Nonlinear Analysis on the Collapse of Daikai Station Subway Tunnel during the Hyogo-ken Nambu Earthquake of January 17th 1995

(Influence of the Vertical Acceleration)

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The RC rectangular tunnel at Daikai Station of Kobe City Municipal Subway System collapsed during the Hyogo-ken Nambu Earthquake of January 17th, 1995. A two-dimensional finite element computer program was developed and utilized in clarifying the failure mechanism of the tunnel. The program can take account of material and geometrical nonlinearity. It was concluded that the horizontal ground motion had the primary influence on the collapse of the tunnel as compared to that of the vertical ground motion. It induced high bending moments and shear forces at the intermediate column due to the increase in the horizontal relative displacement between the top and bottom slabs of the tunnel. The poor ductility and the low shear capacity of the intermediate column triggered the collapse of the tunnel.

1. Introduction

In the past history of earthquakes, no complete collapse was reported for large-scale underground structures. Consequently, many structural engineers believed that they can not be destroyed during earthquakes. However, the RC rectangular tunnel at Daikai Station of Kobe City Municipal Subway System collapsed during the Hyogo-ken Nambu Earthquake of January 17th, 1995 in Japan. It is a duty of structural engineers to clarify the main causes of the collapse of the tunnel, in order to earn some engineering lessons and experiences. For this purpose, a two-dimensional finite element computer program is developed such that the nonlinear behavior of the tunnel in the transverse direction under seismic excitations can be analyzed. Material nonlinearity is included in the formulation of the finite element model of the RC tunnel. The nonlinear model can take account of concrete and/or reinforcing steel yielding, concrete crushing, reinforcing steel failure, as well as shear failure in concrete and in shear reinforcement. Other features such as soil plasticity, the effect of vertical acceleration which was remarkable in this earthquake, dynamic effect of the dead load of the soil above the tunnel, dynamic earth pressure, and geometric nonlinearity are also considered in the computer program.

Key Words: FEM, RC tunnels, nonlinear seismic analysis.

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2. RC Nonlinear Model

The nonlinear behavior of the material of a tunnel element is simplified to a hysteric bilinear relationship with strain-hardening as shown in Fig.1. A combination of the "Plastic Hinge Idealization[1,2]" and the "Basic Principles of Reinforced Concrete Structures" based on the Japanese Standard Specification[3] was used to derive the nonlinear stiffness matrix of an element. The proposed stress-strain relationships[3,4] of concrete and reinforcing steel are shown in Fig.2.

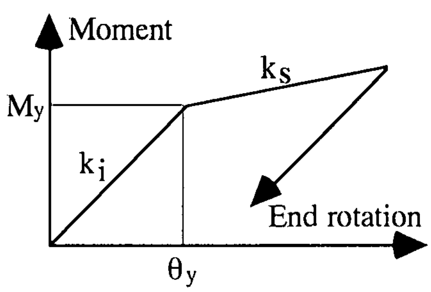
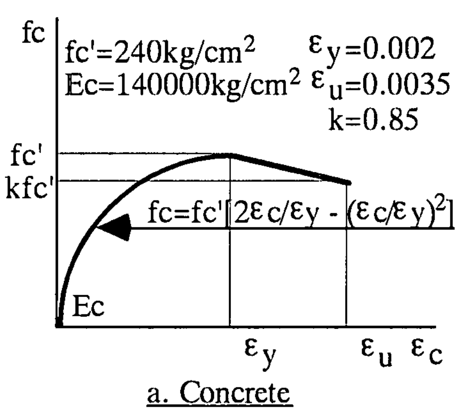
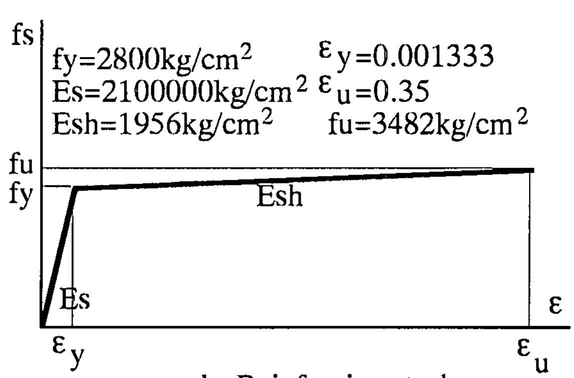


Fig.1 Nonlinear behavior of tunnel elements



a. Concrete



b. Reinforcing steel

Fig.2 Proposed stress-strain relationships

Yielding occurs in the extreme fibers of concrete if strain of 0.002 is reached provided that the reinforcing steel bars had not yielded, or in reinforcing steel bars if yield strain of 0.001333 is reached provided that concrete had not yielded. A balanced yielding occurs if both concrete and reinforcing steel bars yield at the same instant. Moreover, concrete crushes if the strain in its extreme fibers reached 0.0035, while reinforcing steel bars fail if the ultimate stress reached $3.482 tf/cm^2$. A balanced condition occurs if concrete crushes and reinforcing bars yield at the same instant. The approximate yield and failure interaction surfaces shown in Fig.3 are constructed based on the basic principles of reinforced concrete structures given in the literature[3,4]. A plastic hinge is formed at a section if the combination of the axial force and bending moment lie on or outside the yield interaction surface. A section fails if the combination of the axial force and bending moment lie on or outside the failure interaction surface. Concrete may also fail by the diagonal tension shear failure. The basic assumption is that the ultimate shear capacity of a member is the sum of the components carried by concrete and shear reinforcement. The effect of concrete strength, member depth, reinforcement ratio, and axial force are taken into account empirically[3].

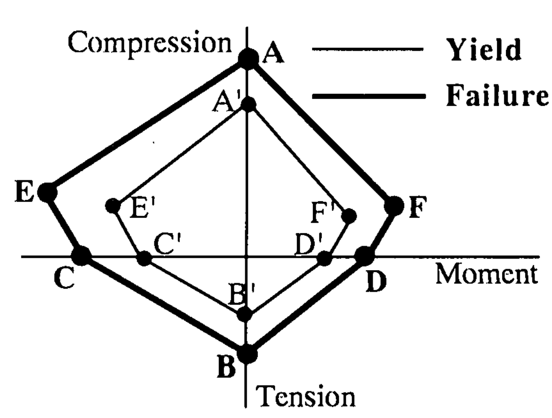


Fig.3 Yield and failure interaction surfaces

The nonlinear stiffness matrix is determined from the elastic one by introducing special coefficients, the values of which depend on the yield condition of it's two nodes. The nonlinear stiffness matrix of a tunnel element was derived in a previous study[5].

3. Geometric Nonlinearity

It is expected that the deflections of the nonlinear system will be large enough to cause significant changes in its geometry. Therefore, a new deformed configuration is determined at the end of each time step of the seismic analysis. The updated stiffness matrices of soil, joint and tunnel elements are then determined according to the new deformed configuration. Moreover, the effect of the axial force in a tunnel element on its shear force and bending moment are also considered by including the geometric stiffness matrix of a beam element[5].

4. Analysis of the Nonlinear System

The nonlinear dynamic analysis is performed by the direct integration method utilizing a numerical step-by-step procedure. The response of the system is assumed to be linear within each time step, and is determined by solving the dynamic equation of equilibrium utilizing the Wilson-θ integration method. At any time step of the seismic analysis, if a plastic hinge is formed at one node of the tunnel, the rotation of that node is assumed to be independent. The global stiffness matrix of the system will have zero coefficients at the column and row corresponding to the rotational degree of freedom of that node. Therefore, the size of the global stiffness matrix shall be reduced by the total number of plastic hinges formed at that time step. The solution of the system is then carried out for the reduced stiffness matrix.

5. Analysis of Daikai Station Tunnel

Daikai Station was opened in 1968. Thirty out of thirty five of the RC intermediate columns of the station supporting the top slab failed by diagonal shear as shown in Photo 1, and lost their axial load carrying capacity. As a result, the top slab collapsed as shown in Fig.4 leading to the failure of the above street. About 95m by 28m area of the asphalt road above the station settled with a maximum displacement of 3m. Soil profile in the site is 2.2m Holocene clay, followed by 3.0m Holocene sand, 3.3m Pleistocene sand, 8.7m Pleistocene clay, and 5.7m Pleistocene clay. The base layer for seismic analysis was estimated at 17.2m under the ground surface. Fig.5 shows the dimensions and reinforcement of the intermediate column. Fig.6 shows the records of Kobe Ocean Meteorological Observatory, but modified on the assumed base rock.

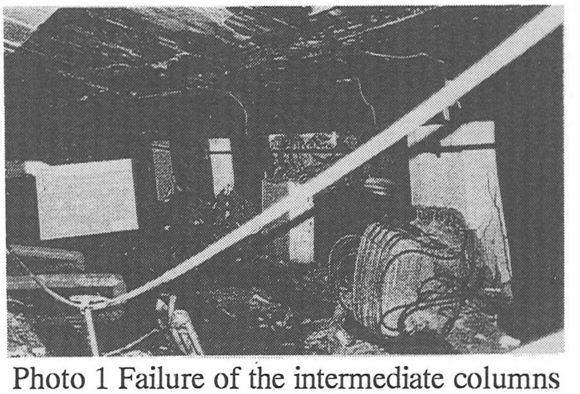


Photo 1 Failure of the intermediate columns

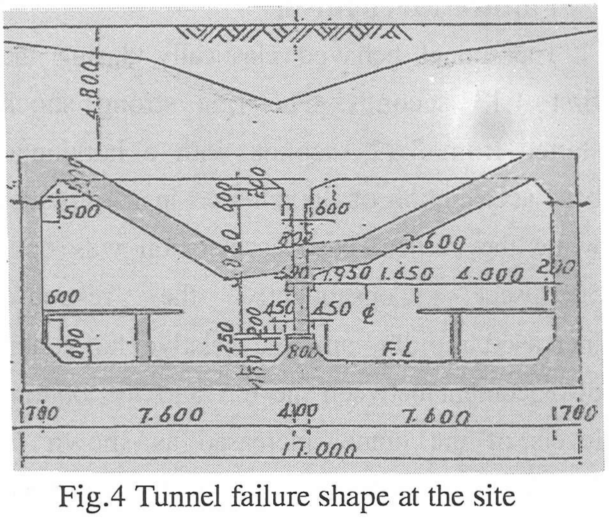


Fig.4 Tunnel failure shape at the site

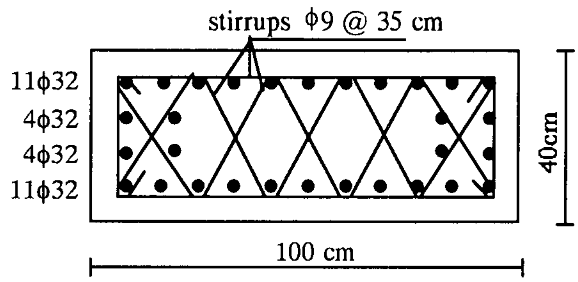


Fig.5 Section in the intermediate column

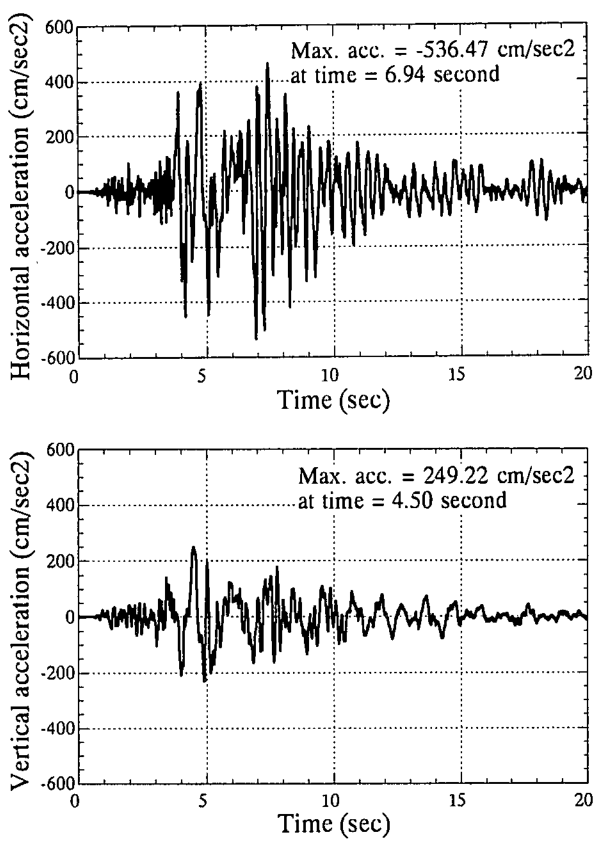


Fig.6 Kobe Ocean Meteorological Observatory Records

6. Tunnel Behavior and Failure Mechanism

The tunnel behaved elastically during the first 4.13 seconds. The first strong shock started from 3.92 seconds with a horizontal base acceleration of 360cm/sec^2 in magnitude, while the vertical base acceleration was only 50cm/sec^2. Consequently the response increased rapidly, and the relative horizontal displacement between the top and the bottom levels of the tunnel increased as shown in Fig.7. This relative displacement generated high bending moments and shear forces at the position of the intermediate column as shown in Fig.8. In the mean time, the vertical base acceleration reached its first peak at 4.04 second with magnitude of about -215 cm/sec^2. Consequently, the axial forces in the intermediate column started to fluctuate with a large range around its static value (about 336 tf) as shown in Fig.9, due to the combined action of the vertical acceleration and the dynamic effect of the ground above the top slab of the tunnel. The maximum increase in the axial force was as much as 1.66 times the static force. On the other hand, the maximum decrease in the axial force was as low as 0.68 times the static force. Yielding of the tensile reinforcing bars due to the interaction between axial force and bending moments started to occur at time 4.13 second.

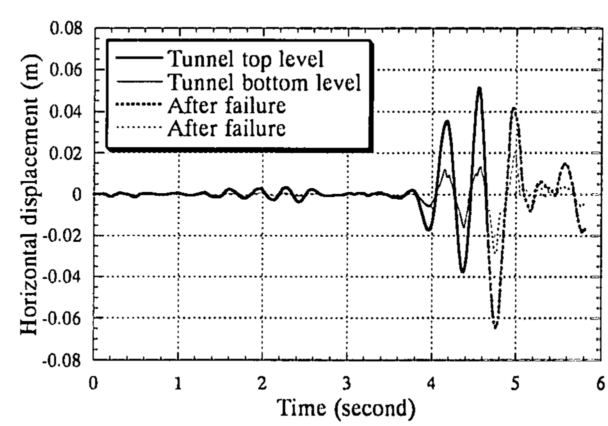


Fig.7 Horizontal displacements of the tunnel

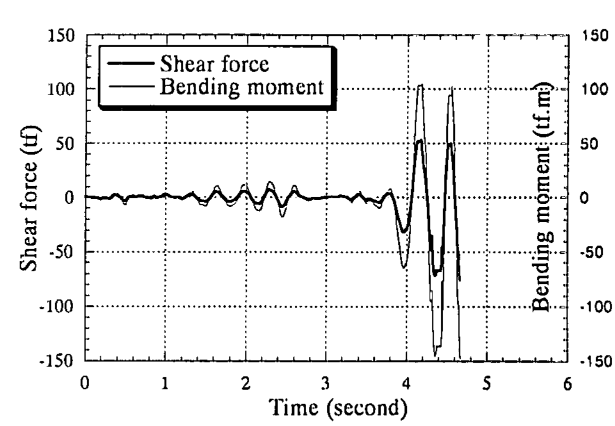


Fig.8 Shear forces and bending moments at the intermediate column

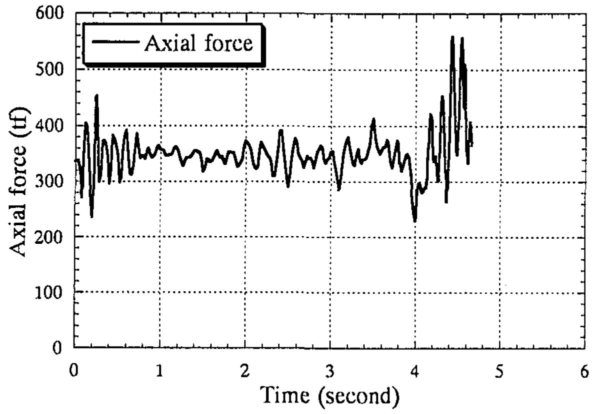


Fig.9 Axial forces of the intermediate column

Fig. 10 shows the ductility of the intermediate column. The figure relates the shear stress in the section (determined by dividing the shear force by the effective area of concrete) with the relative horizontal displacement between the top and the bottom of the column. This relative displacement DX is normalized by the height of the column H. Note that part of this shear stress is resisted by stirrups of the column. The intermediate column failed by diagonal shear at time 4.74 seconds with a maximum shear stress of 22.4 kgf/cm^2, due to its weak shear reinforcement. Photo 1 of site investigation confirms the mentioned failure mechanism.

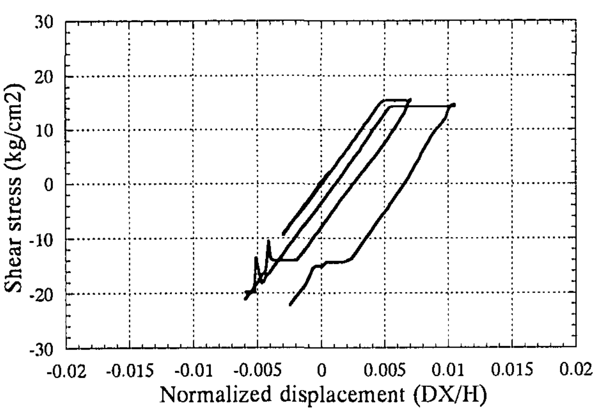


Fig. 10 Ductility of the intermediate column

7. Influence of Vertical Ground Motion

Generally, the vertical component of ground motion is ignored in structural design of tunnels owing to its small contribution to the tunnel's safety and seismic response. Moreover, in the past history of earthquakes the vertical ground motion was small as compared to the horizontal one. However, the vertical ground motion was remarkable in Hyogo-ken Nambu Earthquake. The maximum vertical acceleration was about 46% that of the horizontal acceleration according to the records of Kobe Ocean Meteorological Observatory and its contribution to the collapse of the tunnel can not be ignored. The tunnel is re-analyzed under the same conditions but the vertical ground motion is ignored.

The horizontal displacements of the top and bottom slabs of the tunnel are shown in Fig.11. Comparing Figs. 7&11, it is observed that the vertical ground motion considerably increase the horizontal displacements. This is due to the combined influence of vertical forces arising from the vertical ground motion and the deformed configuration of the system (geometric nonlinearity). The bending moments, shear forces, and the axial forces at the position of the intermediate column are shown in Figs.12&13. The axial forces in the intermediate column fluctuated with a small range not exceeding 20% around its static value.

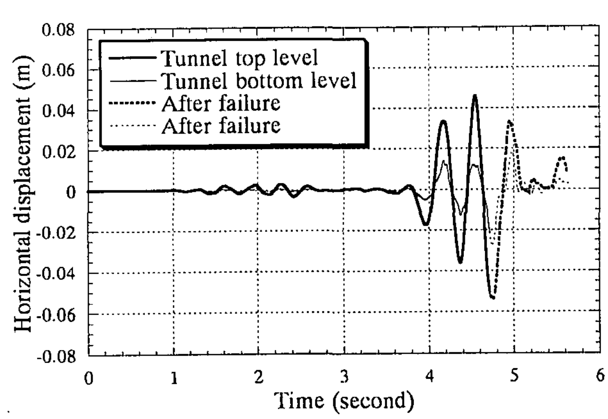


Fig.11 Horizontal displacements of the tunnel

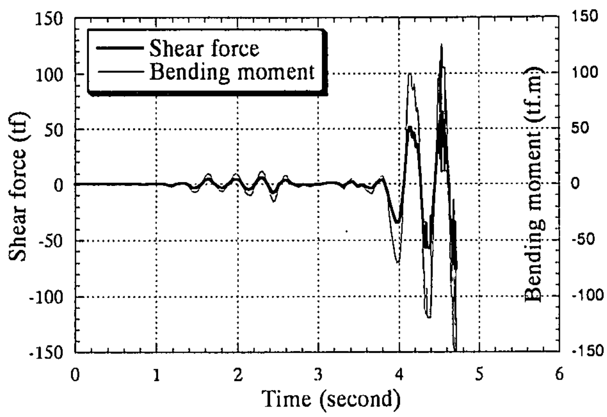


Fig.12 Shear forces and bending moments at the intermediate column

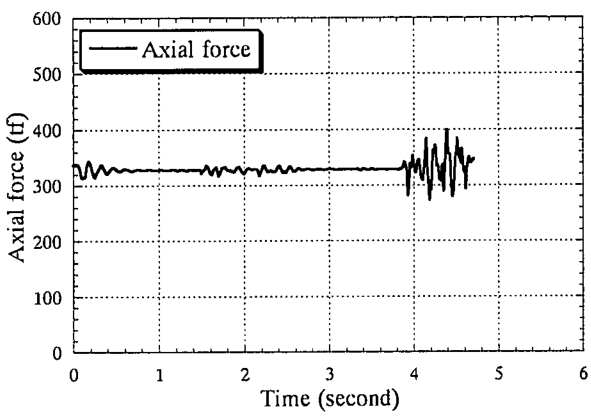


Fig.13 Axial forces of the intermediate column

In this analysis, the intermediate column also failed by diagonal shear at time 4.66 seconds. Therefore, the vertical ground motion had a secondary influence on the collapse of the tunnel. It affected the axial forces in the column, which in turn had a contribution in determining it's shear capacity, and consequently accelerating the time of its failure.

8. Conclusions

Within the analytical results and the presented discussion, the following can be tentatively concluded:

1. The seismic design codes of underground structures shall be improved to take account of the nonlinearity of the structure and the surrounding soil for the case of severe ground motions.
2. The poor ductility and the lack of shear reinforcement in the intermediate column triggered the collapse of the tunnel. In fact, the interaction between the compression capacity, shear capacity, and the ductility of the intermediate column was the main cause of collapse of the tunnel.
3. The horizontal ground motion and the nonlinear behavior of the intermediate column had the primary influence on the collapse of the tunnel.
4. The vertical ground motion -rarely considered by structural engineers- had a secondary influence on the collapse of the tunnel. It affect the axial forces in the intermediate column, which in turn has a contribution in determining the shear capacity of the column.

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