

Damage Analysis of a Two-Span Subway Station Under Earthquake Loading

Behzad Shokati Beyrag¹, Rashid Hajivand Dastgerdi², Alireza Sharafi³, Amin Emadi⁴

¹ Graduate Student, Azad university of Marand, Marand, Iran

² Graduate Student, Amirkabir University of Technology, Tehran, Iran

³ PhD Candidate, Azad University, Science and Research Branch, Tehran, Iran

⁴ SMEC Engineering Services, Sydney, Australia

Abstract— In 1995 during Kobe earthquake in Japan, the Daikai subway station damaged severely and attracted attention significantly on seismic behavior of underground structures. Many researchers did simulation and experimental tests to find exact reason for the failure of the Daikai station. In this study we built three-dimensional models to simulate the failure process and investigate the cause of collapse. There are two models built using ABAQUS software, in the first model we consider nonlinear behavior of concrete using CDP model and equivalent linear model for soil to capture precise dynamic behavior of soil during excitation to evaluate the station during seismic motion. We find out column is the most damaged and critical parts in this station to start the collapse. Then in second model the bearing capacity of the column was investigated and the Axial-moment and shear forces histories were obtained. As a result, the limited behavior of the column is the main reason to start failure in column and then the whole structure. We did a total examination on how the Daikai station collapsed and the structural weakness of the structure was found to be limited shear capacity of the middle column.

Keywords: The Daikai station, Damage Analysis, columns, Seismic motion, Numerical analysis

1. INTRODUCTION

Underground structures are among safest structures during earthquake and until last decade's no seismic considerations were considered in design of such structures. The Daikai station collapsed during Kobe earthquake in 1995 in Japan, it was a strong earthquake with moment magnitude of 6.9 and last for 20 seconds. The Daikai station's collapse got attention world widely to investigate its performance and cause of failure during this earthquake. Results of these investigations help to reconsider underground structures design details. The Kobe underground railway built by cut and cover methods and commenced on 1964. The Daikai station which is part of Kobe rapid transit railway, consists two-track line. The main station is a reinforced concrete structure that has mid-span columns to support the ceilings. According to the reports, the middle columns collapsed in central parts and the roof dropped onto the station, owing that, 2.5 m settlement induced on ground surface. The Daikai station failure effects on settlements of an area of 20*100 m. and it was the only damage that found in Kobe for underground structures. This was the first seriously damage that happened in an underground modern structure.

Since 1995 many researchers investigate the Daikai station and try to simulate the cause of damages and influence parameters on its response. In 1996 Yamato et al [1] built a finite element model to realize the main reason of Daikai station failure. They used equivalent linear model for modeling the dynamic behavior of soil and linear elastic model for structure. Their results showed that the shear force and moment was the reason for destruction of column and then the rest of the central part of the station. In 1997, Samanta [2] showed that the walls and slabs start yielding after column shear failure. In 2000, Iwatate [4] using shaking table tests indicates that, the column failed firstly due to low shear bearing capacity and after that, the scaled model of station collapsed. In 2005 Huo et al [8] built a model with similar condition to what Yasuo et al did in 1996, and showed that the stiffness of structures has an important role in failure on the station during seismic motion. The lower stiffness of the structure, the more damage induce. And since the Daikai station was designed for static loads, its reinforcements were insufficient especially for the column, and it caused shear failure. These were the base of other studies on the Daikai station failure during Kobe earthquake. After all of these researches lack of a perfect modelling and precise examination of the soil-structure system for the Daikai station can be observed.

In this study, we investigated the reason of the Daikai station collapse precisely, to aim that, two 3D dynamic numerical models were built to see the main part that started the failure process, and in second model we focus on that part and evaluated its behavior to find the reason. For increase the accuracy of the models we used two nonlinear complex constitutive model for both soil and concrete.

2. METHODS

2.1. Daikai Station

The Daikai station was built using cut and cover method and constructed in three different sections: The success section, central section, and subway tunnels section. The central section collapsed during excitation and was located 4.8 m below the surface ground. It was a two span box shape concrete structure with 17 m width and 7.17 height. The middle columns were rectangular and spaced 3.5 m in longitudinal direction. More detail of the station [11] can be observed in figure 1.

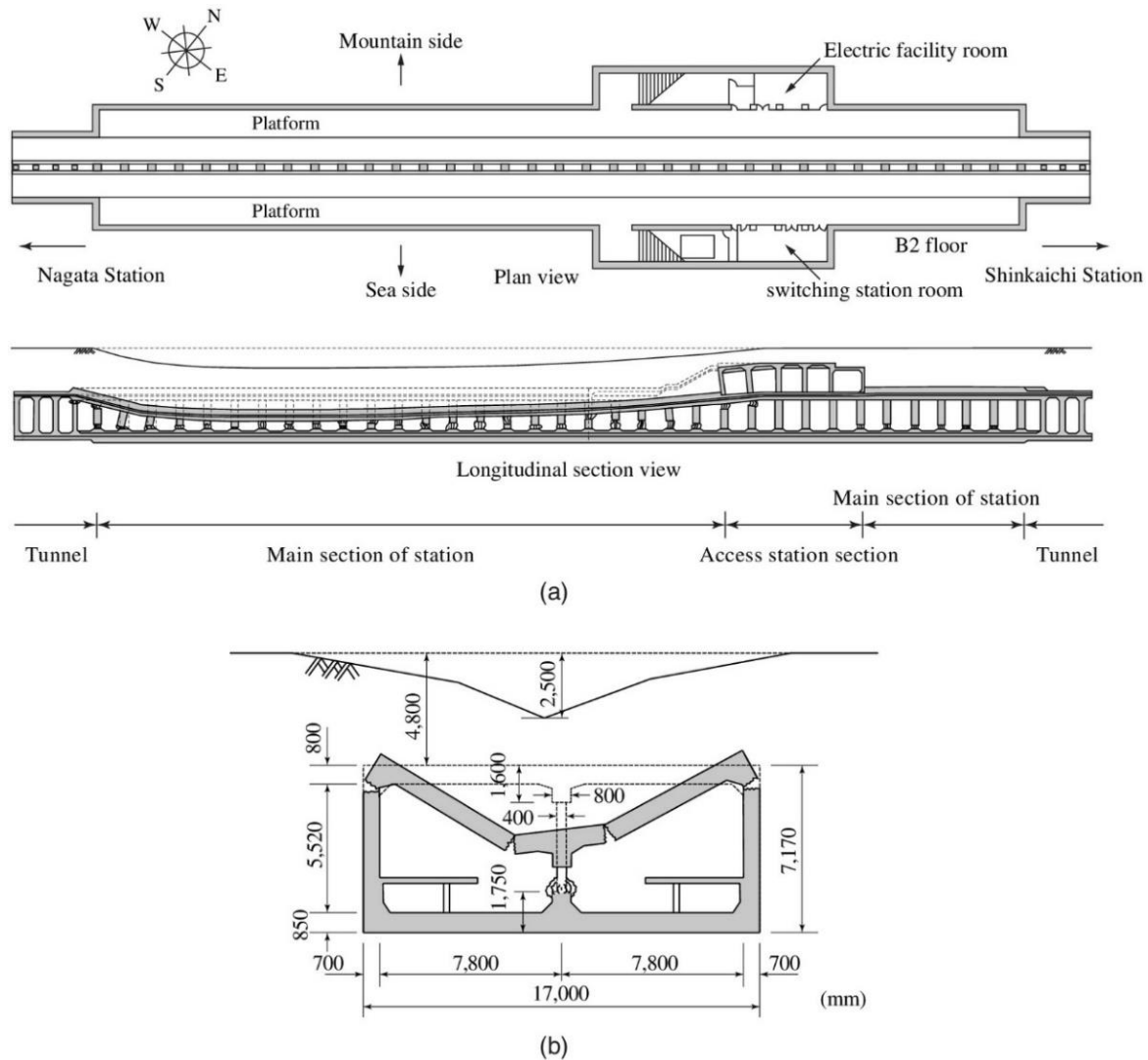


Figure 1. (a) Longitudinal damage patterns to the Daikai Station (b) Details of damaged section (unit: m) [11].

2.2. Numerical Model and Parameters

A numerical model has been built using Abaqus software to simulate the damage process of the daikai station's central part. To eliminate the impact of the lateral boundary, the width of the site was 200 m with semi-infinite elements in both sides to reduce the reflected wave effects, and the depth of the model was 40 m.

The discretization of both the soil and the concrete structure is done with C3D8R elements, as shown in Figure 5. Tied interaction is assumed between the subway station and the soil. The lateral boundaries are taken in the model as semi-infinite boundaries using CIN3D8 elements. In the model the distance from the structure to the lateral boundary is more than ten times the dimensions of the structure. The adequacy of the type of boundary and mesh size was verified by running a number of preliminary numerical tests where the lateral boundaries of the discretization were placed at different distances from the subway structure. The three-dimensional FEM model of subway station structure established based on Abaqus is shown in figure 2.

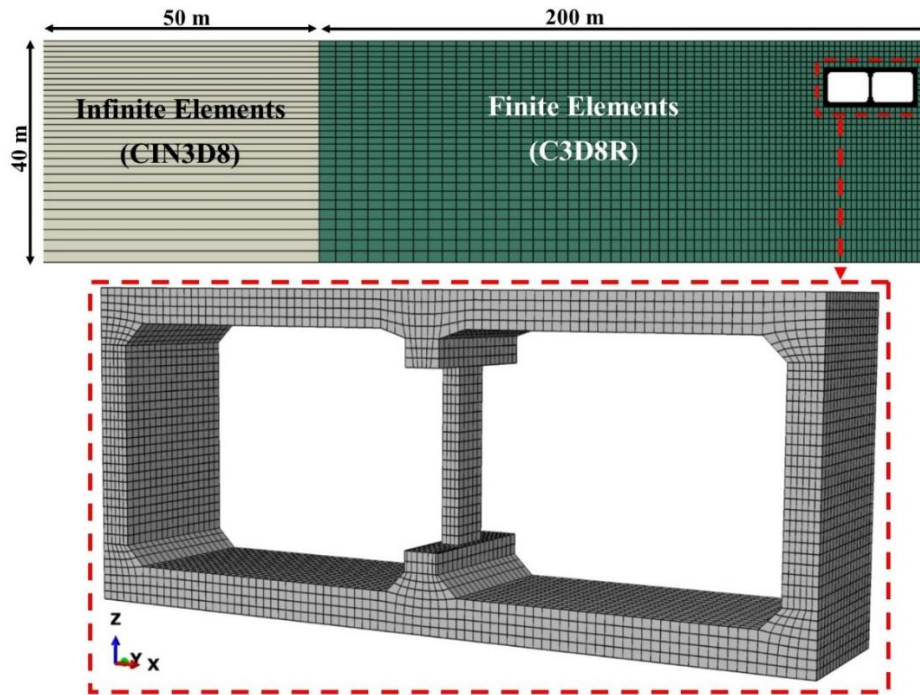


Figure 2. Finite element model for the soil-subway station interaction system

The Kobe wave, which was recorded at Port Island, was adopted as the input earthquake motion. The frequency content of the Kobe Wave was mainly below 10 Hz, and the dominant frequency ranged from 0.60 to 1.36 Hz based on the results of Fourier transformation. The accelerations were applied on the bedrock, which was 40 m away from the ground surface level. The ground motion applied at the bottom of the model is shown in Figure 3. As observed from the figure, the maximum acceleration in the horizontal direction was 0.4g and lasted for 20 second [12]. The vertical seismic motion has a minor effect on the seismic response of the subway station and as a result, it can be neglected for seismic analysis [13].

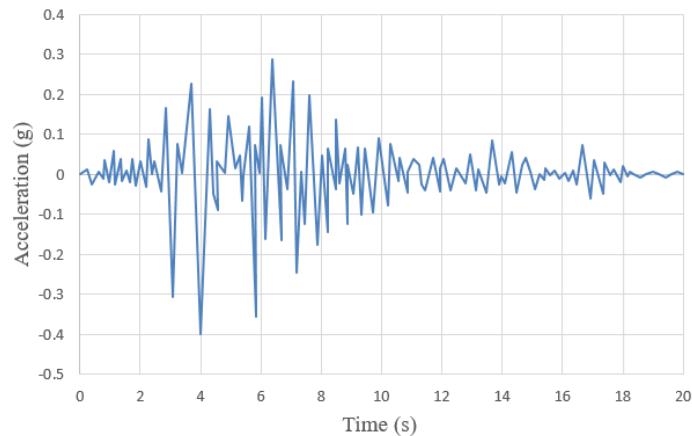


Figure 3. The input horizontal acceleration

3. MATERIALS

3.1. Concrete Materials

Concrete model CDP is one of the possible constitutive models to predict the behavior of concrete [14]. It describes the constitutive behavior of concrete by introducing scalar damage variables. Tensile and compressive response of concrete can be characterized by CDP in Figure 4.

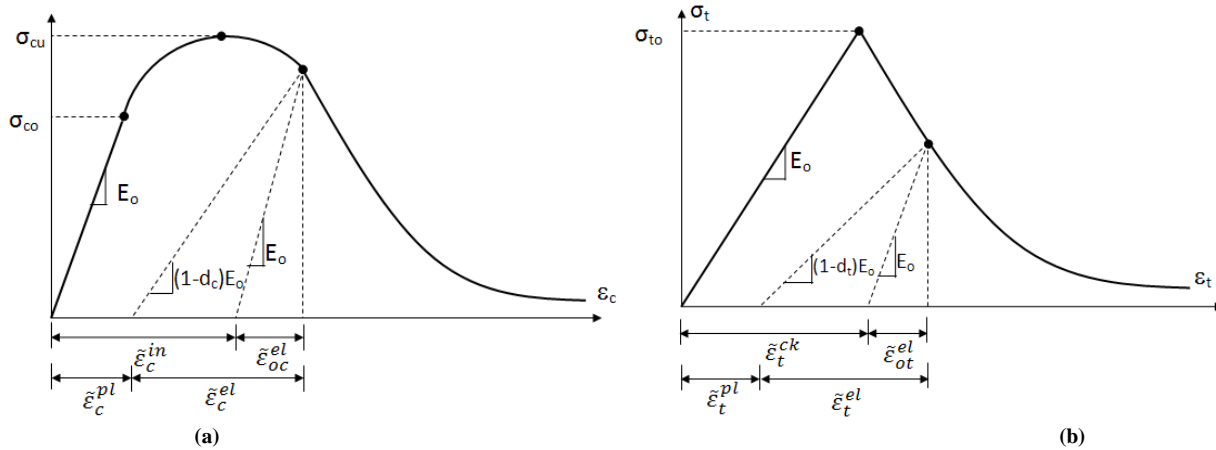


Figure 4. Behavior of concrete under axial compressive (a) and tension (b) strength [14]

As shown in Figure 4, the unloaded response of concrete specimen seems to be weakened because the elastic stiffness of the material appears to be damaged or degraded. The degradation of the elastic stiffness on the strain softening branch of the stress-strain curve is characterized by two damage variables, d_t and d_c , which can take values from zero to one. Zero represents the undamaged material where one represents total loss of strength. E_0 is the initial (undamaged) elastic stiffness of the material and ϵ_c^{pl} , ϵ_t^{pl} , ϵ_c^{in} , ϵ_t^{in} are compressive plastic strain, tensile plastic strain, compressive inelastic strain and tensile inelastic strain respectively. The stress-strain relations under uniaxial tension and compression are taken into account in Eq. (1) and Eq. (2).

$$\sigma_t = (1 - d_t) \cdot E_0 \cdot (\epsilon_t - \epsilon_t^{pl}) \quad (1)$$

$$\sigma_c = (1 - d_c) \cdot E_0 \cdot (\epsilon_c - \epsilon_c^{pl}) \quad (2)$$

Here in this study we used CDP for concrete structure of daikai station to calculate the failure process in the concrete. In table below the CDP parameters can be seen [15].

Table 1. Parameters used in CDP model for numerical calculations

Density (kg.m ⁻³)	Poisson's ratio	Young's modulus (MPa)	Compressive yield stress (MPa)	Tensile yield stress (MPa)
2500	0.2	3.35 E4	39.8	3.4

3.2. Dynamic Behavior of Soil

Study of dynamic properties of soil becomes essential for predicting the behavior of Structure during an earthquake in terms of earthquake resistant design. Soil properties primarily depend upon level of strain induced under the application of cyclic loading. At low strain soil behaves linearly whereas it behaves in a nonlinear manner at the higher strain. Laboratory soil tests for dynamic soil properties developed in the past three decades disclosed that soils change from a linear to nonlinear material as induced shear strain grows from 10^{-6} to 10^{-3} and finally reach failures at a strain larger than 10^{-2} .

3.3. Equivalent Linear Model of Soil

Up to the strain level of 10^{-3} , beyond which nonlinear behavior becomes prominent, soil is often simplified as a linear mass. An approximate analysis is carried out on soil applying "Equivalent Linear Model". A linear model expresses dynamic properties of soil that are independent of the strain amplitude [28]. It is generally useful for low strain levels. A hysteresis loop of a soil mass subjected to symmetric cyclic loading is proposed as shown in figure 5.

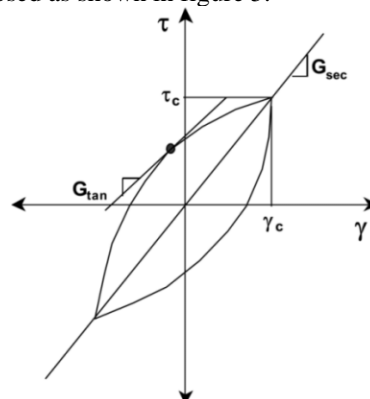


Figure 5. Equivalent linear model of soil with secant shear G_{sec} and Tangent shear modules G_{tan}

This hysteresis loop can be described in two ways: first, by the actual path of the loop itself, and second, by parameters that describe its general shape. The two important properties of the shape of a hysteresis loop are its inclination and its width. The inclination of the loop depends on the stiffness of soil, which can be described at any point during the loading process by the tangent shear modulus G_{tan} , that varies throughout the loading, Tangent modulus G_{tan} varies throughout the cycle of loading, but its average value over the entire loop can be approximated by the secant shear modulus as,

$$G_{sec} = \frac{\tau}{\gamma} \quad (3)$$

Where, τ and γ are the shear stress and shear strain amplitudes. Thus G_{sec} describes the general inclination of the hysteresis loop. The width of the hysteresis loop is related to the loop area which is in fact a measure of energy dissipation that can be conveniently described by the damping ratio as,

$$\xi = \frac{1}{2\pi} \cdot \frac{A_{loop}}{G_{sec} \gamma^2} \quad (4)$$

Where A_{loop} is the area under the hysteresis loop. The parameters G_{sec} and ξ are called equivalent linear soil parameters. As most commonly used methods of ground response analysis are based on this "Equivalent linear model", considerable effort has been made in the characterization of G_{sec} and ξ for different soil types. Equivalent linear model implies that strain will always returns to zero after cyclic loading and failure cannot occur. This method hence cannot be used in the problems involving permanent deformation and failure. In this study we choose linear equivalent model to capture the soil nonlinear dynamic response during Kobe earthquake. The parameters used in this model [16] can be seen in table 2.

Table 2. Physical properties of soil layers [1]

Soil type	depth/m	Density/kg m ³	Vs/m.s ⁻¹	Shear modulus/MPa	Young's modulus/MPa
Backfilled	2.20	1.80	188.0	62.03	180.03
Holocene clay	1.00	1.60	199.0	62.03	180.03
Holocene sand	5.80	1.80	183.0	62.03	180.03
Pleistocene clay	1.10	1.80	197.0	68.21	197.76
Pleistocene gravel	2.40	2.10	240.0	118.78	344.37
Pleistocene clay	4.75	1.80	228.0	91.53	265.38
Pleistocene gravel	>10	2.10	453.0	430.32	1247.93

Table 3. The nonlinear data of sand and clay [16]

$\gamma_{eff}(\%)$	$lg\gamma_{eff}$	attenuation coefficient of shear modulus		attenuation coefficient of damping	
		sand	clay	sand	clay
$\leq 1E-4$	-4.0	1.000	1.000	2.50	0.50
3.16E-4	-3.5	0.913	0.984	2.50	0.80
1 E-3	-3.0	0.761	0.934	2.50	1.70
3.16 E-3	-2.5	0.565	0.826	3.50	3.20
1.00E-2	-2.0	0.400	0.656	4.75	5.60
3.16 E-2	- 1.5	0.261	0.443	6.50	10.0
1 E-1	- 1.0	0.152	0.246	9.25	15.5
0.316	-0.5	0.076	0.115	13.8	21.0
1.00	0.0	0.037	0.049	20.0	24.6
3.16	0.5	0.013	0.049	26.0	24.6
≥ 10	1.0	0.004	0.049	29.0	24.6

4. RESULTS AND DISCUSSION

4.1. Failure Process

After applying the earthquake record to the bottom of the three-dimensional finite element model, the failure results of the station structure was obtained as shown in Figure 6. Values $d_t > 0$ means the onset of tensile failure in concrete and the value $d_t = 1$ means complete tensile degradation of concrete.

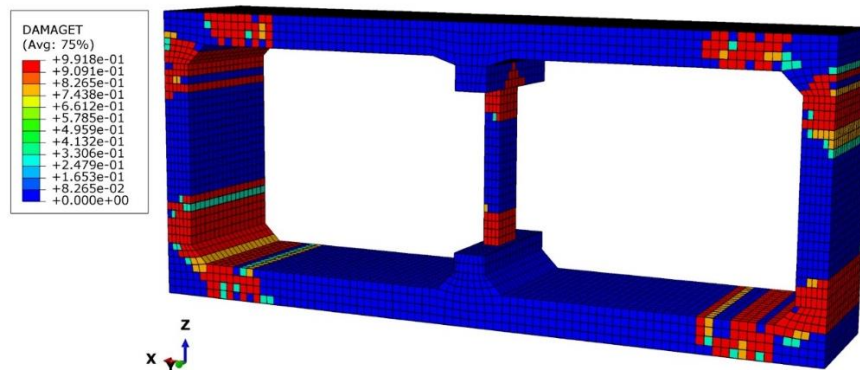


Figure 6. Seismic tension damage of the station

Based on the output observations of the numerical model through the excitation, it can be deduced that the destruction of the station was due to start of the failure of the middle column, and this result is consistent with the observations of the station failure at the site and the findings of other researchers [1, 2, 3, 13]. The failure of the column was on both sides of the connection of the column to the slabs. These columns were subjected to axial, bending and shear forces that change with time during the earthquake. And after the columns, the walls and slab failed onto the station. In many sources, column failures have been reported due to their low cross-section [24], which will be investigated in the second part of this study.

4.2. Displacement Responses

Horizontal displacement of the station structure (drift) also plays an important role in structural failure mode. Therefore, the history of horizontal displacement of the floor and ceiling of the station was taken for the right wall (approximately equal to the displacement of the top and bottom of the column), which can be seen in Figure 7.

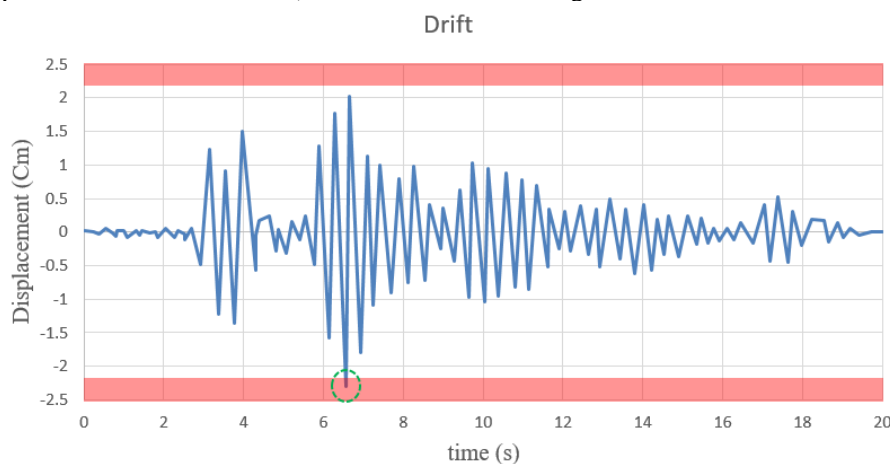


Figure 7. Drift time histories

Based on the suggested elastic interlayer displacement angle for the frame-shear wall reinforced concrete structure in the Chinese Code of seismic Design of Buildings (Code No. GB50011-2010) [17], the relative displacement between the base and the ceiling level of the subway station should be 22mm for this structure. As can be seen in figure 7, the maximum drift obtained 23 mm and happened in 6.6 seconds after the start of the Kobe earthquake excitation. Therefore, the station responded in plastic state as seen in figure 6.

4.3. Column Analysis

Force analysis of the middle column has been done to determine the mechanism of failure of the columns and for this purpose the same model was used and only for the structure, wire and shell elements were used as columns, slabs and walls, respectively. Linear elastic model was also considered for structural elements to evaluate the bearing capacity of the column during the earthquake history. Thus, the results of this analysis are as follows.

Station columns are one of the the most important structural elements and their main task is mainly to withstand vertical loads .Figure 8 shows the axial-bending capacity curve of the column section and the history of the axial-bending force induced during the Kobe earthquake .As can be seen, during the earthquake, all points were included in the graph, so the failure of the column was not due to the intolerance of the bending force and the axial load or the combination of the two.

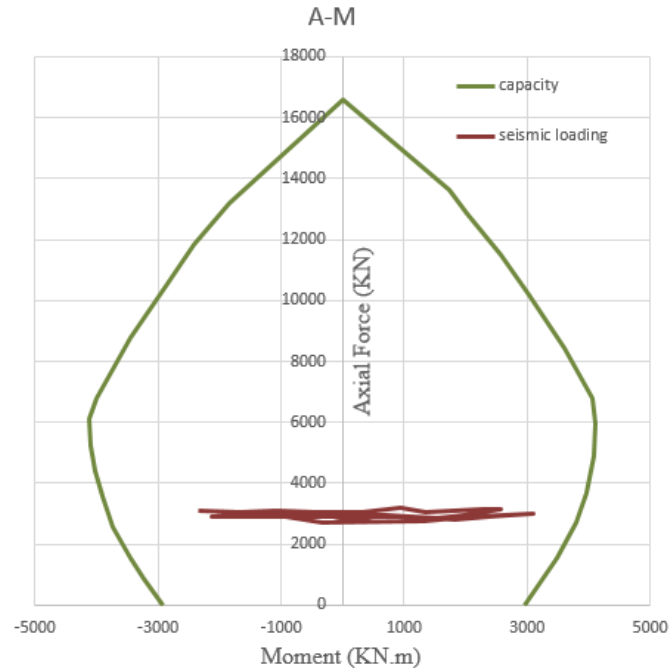


Figure 8. Axial and moment force histories of central column

In this section, the induced shear force in the column during the Kobe earthquake is investigated. Also, to determine the allowable shear strength of the column, the relations (ACI 318M-05) are used as follows [18].

$$V_u = (0.16\sqrt{f_c} + 17\frac{\rho}{\lambda}).bh_0 + f_{yv}\frac{A_{sv}}{s}h_0 \quad (5)$$

Where V_u is shear capacity, λ is shear span ratio, f_t is concrete tensile strength, f_c is concrete compressive strength, b is section width, h_0 is section height, f_{yv} is stirrup yield strength, A_{sv} is stirrup area section, s is stirrup spacing, N_c is axial load, ρ is reinforcement ration. From the above relation, the shear capacity of the column is 540 KN. Figure 9 shows the history of induced shear force in the column during the Kobe earthquake and shows that the column has repeatedly deviated from its shear capacity. And as shown in Figure 7, the drift or horizontal displacement of the station was beyond the allowable limit, so we can say with more confidence that the mechanism of column failure was shear type.

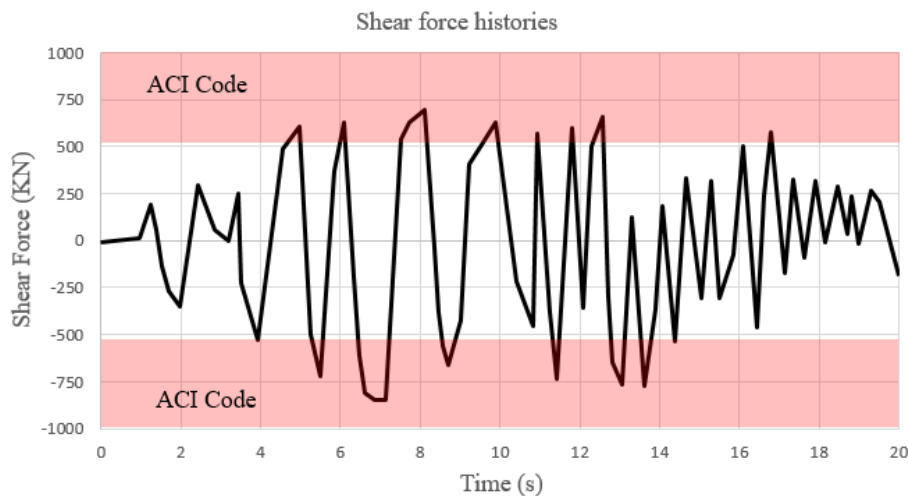


Figure 9. Shear force histories of central column

As can be seen from the figures above, the shear capacity of the middle column has reached to its maximum and the main cause of the destruction of the columns has been shear failure mechanism and not the bending or axial forces. After the columns had

suffered shear damage on both sides and lost their load-bearing capacity, the roof slab and walls cracked due to over loading horizontal or vertical loading and dropped onto the station.

5. CONCLUSIONS

In this paper, the failure of the daikai station during the Kobe earthquake in Japan was investigated and the mechanism of failure and the cause of the destruction of the station were described in more detail using dynamic numerical analysis. The soil and structure system was simulated by a three-dimensional finite element model using Abaqus software 2017 version. Equivalent linear model was used for soil and non-linear CDP model for concrete. The simulations are consistent with what was observed at daikai station and according to the results, the column is the first member of the station structure that failed and is the main cause of the station destruction. In the second model, the bearing capacity of the column during the earthquake was investigated and it was found that the main cause of the failure of the column was not axial and flexural loadings, but low shear capacity of the column was the reason for the destruction of the column. This findings is consistent with the shear damage that occurred at the post-earthquake daikai station's column. Therefore, the main reason for the destruction of the middle part of the station can be considered the low shear capacity of the columns.

6. REFERENCES

- [1] Yamato T, Umehara T, Aoki H, Nakamura S, Ezaki J et al. (1996) Damage to Daikai subway station, Kobe rapid transit system and estimation of its reason. In Proceedings of Technical Conference on the Great Hanshin- Awaji Earthquake. JSCE, Tokyo 1996: 303-320
- [2] Samata S, Ohuchi H, Matsuda T (1997) A study of the damage of subway structures during the 1995 Hanshin-Awaji earthquake. Cement and Concrete Composites 19: 223-239.
- [3] An XH, Shawky AA, Maekawa K (1997) The collapse mechanism of a subway station during the Great Hanshin Earthquake. Cement and Concrete Composites 19: 241-257.
- [4] Iwatate T, Kobayashi Y, Kusu H, Rin K (2000) Investigation and shaking table tests of subway structures of the Hyogoken-Nanbu earthquake. Proc. of the 12WCEE.
- [5] Parra-Montesinos GJ, Bobet A, Ramirez JA (2006) Evaluation of Soil-Structure Interaction and Structural Collapse in Daikai Subway Station During Kobe Earthquake. ACI Structural Journal 103: 113-122.
- [6] Huan Y, Liu J, Chen L (2009) 3D Seismic Response Study on Underground Structures Based on Damaged Plasticity Theory. International Conference of the Associated Research Centers for Urban Underground Space. Response of Subway Station Based on the Substructuring Subtraction Method. Journal of Disaster Prevention and Mitigation Engineering 24: 396-401.
- [7] Sakai H, Nozu A (2002) Dominant frequency component of seismic wave for the damage of underground structure. The 11th Proceedings of Japan earthquake engineering 1033-1038.
- [8] Huo H, Bobet A, Fernández G, Ramírez J (2005) Load Transfer Mechanisms between Underground Structure and Surrounding Ground: Evaluation of the Failure of the Daikai Station. Journal of Geotechnical and Geoenvironmental Engineering 131: 1522-1533.
- [9] Nakamura S (2010) Evaluation of Damage Mechanism of Subway Station Based on the Difference Damage Between Two Damaged Subway Stations Due to the Earthquake. Proceedings of JSCE 654: 335-354.
- [10] Uenishi K, Sakurai S (2000) Characteristic of the vertical seismic waves associated with the 1995 Hyogo-ken Nanbu (Kobe), Japan earthquake estimated from the failure of the Daikai Underground Station. Earthquake Engineering and Structural Dynamics 29: 813-821.
- [11] Kishi ·Norimitsu. 2021. Numerical Simulation of the Daikai Station Subway Structure Collapse due to Sudden Uplift during Earthquake. Journal of Engineering Mechanics
- [12] Yoshida N, Iai S (1998) Nonlinear Site Response and Its Evaluation and Prediction. Proceeding of the 2nd International Symposium on the Effect of Surface Geology on Seismic Motion, Yokosuka, Japan 71-90.
- [13] Haitao ·Yu. 2017. Investigation on Damage Mechanism of the Daikai Station Induced by the Strong Kobe Earthquake. International Journal of Earth & Environmental Sciences
- [14] ABAQUS Inc (2010) Analysis User's Manual. Volume III: Materials.
- [15] Iida H, Hiroto T, Yoshida N, Iwafuji M (1996) Damage to Daikai subway station. Soils and Foundations, Special Issue 36: 283-300.
- [16] Seed HB, Idriss IM (1970) Soil Moduli and Damping Factors for Dynamic Response Analysis. Earthquake Engineering Research Center, University of California, Berkeley, Report No. EERC 70-10.
- [17] Chinese Code of Aseismic Design of Buildings (Code No. GB50011-2010)
- [18] Building Code Requirements for Structural Concrete and Commentary, (ACI 318M-05)
- [19] Liu J, Liu X (2008) Pushover analysis of Daikai subway station during the Osaka-Kobe earthquake in 1995. The 14th World Conference on Earthquake Engineering.
- [20] Yang J (2000) Interpretation of Seismic Vertical Amplification Observed at an Array Site. Bulletin of the Seismological Society of America 90: 275-285.
- [21] Wang R Luo Q (1998) Analysis on the Damage to Underground Structure and Tunnel in Kobe Are Caused by Hanshin Earthquake. Journal of Catastrophology 02: 63-66.
- [22] Kongai K, Kamiya H, Nishiyama S (2001) Deformation buildup in soils during the Kobe earthquake of 1995. Seismic Fault-induced Failures 01: 81-90.
- [23] Chen GX, Zhuang HY, Zhang JL (2004) Analysis on the Earthquake
- [24] Xuesong ·Chai. 2019. Evaluation of the Failure of the Daikai Station. OpenSEES Days Conference
- [25] Iida H, Hiroto T, Yoshida N, Iwafuji M (1996) Damage to Daikai subway station. Special issue on geotechnical aspects of the January 17 1995 Hyogoken-Nanbu earthquake. Soils Found 283-300.
- [26] Nakamura S, Yoshida N, Iwatate T (1996) Damage to Daikai subway station during the 1995 Hyogoken-Nambu earthquake and its investigation. Japan Society of Civil Engineers, Committee of Earthquake Engineering: 287-295.
- [27] C. Phillips, A. R. Kottke, Y. M. A. Hashash, and E. M. Rathje, 2012. "Significance of ground motion time step in one dimensional site response analysis," Soil Dyn. Earthq. Eng., vol. 43, pp. 202-217.
- [28] J. Kalkan, L. G. Baise, E. M. Thompson, and L. Dorfmann, 2015. "Comparison of 1D linear, equivalent-linear, and nonlinear site response models at six KiK-net validation sites," Soil Dyn. Earthq. Eng., vol. 69, pp. 207-219.