PUSHOVER ANALYSIS OF DAIKAI SUBWAY STATION DURING THE OSAKA-KOBE EARTHQUAKE IN 1995

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ABSTRACT:
Pushover analysis method is a simple and effective method in seismic analysis and design of structures. Because of the restriction of the surrounding foundation, the deformation shapes of underground structures and super-structures are different. The deformation of the underground structure under horizontal seismic loads appears a shear shape mainly, however, the super-structure bears a combined action of moments, shear forces and torques. The foundation-underground structure system bears body forces mainly under seismic loads. Interaction exists between the foundation and the underground structure, so the restriction of the surrounding foundation can not be neglected, which is different from super-structures. Therefore so far, pushover analysis methods applied to super-structures can not be used in seismic analysis and design of underground structures. So it is necessary to develop a pushover analysis method applied to underground structures considering the restriction of surrounding foundation, and put forward seismic analysis and design method for underground structures based on displacement. Thus the disadvantages of present analysis methods based on bearing force can be improved, and accordingly, seismic analysis and design of underground structures will get easier and more reasonable. A pushover analysis method for seismic analysis and design of underground structures has been presented. Solution methods of distributions of equivalent horizontal inertial acceleration and target displacement are introduced together with the implementation procedure. Pushover analysis is carried out on Daikai subway station, which was greatly destroyed in Osaka-Kobe earthquake in 1995. A whole range seismic response of the structure members from elastic status to failure is performed and the capacity curves are given. Also included are the sequence and distribution of the plastic hinges and the failure mechanism of the structure is analyzed further. The results indicate the structure first fails at the columns in compressive bending under strong horizontal and vertical seismic loads. Ultimately it results in the failure of the integral structure and large subsidence of the super-road. Besides, the results obtained through the pushover analysis method are in good agreement with the real seismic damages, so the validity and applicability of the method are verified.

KEYWORDS:
underground structure; seismic response; pushover analysis; soil-structure interaction

1. INTRODUCTION

In recent years, with the development and frequent seismic damages of underground structures, seismic analysis and design of underground structures have been receiving more and more attention. Especially in the great Osaka-Kobe earthquake in 1995, some subway stations and tunnels suffered severe damage, which is the first case of severe earthquake-induced damage to modern underground facilities. After this, scholars investigated the reasons causing seismic damage of underground structures, and hereby built analysis theories and put forward design methods. The efforts brought an upsurge to seismic research of underground structures, which turned into an important direction in the earthquake engineering field.

At present main elasto-plastic analysis methods include static increment analysis method, dynamic time history analysis method, and pushover analysis method and so on. Static increment analysis method has an easy operation, but it does not take the close relation between earthquake action and dynamic characteristic of the structure into account. Dynamic time history analysis method can calculate the internal forces and deformations of the structure in every minute of seismic loading with comparatively high accuracy. Then the sequence of frame cracking and yielding can be given and positions with concentrated stress and plastic deformations can be

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found, so failure mechanism and weak positions can be obtained further. But dynamic time history analysis will inevitably increase the complexity and uncertainty in the aspects of seismic loads input, artificial boundary setting and dynamic nonlinear parameters of soil. The computation workload by the method is large and time-consuming. The results obtained by the method are influenced by the selection of seismic wave loads. So it is difficult for dynamic time history analysis to be popular and widely used in common projects. Compared with the methods above, pushover analysis method is simpler and more effective. The method has clear conceptions and easy operations, and in addition seismic response of the structure can be evaluated effectively by the method, so it has been getting widely used and studied [1-3].

So far, the pushover methods applied to seismic analysis of superstructures can not be used in seismic analysis and design of underground structures because the rules and characteristics of seismic responses of the two are not absolutely the same. So it is necessary to develop a pushover method applied to underground structures considering the restriction of surrounding foundations, and put forward seismic analysis and design method for underground structures based on displacement. Thus the disadvantages of present methods based on bearing force can be improved, and accordingly, seismic analysis and design of underground structures will get easier and more reasonable [4-5].

A pushover analysis method for seismic analysis and design of underground structures has been presented [6-7]. Solution methods of distributions of equivalent horizontal inertial acceleration and target displacement are introduced together with the implementation procedure. Then the applicability of the pushover method is proved by changing some key parameters such as the burial depth of the structure and the relative stiffness between the structure and the soil. [6-7]

In this paper, pushover analysis is carried out on Daikai subway station, which was greatly destroyed in Osaka-Kobe earthquake in 1995. A whole range seismic response of the structure members from elastic status to failure is performed and the capacity curves are given. Also included are the sequence and distribution of the plastic hinges and further the failure mechanism of the structure is analyzed.

2. PUSHOVER ANALYSIS METHOD FOR SEISMIC ANALYSIS AND DESIGN OF UNDERGROUND STRUCTURES

2.1. Solution methods of distribution of horizontal loads

In the pushover analysis method for superstructures, monotonically increasing horizontal loads are put along the height of the structure until the target displacement is reached or the structure turns into mechanism. Then, weak positions of the structure and other nonlinear seismic response can be analyzed, so as to evaluate whether the structure and its deformation capacity under the potential seismic effect can meet the requirements of the design and its function. The common shapes of lateral forces include uniform distribution, inverse triangular distribution, and parabolic distribution.

During the pushover analysis for superstructures, distribution of horizontal loads and target displacement are two key parameters, so the paper pays great attention to them when developing the pushover analysis method for underground structures.

In pushover analysis methods for superstructures, distribution of lateral forces along the side of structure is the most important, while the loading manner takes the second place. The foundation-underground structure system bears body forces mainly under seismic loads. Dynamic interaction exists between the foundation and the underground structure, and the restriction of surrounding foundations can not be neglected. If lateral loads are applied directly on the underground structure only, interactions between the soil and structure will not be observed. At the same time, if lateral forces are applied along the side of the soil-structure analysis model, distribution and features of body forces in the system will not be reflected. So when pushover analysis of the soil-structure system is carried out, loading manner of lateral forces should reflect the distribution feature of body forces of the soil layers and underground structure under seismic loads well. What is more, the distribution of lateral forces should make the displacement solution reflect the displacement status of the soil layers and underground structure under seismic loads approximately.

To sum up the above discussion, when pushover analysis of the soil-structure system is carried out, we can apply monotonically increasing horizontal inertial body forces in the whole soil-structure system. To realize the idea, monotonically increasing equivalent horizontal inertial acceleration can be applied on the soil layers and structure according to their depths.

Distribution of horizontal inertial acceleration can be solved from one dimensional soil layer seismic response analysis methods. The following programs can be used, including equivalent linear programs such as SHAKE91 [8], EERA [9] and FEA soft wares such as MSC.Marc etc. Then the paper presents two solution methods of horizontal inertial acceleration from one dimensional soil layer seismic response results. In addition, because the seismic response of foundation-underground structure system shows the first order mode mainly, and the effect of high order modes is comparatively small, the paper also adopts the inverse triangular distribution of horizontal inertial acceleration.

(1) first distribution of horizontal inertial acceleration

The first distribution of horizontal inertial acceleration directly adopts the peak absolute accelerations of finite element model nodes of every soil layer under incident seismic loads, and the peak accelerations of every soil layer can be obtained through one dimensional soil layer seismic response analysis.

(2) second distribution of horizontal inertial acceleration

The second distribution of horizontal inertial acceleration is obtained from the maximum deformation status of soil layers. When the maximum deformation happens, each soil layer bears a maximum strain status, so it can be concluded that each soil layer bears a maximum shear stress status. Firstly we can solve the peak absolute values of shear stresses of each soil layer by programs referred to above. Set τ_{i-1} and τ_i to denote the maximum shear stresses at the top and bottom of the soil layer respectively, and also set ρ_i, h_i and a_i denote the density, height and horizontal equivalent inertial acceleration of the soil layer respectively. Especially when i=1, τ_0 equals to zero, which denotes the shear stress of free ground. So the horizontal equivalent inertial acceleration can be solved by the following equation.

$$a_i = \frac{\tau_i - \tau_{i-1}}{\rho_i h_i} \tag{1}$$

(3) Inverse triangular distribution of horizontal inertial acceleration

Seismic response of foundation-underground structure system shows the first order mode mainly, and the effect of high order modes is comparatively small. So based on the idea of inverse triangular distribution of lateral forces in the pushover analysis methods for superstructures, inverse triangular distribution of horizontal inertial acceleration can be adopted when pushover analysis of the soil-structure system is carried out.

Afterwards the distribution of equivalent horizontal inertial acceleration can be applied to the soil layers and underground structure in the static soil-structure analysis model according to their depths. Then elasto-plastic analysis can be carried out by means of static finite element analysis method until pushover analysis is finished or the underground structure is destroyed.

2.2. Solution method of target displacement

Target displacement in pushover analysis methods for superstructures refers to the maximum displacement that the structure can reach during an earthquake action, which commonly refers to the displacement of the top of the structure. It denotes the ultimate status of the structure after pushover analysis, and is also the terminal of pushover analysis. When pushover analysis of the soil-structure system is carried out, a reasonable, effective and convenient design parameter of ground motion as target displacement is necessary.

The peak ground displacement relative to bedrock which is called PGRD for short in the following denotes peak relative displacement between the ground and bedrock [5, 6, 10]. Available researches have proved that PGRD is a more effective and reasonable design parameter for seismic analysis and design of underground structures than peak ground acceleration which is called PGA for short in the following [5, 6, 10], so the parameter of PGRD can be regarded as the target displacement of the pushover analysis method for underground structures. Since the parameter of PGA is still used frequently in project design, the paper presents a solution method of PGRD from PGA. Firstly choose N seismic waves with the same PGA, and then adopt the one dimensional soil layer seismic response analysis methods to compute the seismic responses of free field under various seismic waves respectively, so the values of PGRD can be obtained. Further, expected value or a value with a certain guaranty rate of PGRD can be solved. Thus the parameters of PGRD and PGA can be connected.

In order to realize the purpose of setting PRRD as target displacement, during pushover analysis for soil-structure system, finite element model of soil-structure interaction system with additional free field should be built. The so-called finite element model of soil-structure interaction system with additional free field means that the soil-structure interaction model and corresponding soil column model (also called additional free field in the paper), are built in a same computation. The both are independent to each other, but can apply the same horizontal inertial acceleration at the same time. Pushover analysis is carried out both in soil-structure model and additional free field model using the horizontal inertial acceleration distribution which increases monotonically at the same ratio. When the relative displacement between the ground and bedrock reaches PGRD, pushover analysis can be finished. Now the deformations and internal forces of soil-structure model are the expected responses in the condition of PGA. Pushover analysis is carried out in soil-structure model and additional free field model at the same time, so we can obtain the following results: relationship of internal forces of the structure and relative displacement between the ground and bedrock of the additional free field model, relationship of internal forces and story drifts of the structure, and relationship of internal forces of the structure and relative displacement between the ground and bedrock of the soil-structure model, etc. What is more, horizontal inertial acceleration can be loaded continuously until the underground structure is destroyed entirely, if needed. Thus we can get the complete capacity curves of the structure for evaluating seismic performance of it in greater earthquake actions.

The discussion above indicates that when pushover analysis of the soil-structure system is carried out, the target displacement has already been predefined. And this is one difference between pushover analysis methods for underground structures and superstructures.

3. FINITE ELMENT MODEL AND PARAMETERS OF THE DAIKAI SUBWAY STATION

3.1. Introduction of the Daikai subway station and the ground condition

The Daikai subway station is a close frame structure using reinforced concrete. Figure 1 shows a typical cross section with a total width 17 m and a total height 7.17 m. The thickness of side walls is 0.7 m and the reinforcement ratio is 0.8%. The thicknesses of the top and bottom plates are 0.8 m and 0.85 m respectively, and the reinforcement ratio are both 1.0%. The size of the middle column is 0.4 m×1 m with a middle-to-middle space 3.5 m and the reinforcement ratio is 6.0% [11-13]. The control cross sections considered are A-D shown in Figure 1, in which A denotes the connection position of the column top and top beam, B denotes the connection position of the column bottom and bottom beam, C denotes the upper left (upper right) corner, and D denotes the down left (down right) corner. Physical properties of soil layers are illustrated in table I [11-13].

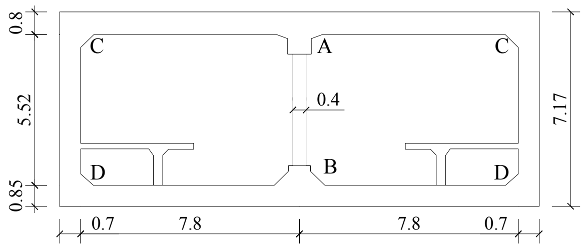


Fig. 1 Typical cross section of Daikai subway station (unit: m)

Table 1 Physical properties of soil layers

Soil layers	Depth /m	Density /(g·cm^{-3})	Shear wave velocities /(m·s^{-1})	Poisson ratio
Artificial filling	0-1.0	1.9	140	0.33
Sand in Pleistocene	1.0-5.1	1.9	140	0.32
Sand in Pleistocene	5.1-8.3	1.9	170	0.32
Clay in Pleistocene	8.3-11.4	1.9	190	0.40
Clay in Pleistocene	11.4-17.2	1.9	240	0.30
Sand in Pleistocene	17.2-22.2	2.0	330	0.26

3.2. Pushover analysis model and computational parameters

The Daikai subway station is a close frame structure using reinforced concrete. Figure 1 shows a typical cross section with a total width 17 m and a total height 7.17 m. The thickness of side walls is 0.7 m and the reinforcement ratio is 0.8%. The thicknesses of the top and bottom plates are 0.8 m and 0.85 m respectively, and the reinforcement ratios are both 1.0%. The size of the middle column is 0.4 m×1 m with a middle-to-middle space 3.5 m and the reinforcement ratio is 6.0%. The control cross sections considered are A-D shown in Figure

1, in which A denotes the connection position of the column top and top beam, B denotes the connection position of the column bottom and bottom beam, C denotes the up left (up right) corner, and D denotes the down left (down right) corner. Physical properties of soil layers are illustrated in table I.
Finite element model of pushover analysis on Daikai subway station is shown in Figure 2. In Figure 2, the left is the additional free field model and the right is the soil-structure interaction model. In both models, the top and bottom faces are the ground and the top of bedrock respectively, and the thickness of the models are 3.5m. The beam element based on fiber model is applied to simulate the structure and the hexahedron element is applied to simulate the soil. The beam element and the hexahedron element use a coupling connection between them.

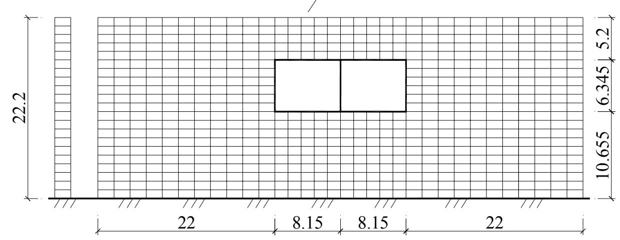


Fig. 2 Finite element model of soil-structure interaction system with additional free field (unit: m)

The reinforced concrete fiber model program THUFIBER [14] (Figure 3), developed by Department of Civil Engineering, Tsinghua University based on FEA software MSC.Marc is adopted, in order to simulate the nonlinear characteristics of reinforced concrete. The fiber model divides the reinforced concrete member into thirty-six concrete fibers and four steel fibers, and the deformations of all the fibers accord with the plane section assumption. In the THUFIBER program, elastic-perfectly plastic constitutive model is used for the steel fiber. The concrete adopts a hysteretic model which points to the origin in the unloading period, not taking the tensile strength of concrete into account. Lu *et al*. [14] proved that the bending-curvature relationship and relevant softening behavior of the reinforced concrete member in various axial forces can be simulated well by the program.

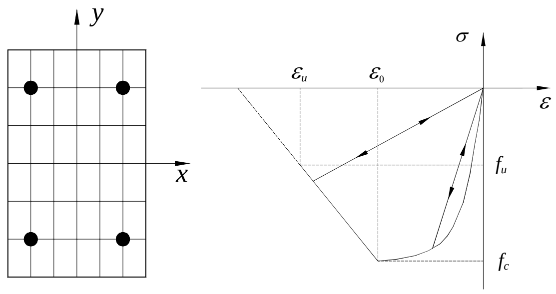


Fig. 3 Fiber model of reinforced concrete members and material constitutive model of concrete

In the numerical analysis, material parameters are set as the following: for concrete, initial elastic modulus E_0=30 GPa, peak compressive strength f_c=24 MPa, peak compressive strain ε_0=0.002, ultimate compressive strength f_u=19 MPa, ultimate compressive strain ε_u=0.0038; for steel, elastic modulus E_s=200 GPa, yield strength f_y=235 MPa. The values of the parameters are cited from available papers [11] and [12]。

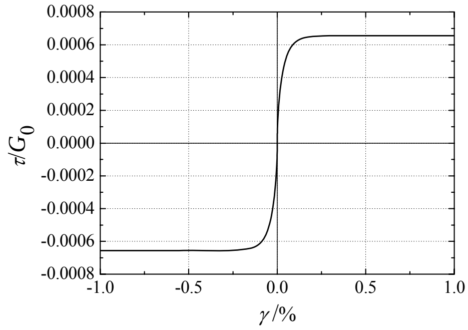


Fig. 4 Elasto-plastic stress- strain skeleton curve

To simulate the nonlinear characteristics of soil under the complicated loads of gravity and earthquake action, elasto-plastic model which obeys the isotropic hardening law is used. The stress-strain relationship is demonstrated in Figure 4 and the yield criterion adopts parabolic Mohr-Coulomb criterion. According to the references [8] and [15], set the soil element enter the plastic flow period until the plastic strain of the soil reaches 5.29×10^{-4}.

3.3. Distribution of equivalent horizontal inertial acceleration

The distributions of horizontal inertial acceleration can be obtained through the methods introduced above. The paper adopts the inverse triangular distribution.

It is indicated from several earthquake records that the Osaka-Kobe earthquake is a shallow earthquake. The vertical vibration is strong and the rate between the vertical part and horizontal part varies from 0.4 to 1.4. The author takes the influence of the vertical vibration into account when pushover analysis is performed. Set $a_v=\alpha a_p$, in which a_v and a_p denote the vertical and horizontal inertia accelerations respectively. According to the real earthquake record and the Code for Seismic Design of Buildings in China (GB50011-2001), set α=0.65. The direction of the vertical inertia acceleration points downwards and the accelerations are put on all available soil and structure elements with the same manner as the horizontal inertia accelerations.

4. RESULTS OF PUSHOVER ANALYSIS ON THE DAIKAI SUBWAY STATION

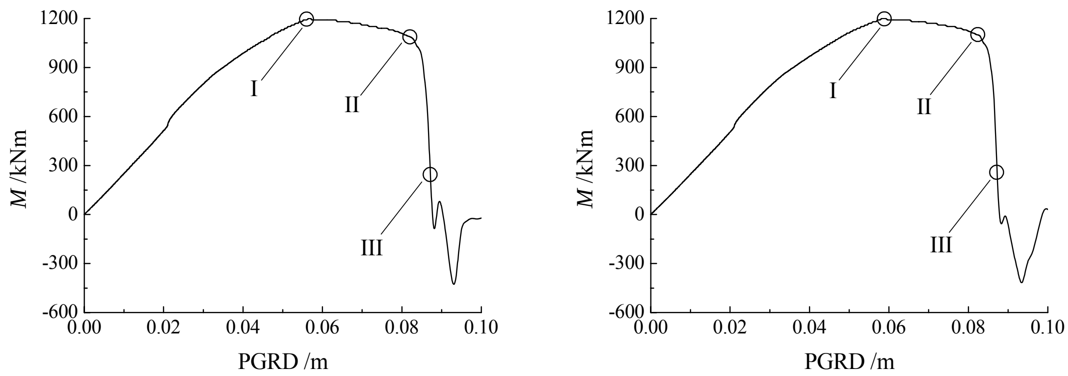
4.1. Capacity curves of the structure

Figure 5 shows the capacity curves of the column top, column bottom, the upper left (upper right) corner and the down left (down right) corner. The positions of the cross sections are illustrated in Figure 1. In the figures, the abscissa denotes the relative displacement between the ground and bedrock, and the ordinate denotes the moment of the corresponding cross section.

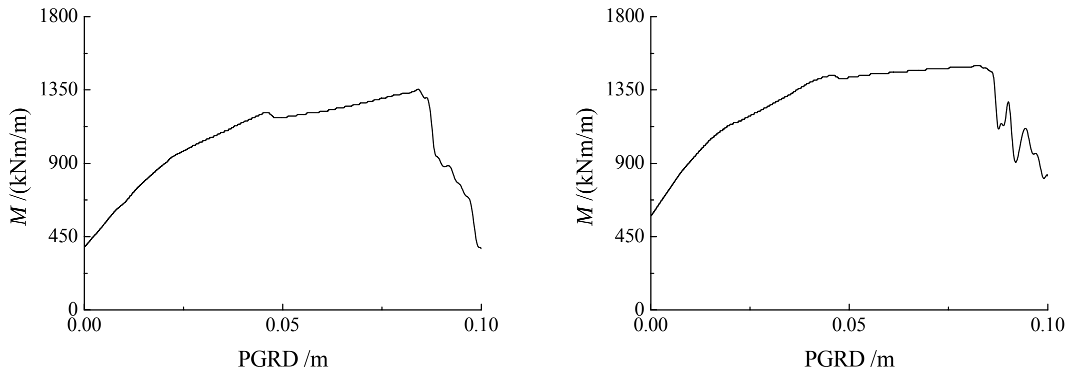
The capacity curves of the column top and column bottom are very close, so it is indicated that the moments the column top and column bottom bear under earthquake action are very close. It is also indicated that the column show a clearly brittle damage when the horizontal loads increase to a degree. The reinforcement ratio of the column is 6%, so the phenomenon is consistent with the real structure method. In Figure 5(a) and (b), the points of I, II and III are marked. At the point I, the top and bottom of the columns reach the ultimate bearing capacity. At this moment, the values of a_p and a_v are 0.600 g and 0.390 g respectively, and the story drift is 0.025m corresponding to an approximate story drift angle 1/252.

At the point II, the bearing forces of the column top and column bottom fall down rapidly. At this moment, the values of a_p and a_v are 0.723 g and 0.470 g respectively, and the story drift is 0.048 m corresponding to an approximate story drift angle 1/133. At the point III, the column top and column bottom break. At this moment, the values of a_p and a_v are 0.733 g and 0.476 g respectively, and the story drift is 0.074m corresponding to an approximate story drift angle 1/86. As to the upper and down corner, after the column loses bearing force, the internal forces of the positions fluctuate because of the re-distribution of the loads.

Also, the other capacity curves such as the relationship between the internal force and the story drift, the relationship between the internal force and the relative displacement between the ground and bedrock. Besides, except for the four cross sections selected in the paper, capacity curves of any cross sections of the structure can be obtained, so as to evaluate the seismic responses of the positions.



(a) top of the middle column (b) bottom of the middle column



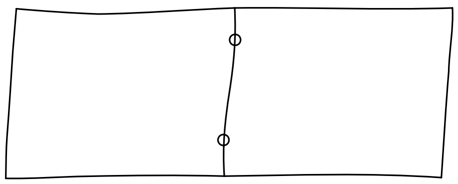
(c) up left (right) corner (d) down left (right) corner

Fig. 5 Capacity curves of appointed cross sections

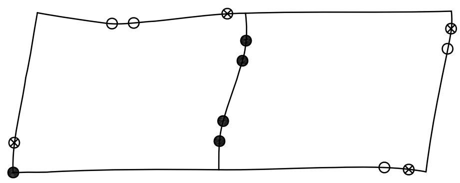
4.2. The development of the deformation and the plastic hinges

Figure 6 shows the deformation and the distribution of the plastic hinges in various periods. The deformation of the structure shows a shear shape and the plastic hinges firstly appear at the joint of the column and the upper beam and the joint of the column and the bottom beam. Figure 6(b) corresponds to the point II in Figure 5. At this moment, the plastic hinges of the column top and bottom are serious, and the plastic hinges appear on the side wall, upper plate and bottom plate.

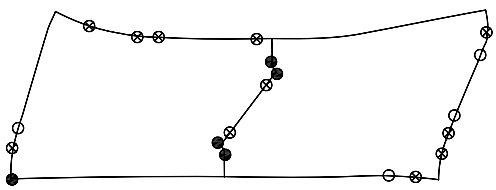
Figure 6(c) corresponds to the point III on the capacity curve of the column. At this moment, the joint of the column top and upper beam and the joint of the column and the bottom beam both break, and the plastic hinges increase on the left and right side walls. The real seismic damage record is shown in Figure 7(a) and great damages happen to the column top and bottom, so it is indicated that the numerical results and the real seismic damages are in good agreement.



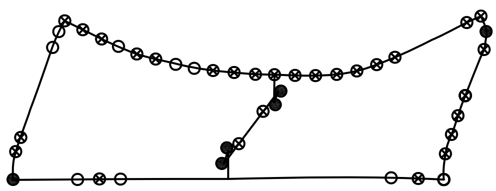
(a) the plastic hinges began to appear



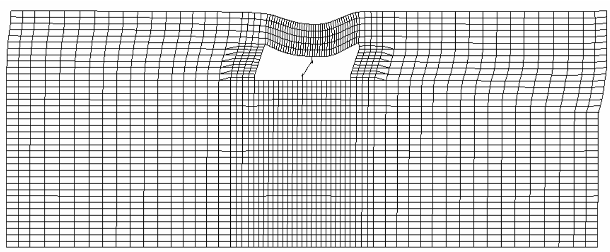
(b) bearing forces of column dropped quickly



(c) the column broke



(d) loads increasing after the column broke



(e) the deformation of the soil-structure system corresponding to figure (d)

Fig. 6 Deformation shapes and sequence and distribution of plastic hinges in different phases

After the columns break, pushover analysis on the structure can still be carried out. Figure 6(d) corresponds to the moment of a_p=0.744 g、a_v=0.484 g, and the story drift angle reaches 1/4.6. In fact the structure can be considered as collapsed now. The vertical displacement of the middle point of the upper plate relative to the original time reaches 1.9 m, so big subsidence happens to the road. The deformation sketch map of the soil-structure interaction system at this time is shown in Figure 6(e), and Figure 7(b) shows the real subsidence of the road in 1995. So it is indicated that the numerical results and the real seismic damages are in good agreement.

(a) The real seismic damage of the Daikai station (b) The subsidence of the road upon the Daikai station

Fig. 7 Seismic damage of Osaka-Kobe earthquake in 1995

4.3. The seismic failure mechanism analysis of the Daikai station

Firstly from an angle of construction method, the Daikai station applied the open excavation。The soil on the upper plate is artificial and the surrounded earth is also disturbed by the construction. So the soil referred to above is very easy to get loose and the loads the column and upper plate bear increase. The axial forces the column bear under strong seismic loads increase again under strong vertical seismic loads, so the ultimate bearing moments reduce rapidly with relatively strong axial forces. By the strong horizontal seismic loads at the same time, the column top and bottom bear great moments, so the plastic hinges will appear at an early time. With a high axial force and great moment, the column top and bottom will develop further, and finally the positions break. Presently the upper plate changes from a continuous beam to a beam of two rigid sides and no middle support. Now the two sides and middle position of the upper plate bear big moments, and serious hinges will occur on the positions under strong seismic loads. So great deflection will occur on the upper plate along the vertical direction, and the great subsidence also occurs.

The Daikai station is built in 1964. The original design did not take seismic design into account, so the structure tends to be dangerous under seismic loads. For example, the size of the column is 0.4 m×1 m, which is a little smaller than those at present. Under strong horizontal and vertical seismic loads, the internal forces of the column exceed the utmost bearing forces the ultimate bearing capacity and break, and the break of the columns result in the collapse of the whole structure. So it is indicated that the columns should be considered as key points when seismic design of the subway structure is carried out.

To sum up, it is concluded from the results that the factors below result in the great seismic damage of the Daikai station together: strong horizontal and vertical seismic loads, open excavation, no seismic design of the structure, etc.

The real seismic records of the Daikai station and the ground nearby are lacking, and the studies on the Daikai station presently apply the acceleration records in the other areas in Kobe, which may differ greatly from the real seismic records. Combining the numerical results and the real seismic damages, the author concludes that the peak ground acceleration of the Daikai station and the surrounded foundation may be in an area of 0.6 g-0.8 g.

5. CONCLUSIONS

Pushover analysis on the Daikai station is carried out in the paper, and the static elasto-plastic method is used to study the seismic response and failure mechanism of the structure. The following conclusions can be obtained:

(1) By the strong horizontal and vertical seismic loads together, the column loses bearing force. The upper plate bears big loads because of the break of the column top and bottom. So many serious plastic hinges occur

on the upper plate, and finally result in the collapse of the structure and big subsidence of the upper road.

(2) The break of the columns result in the collapse of the whole structure, so it is indicated that the columns should be considered as key points when seismic design of the subway structure is carried out.

(3) It is concluded from the results that the factors below result in the great seismic damage of the Daikai station together: strong horizontal and vertical seismic loads, open excavation, no seismic design of the structure, etc.

(4) Combining the numerical results and the real seismic damages, the author concludes that the peak ground acceleration of the Daikai station and the surrounding foundation may be in an area of 0.6 g-0.8 g.

At the same time, it is indicated that the pushover analysis results are in good agreement with the real seismic damages, such as the break of columns and the subsidence of the road. So the reasonability and applicability of the pushover analysis method for underground structures are verified by the study in the paper.

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