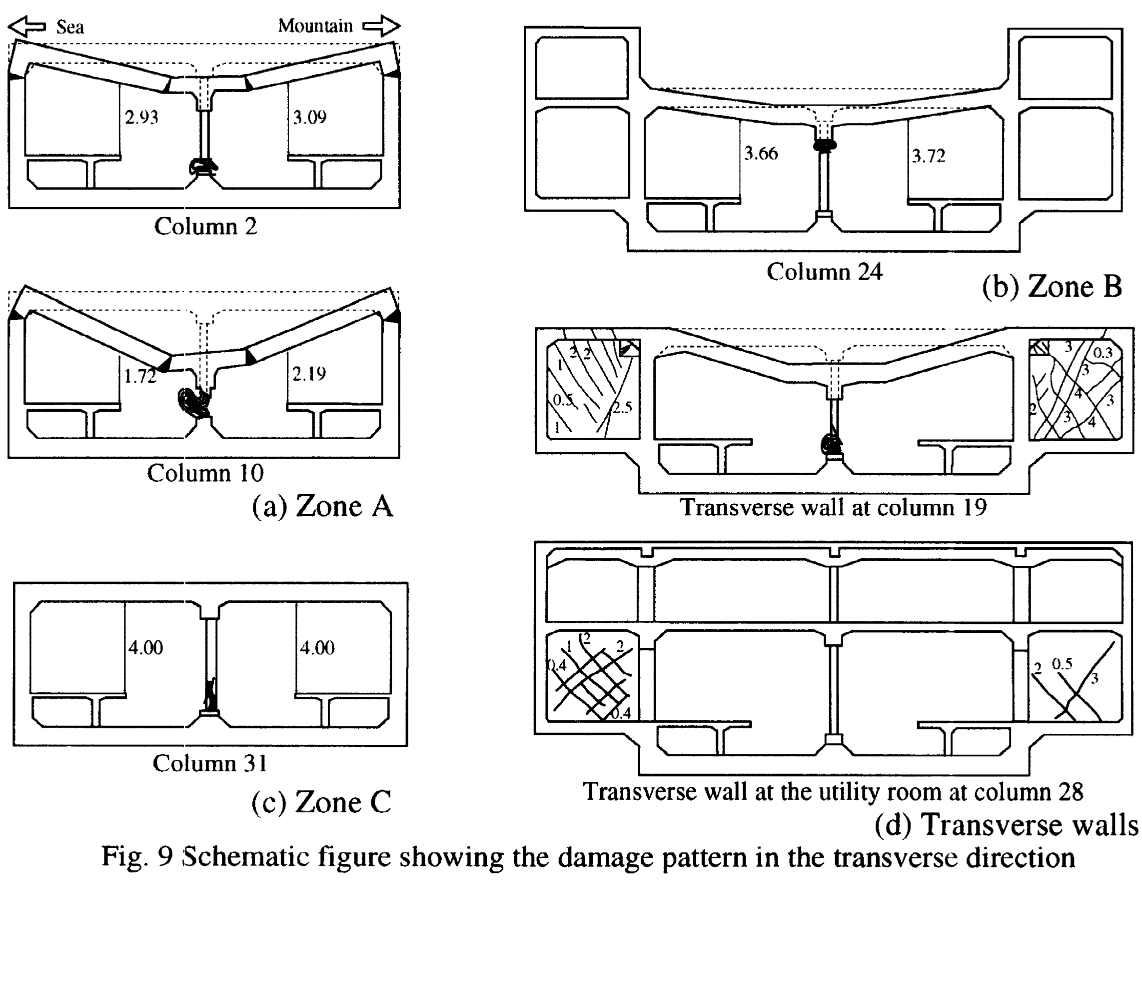


Fig. 9 Schematic figure showing the damage pattern in the transverse direction

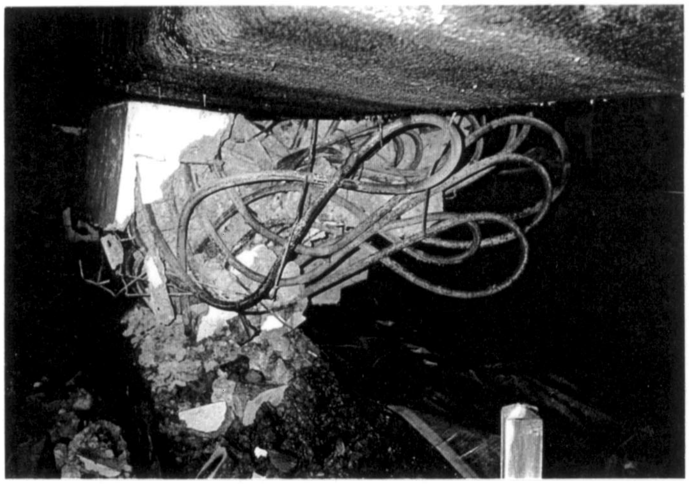


Photo.1 Collapse of no.10 column in Zone A

Photo 2 Cracks in the transverse wall in electric facility room

Based on the observation of the damage to these columns and walls, the mechanism of the damage of the collapsed column in zone A is evaluated to be as follows: 1) Due to strong horizontal force, the member reaches its strength under the combination of bending moment and shear force acting near the end of the column, which resulted in collapse of the end of the column. 2) The load carrying capacity of the box frame was reduced, and therefore excess relative horizontal displacement occurred.

ANALYSIS

Two step analyses were carried out each of which considered either nonlinear behavior of subsoils or that of the structure. At first step, in order to appropriately estimate the dynamic response of the structure during the earthquake, dynamic response analysis of soil-structure system was conducted using two dimensional finite element method considering nonlinearity of soil by equivalent linear method. Based on the behavior of the dynamic response, static nonlinear analysis was conducted to estimate the damage process of the frame. These analyses were conducted on one story box frame in A zone where damage was the most severe.

<u>Estimation of dynamic response of structure</u>

The analytical code Super-Flush was used. Both horizontal motion and vertical motions observed at KOBE university were applied as input motion. Dynamic response analysis was first carried out under the horizontal input motion. Then, using converged nonlinear characteristics (shear modulus and Poisson's ratio), dynamic response was calculated by linear analysis under both horizontal and vertical input motion.

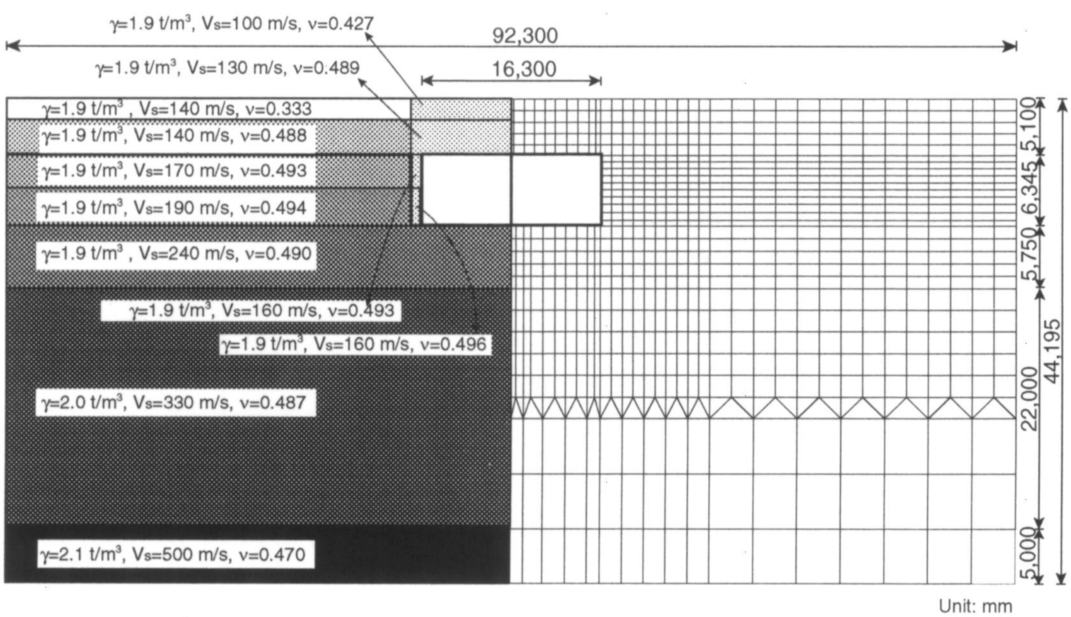


Fig. 10 Soil-structure system used in the dynamic response analysis.

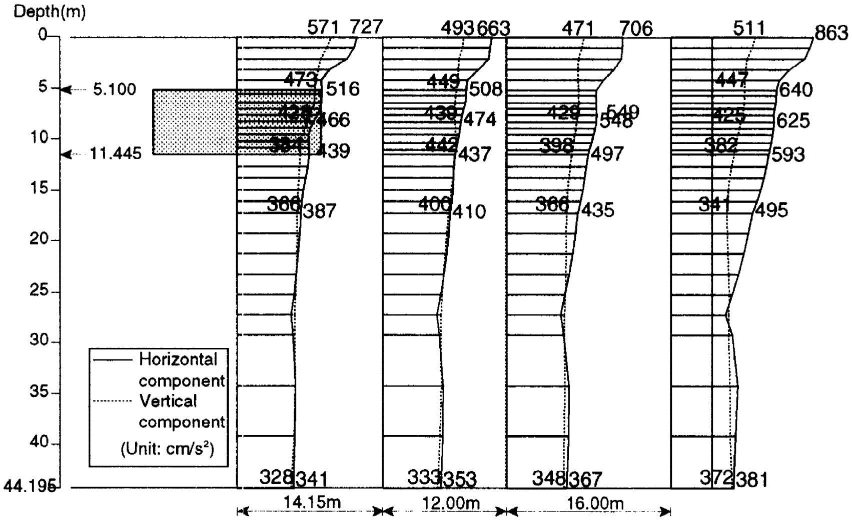


Fig. 11 Peak acceleration distributions

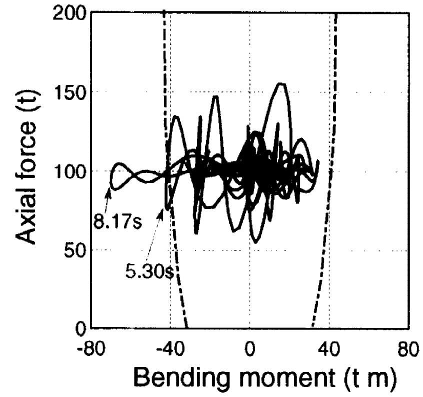


Fig. 12 Bending moment-axial force relationship at the bottom of the center column

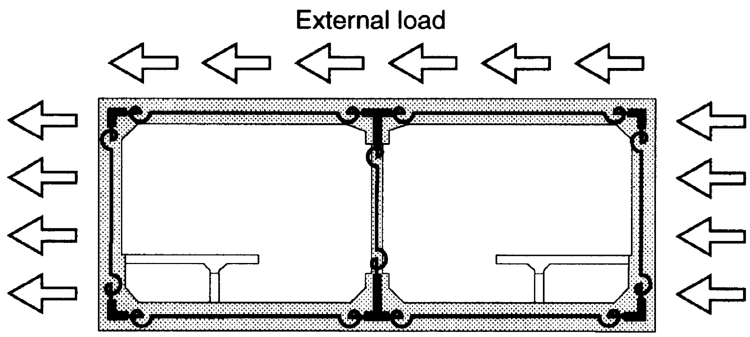


Fig. 13 structure system used in the static nonlinear analysis.

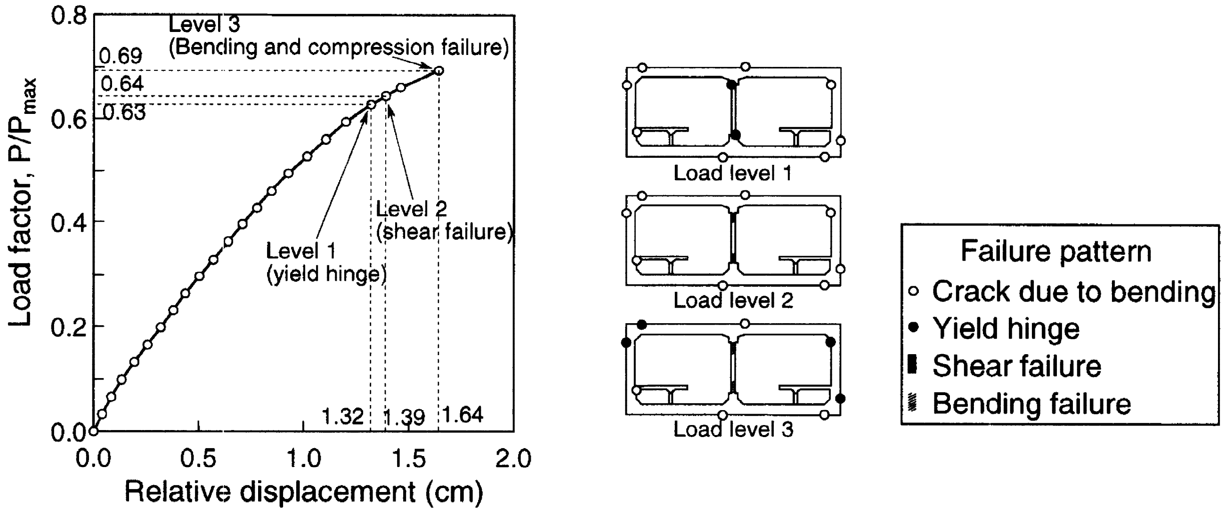


Fig. 14 The relationship between the applied load and the displacement of ceiling relative to base slab

Soil-structure model used in the first step analysis is shown in Fig. 10. Energy transmitting boundary and viscous boundary are used along the lateral and base boundaries, respectively. Ground is assumed to be horizontally layered and the depth of base whose shear wave velocity is 500 m/s is about 40 m. Material property used in the analysis, such as unit weight, initial Poisson's ratio and shear wave velocity, is also shown in Fig. 10. Empirical equation proposed by Yasuda and Yamaguchi (1985) are used as strain dependent characteristics of shear modulus and damping ratio of the subsoil except fill; those shown in Fig. 7 is used for fill material. Structural members are modeled to elastic beam elements considering rigid zone. The rigidity of each member is evaluated by considered the property of both concrete and reinforced bar.

Peak acceleration distributions in both horizontal and vertical directions are shown in Fig. 11 at a length of 14.15, 26.15 and 42.15 meters from the center of structure. Acceleration at the ground surface is more than 400 cm/s^2, which correspond to the JMA seismic intensity 7 around this area. The relationship between axial force and bending moment at the bottom of center column is shown in Fig. 12. Here, axial force is the sum of the dynamic response value and initial force under ordinary load condition. In the figure, the ultimate bending moment under given axial force is shown as a chained line. The maximum axial force (sum of forces under ordinary load and increment due to earthquake) is about 1520 kN (13300 kN/m^2), which is high in comparison with the strength of concrete. The ratio of the axial stress to the strength of concrete is about 0.36. Response bending moment at the bottom of the center column exceed the ultimate bending moment twice, at around 5.3 and 8.13 seconds, respectively. The same tendency is observed at the response of lateral wall, ceiling and base slab. The ductility of the center column is a very small value, 1.3. The ductility is defined as the ratio of the rotational angle of member at the ultimate bending moment to that at the yield of reinforcing bar with respect to the axial force at 5.3 seconds. Therefore, center column was possible to collapse just after the bending moment exceed the yield bending moment. Furthermore, the ratio of the shear strength of center column calculated by the Standard Specification of Reinforced Concrete (JSCE, 1991) to the converted shear force from the ultimate bending moment is less than 1.0. Therefore, center column was also possible to collapse under shear.

Estimation of damage process

Nonlinear static analysis was carried out. One story box frame is modeled as shown in Fig. 13. Each member is modeled into a beam element. Moment resisting joint is employed at the ends of each member, which is modeled to be a rotational spring. The spring is modeled to tri-linear model, whose three lines corresponds to the behavior until crack, until yield and from yield to ultimate stale, respectively. These behavior were calculated based on the total axial force and dynamic response force. Shear force and axial force obtained by the dynamic response analysis were applied to the location where it occurred as the external loads. Here, forces when the bending moment at the center column first exceed the ultimate moment are employed. They are divided into fifty and applied incrementally. These overburden soil load and static earth pressure are considered in the initial state.

The relationship between the applied load and the displacement of the ceiling slabs relative to base slab are shown in Fig. 14. The vertical axis (sum of the applied forces) is the normalized by the sum of the final force. Shear failure of center column occurred when the relative displacement was 1.39cm, just after the yield. Damages more than crack isn't recognized to the other members. After subsequent loading the load, bending combined with compression failure occurred right after the relative displacement and increase to 1.64 cm where normalized load was 0.69. Therefore, failure mechanism of one story box frame in Zone A is that ceiling slab failed the lack of the load carrying capacity of center column.

CONCLUSION

The Daikai station is the first subway structure that completely collapsed due to the earthquake. A detailed reconnaissance survey of the damage was made in order to determine the behavior of the station during the earthquake. Furthermore, the damage mechanism was verified by the nonlinear analyses. Based on these study , the mechanism of the collapse of the station is concluded to be as follows:

The B2 floor of the station was subjected to a strong horizontal load from the adjacent subsoil, which caused deformation of the box frame structure. In zone A where amount of wall in the transverse direction is small, center columns initially collapsed due to a combination of bending and shear resulting in the deformation of the box frame. Then, as a result of the relative displacement between the top and bottom of the columns, additional moment by gravity of the overburden soil became predominant resulting in the failure of the column. Since the walls in the transverse direction carry most of the horizontal force in zones B and C, damage to columns was much smaller compared with that in zone A. Instead, many diagonal cracks appeared on the walls in the transverse direction, such as walls at both ends of the station and walls in the utility rooms.

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