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Seismic Design for Large-Scale Cut-and-Cover Tunnels

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Summary

The cut-and-cover tunnel described in this paper pertains to a large-scale underground station with a length of 400 m, a cross section of 25 m height, and 25 m width in soft ground. . One part of the top slab of the station will be required to serve as the foundation for an elevated viaduct structure, and the entire structure will be a very complicated to be built in a seismically high risk area, and in view of the fact that the structure will be an important train station, there will be many passengers, and seismic safety will be an important design task.

It has been the practice to consider underground structures as highly safe against earthquakes,. but in 1995 in the South Hyogo Prefecture Earthquake struck for the first time in the world. The cause for this disaster was due to the fact that the underground structure was located where relative displacement occurred in the soils and shear deformation took place during the earthquake. The capability of the reinforced concrete center column to resist the shear forces was exceeded.

In view of the calamity experienced by this catastrophe, in this report seismic safety could be satisfied to secure ductility for large-scale earthquake loads. The important factors in the seismic design are

1) to understand the non-linear seismic behavior of structures with high-degree redundancy supported by the ground, and
2) to secure the seismic safety as a whole structure.

1. The Earthquake Damage Experienced by the South Hyogo Prefecture (Kobe) Earthquake and The Causes of Why It Happened.

1.1 The Damage Experienced by the South Hyogo Prefecture (Hyogo) Earthquake.

On 17 January 1995, Hyogo Prefecture was struck by an earthquake with a magnitude of 7.2 in the "South Hyogo Prefecture Earthquake". The earthquake occurred near a densely populated area and more than 5,500 people perished with a high loss of property. All types of structures were damaged including high-speed railways and roadways, harbor facilities, water supply, sanitary collection and treatment facilities, gas, and electric power and distribution systems.

Of the civil structures that were damaged, this was the first time in the world that a subway system was subjected to large damage on a major scale. Especially, the Daikai Subway Station was almost completely demolished . Fig.-1.1 gives a cross section view of the station and describes the damages incurred[1]. The ground in the vicinity of the station had a shear wave velocity Vs of more than 200 m/s which is fairly good, the ground higher than the tunnel was Vs = 140 m/s. From the damage sustained by the station, it can be seen that the reinforced concrete central columns supporting the station upper slabs failed, and the upper slabs collapsed by breaking in two.

Although they did not collapse in a similar manner to the Daikai Station, there were ruptures due to shear failures in the reinforced concrete columns at Kamisawa and Sanno-Miya Stations, and the top and intermediate slabs gave way.

It has been the rule that underground structures are almost never damaged in earthquakes, and the few damages that have occurred have been very minor. There has been a strong belief that "underground structures are very stable since they will move in the same manner as the surrounding ground". In fact, the underground subway stations that sustained earthquake damage were not designed for earthquakes of the South Hyogo Prefecture Earthquake Class. With the experience gained by this earthquake, it has become necessary to reconsider the design methods for the design of underground structures.

1.2 The Causes of the Earthquake Damage to the Excavated Tunnels[2)~5)]

The damages sustained by the Daikai Subway Station and the excavated tunnels are now being investigated by the various authorities to determine why the damages were so widespread. From their findings, the following points are some of the reasons for the damages to the excavated tunnels.

1) At the time of the earthquake, there was shear deformation within the layers of the ground, and there was shear deformation caused transverse to the tunnels. As a result, there was relative differential deformation caused between the upper and lower slabs.
2) There was average axial stress in excess of 80 MPa in the reinforced concrete center columns which was greater than in the side walls, and the center columns had less ductility where shear collapse set in ahead of bending collapse.
3) Due to the relative displacement between the top and bottom slabs, the central column support failed as it could not move in consonance with the side wall.

From the above, in order to maintain the earthquake strength of the excavated tunnels, deformation of ground-structure system considering its non-linear characteristics is necessary to be found out and sufficient ductility shall be ensured. Especially, it has become necessary to ensure that the center column does not collapse by shear force.

2. The Structure to be Targeted.

2.1 A Description of the Yokohama MM21 Line Underground Station.

The objective of this report is the large-scale subway station in the excavated tunnel as shown in Fig.-2.1. The main features that will require earthquake design are as follows:

1) The structure is located in an area with high seismic risk.
2) The station has heavy passenger traffic, and be a large-scale underground structure of high importance.
3) The station is located some -35 m underground in a soft clayey ground.
4) One part of the upper slab of the station will serve as the foundation for an elevated viaduct structure and the subway station will have a pile foundation.

The center column is a steel pipe column filled with concrete and all other member are made of reinforced concrete. The reasons for using concrete filled steel pipe is to ensure sufficient ductility of the center column, and not to set in ahead of shear collapse.

2.2 Description of the Earth Foundations.

Fig.-2.1 gives a description of the foundation soils for the seismic analysis. The depth of ground layer is approximately 30 m from the bed rock for the seismic design, and the soils are soft clay with a $V_s = 120 \sim 150$ m/s.

3. Seismic Analysis Conditions and the Methods Used.

3.1 The Purpose for Seismic Analysis

The following three types of seismic analysis was performed for this study:

1) Seismic deformation method by the uniform loss of stiffness model[6].
2) Dynamic analysis by the uniform loss of stiffness model.
3) Non-linear seismic deformation method.

In comparing the "Dynamic analysis by the uniform loss of stiffness model" with the "Seismic deformation method by the uniform loss of stiffness model", the static model of the seismic deformation method can be evaluated.

From these results, the seismic characteristics of structures can be evaluated by the "non-linear seismic deformation method".

3.2 Seismic Deformation Method by the Uniform Loss of Stiffness Model.

Fig.-3.1 is the concept of the Seismic Deformation Method, and Fig.-3.2 gives the flow diagram of the Seismic Deformation Method. The analysis model is a framework model supported by the ground spring, and the bending stiffness are assumed at $1/5^{th}$ of their total cross sectional state considering the loss of stiffness caused by earthquakes.

3.3 Dynamic Analysis by the Uniform Loss of Stiffness .

The analysis model is a 2-dimensional FEM "FLUSH" of the ground~structure interaction system. The structure (excavated tunnel, viaduct, pile foundation) is a space frame model, and the ground foundation is displayed as a 2-dimensional solid model. In addition, the bending stiffness of the components are similar to the seismic deformation method. For the ground ,the loss of stiffness at earthquake time is considered.

3.4 Non-Linear Seismic Deformation Method.

The non-linear seismic deformation method is basically similar to the uniform loss of stiffness model. The different points are that the stiffness of the structure is not reduced uniformly, but the components have been given non-linear characteristics. For this reason, the components have been given an initial stress analysis under permanent load prior to applying earthquake loads.

4. Input Motions.[7)]

For the earthquake motion inputs, Fig.-4.1 gives the velocity response spectrum, and Fig.-4.2 gives the acceleration corresponding to the spectrum. The velocity response spectrum in Fig.-4.1 is based on the ground motion observed of the South Hyogo Prefecture Earthquake .

5. Seismic Deformation Method by the Uniform Loss of Stiffness Model.

5.1 Calculation of the Ground Response

Fig.-5.1 gives the ground response to be used for the seismic deformation method. This is calculated according to the One Dimensional Dynamic Analysis "SHAKE".

5.2 Establishing of the Analysis Model.

Fig.-5.2 gives the analysis model of the seismic deformation method . The analysis model is a two dimensional framework supported by the ground spring, and the bending stiffness are assumed at $1/5^{th}$ of their total cross sectional state considering the loss of stiffness caused by earthquakes.

The various loads of the seismic deformation method is based on the ground earthquake response from Fig.-5.1, and the ground displacement loads are obtained from the ground displacement and the ground spring, and the peripheral shear force is obtained from the ground shear force, and the inertia force was obtained from the ground acceleration

5.3 Results of the Analysis.

Fig.-5.3 gives a display of the displacement, bending moment, and the shear forces obtained from seismic deformation method by the uniform loss of stiffness .

6. Dynamic Analysis by the Uniform Loss of Stiffness Model.

6.1 Establishment of the Analytical Model.

Fig.-6.1 gives the Dynamic Analysis Model by the Uniform Loss of Stiffness Model. The input acceleration from Fig.-4.2 has been applied to the bed rock.

6.2 Results of the Analysis.

Fig.-6.2 gives a display of the deformation and maximum acceleration obtained from dynamic analysis.

6.3 Comparison of the Seismic Deformation Method and the Dynamic Analysis Method.

Fig,-6.3 gives a comparison of the bending moment of analysis by the Seismic Deformation Method and the Dynamic Analysis. This shows that they are generally compatible, and the static analysis of the Seismic Deformation Method indicates that the interaction response (ground~structure, viaduct~subway tunnel~pile foundation) are generally displayed.

7. Non-Linear Seismic Deformation Method.

7.1 Establishment of the Analytical Model.

The analysis model in Fig.-5.2 of the Seismic Deformation Method by the Uniform Loss of Stiffness Model basically agree with the Non-Linear Seismic Deformation Method. The different points are that the stiffness of the structure is not reduced uniformly, but the components have been given non-linear characteristics.

Fig.-7.1 gives the non-linear characteristics which have been obtained from References 8) and 9). The characteristics are M ~ Φ Type (bending moment ~ curvature).

7.2 Results of the Analysis.

Fig.-7.2 gives a display of the displacement, bending moment, and the shear forces obtained by the non-linear seismic deformation method

7.3 Evaluation of the Seismic Safety of Structures.

The shear reinforcing steel in each member have been placed so that bending collapse will not set in ahead of shear collapse. After which, the deformation capability of side walls and center columns at each floor are evaluated by comparing the relative displacement of side walls and center columns at each floor and allowable displacement generated. In this case, the allowable displacement was determined based on reference 10). The result is given in Table.-7.1. show that the side wall and center column at each floor ensure sufficient ductility.

8. Conclusion.

As a result of the experience gained by the South Hyogo Prefecture Earthquake, and the results of analysis performed for large-scale seismic safety of structures, the following facts have been disclosed:

1) A comparison of the results of the Seismic Deformation Method and the Dynamic Analysis show that both methods are relatively compatible, and that the Seismic Deformation Method can almost fully display dynamic interaction (ground~structures, elevated bridge structure~excavated tunnel~pile foundation).
2) According to the non-linear seismic deformation method, the seismic design for large-scale earthquake loads are to secure the seismic safety as a whole structure.

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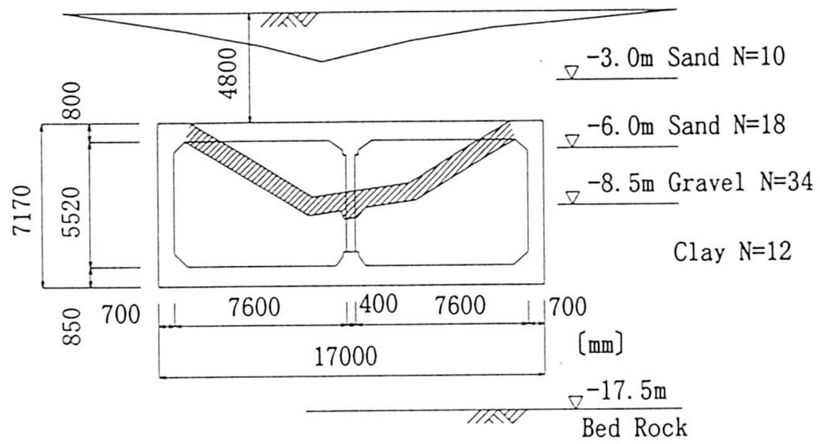
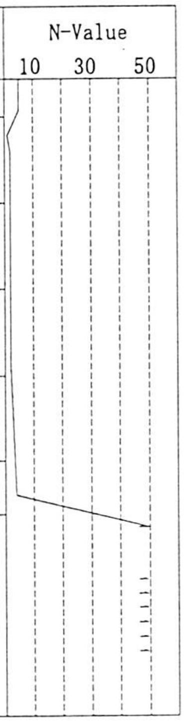


Fig.-1.1 Damages at Daikai Station

Depth (m)	Soil Type	γ t (tf/m³)	V s (m/s)
	A sl	1.70	140
	A c2-1	1.60	120
	A c2-2	1.60	120
	A c2-3	1.60	120
	A c2-4	1.60	120
	A c3	1.60	150
	K hc	2.00	420



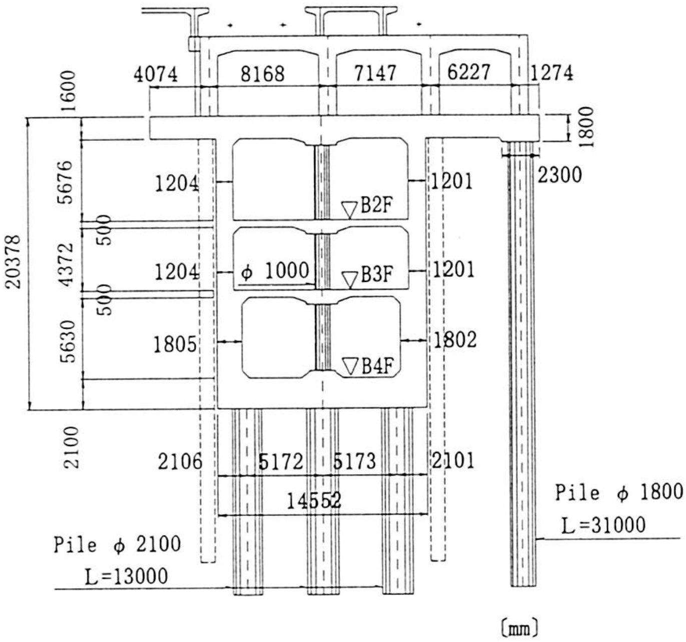


Fig.-2.1 Cross Section View of Yokohama MM21 Line Underground Station

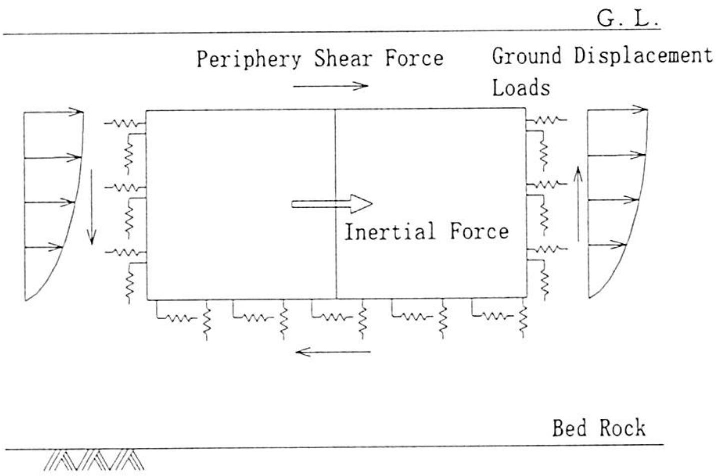


Fig.-3.1 The Concept of
Seismic Deformation Method

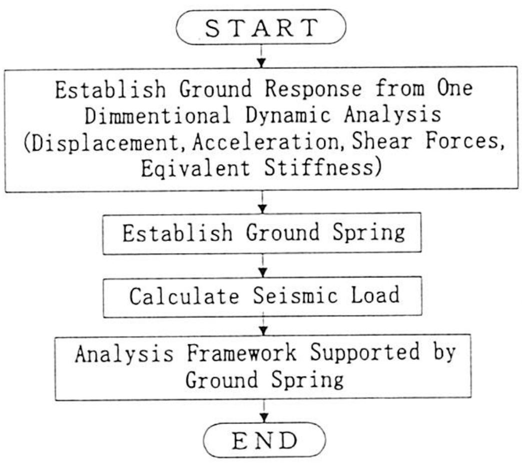


Fig.-3.2 Flow Diagram of
Seismic Deformation Method

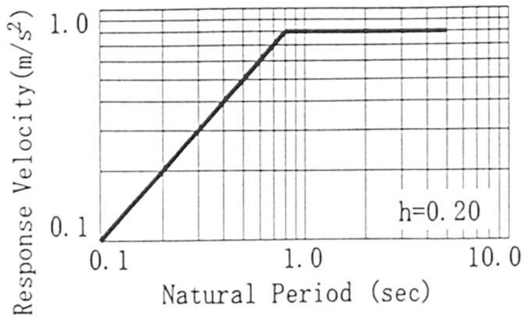


Fig.-4.1 Response Velocity Spectrum

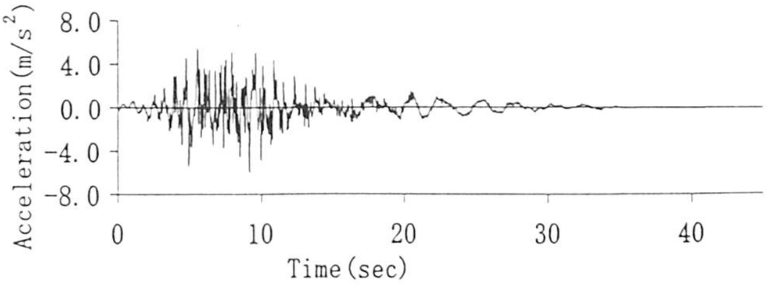
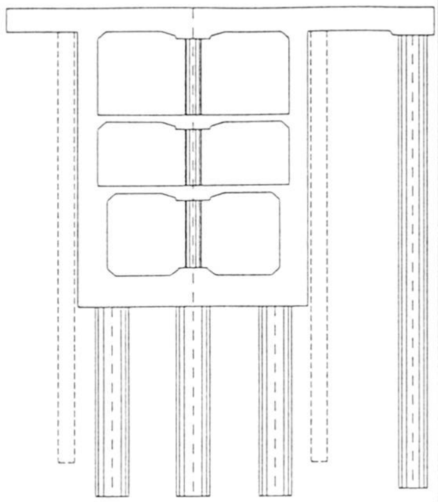


Fig.-4.2 Input Motion



Cross Section of Structure

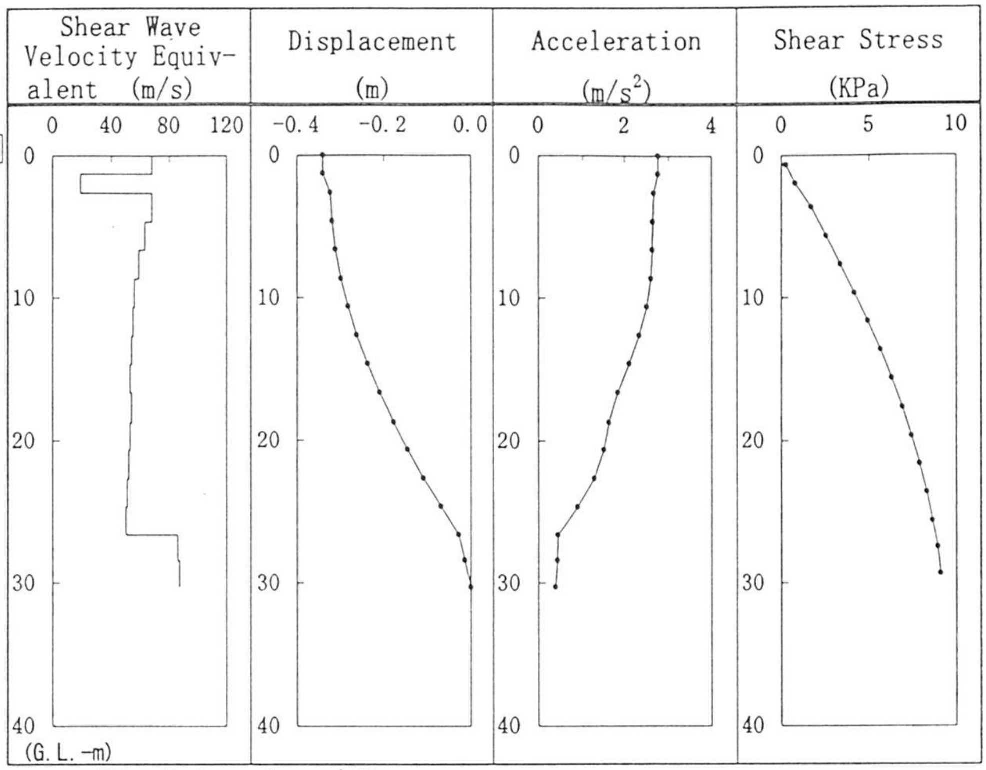


Fig.-5.1 Ground Response

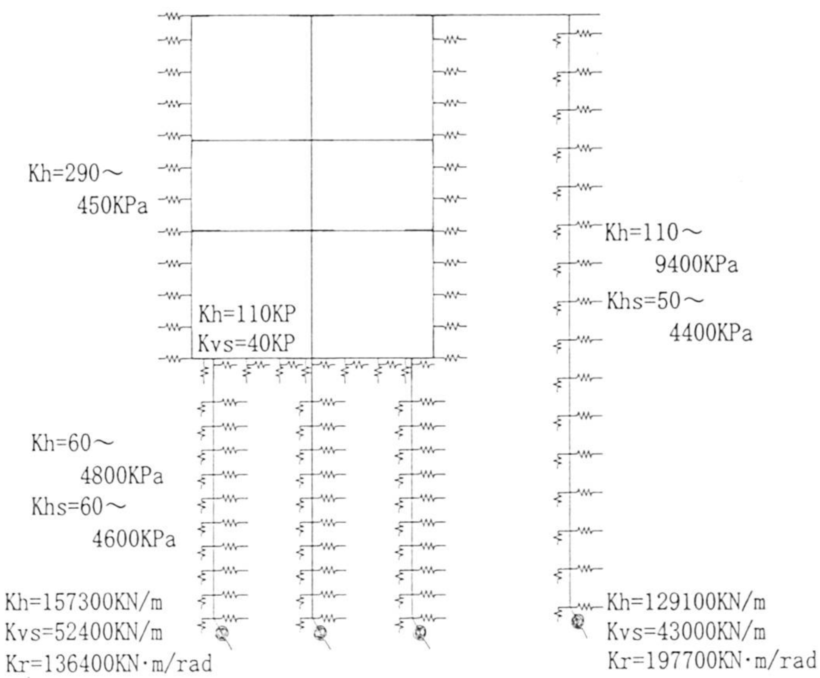
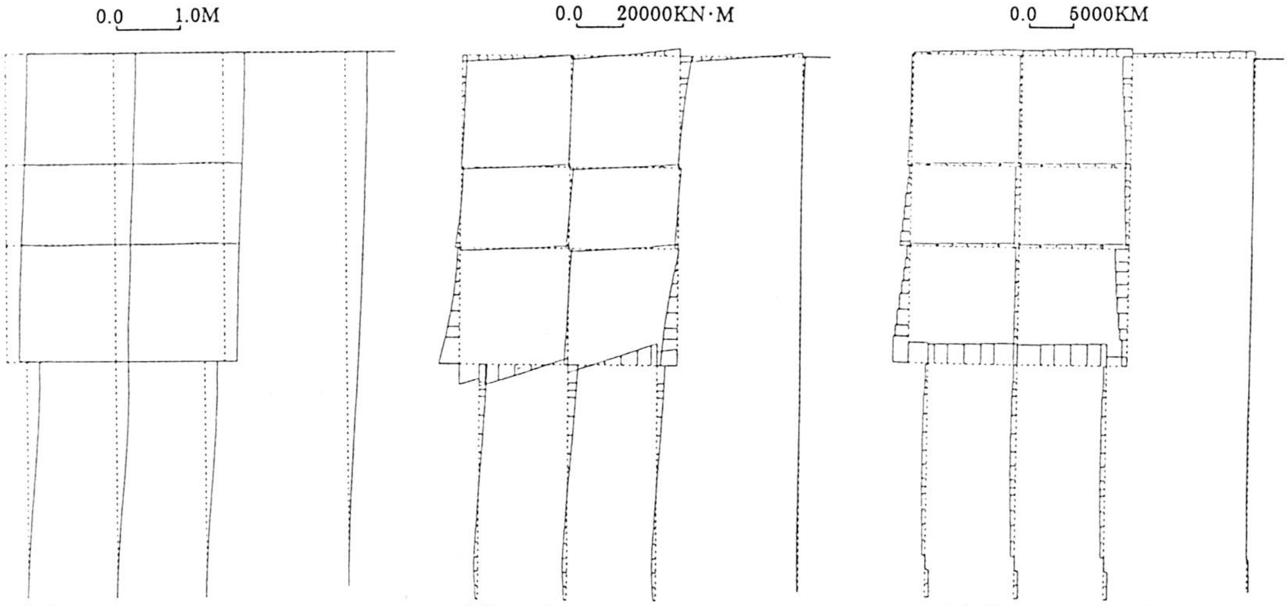


Fig.-5.2 Model of Seismic Deformation Method



(a)Deformation Diagram (b)Bending Moment Diagram (c)Shear Force Diagram

Fig.-5.3 Result of Seismic Deformation Method

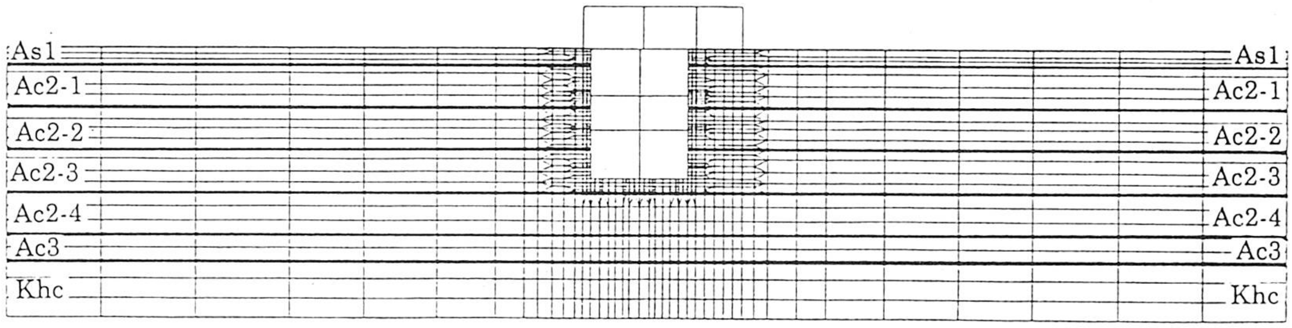
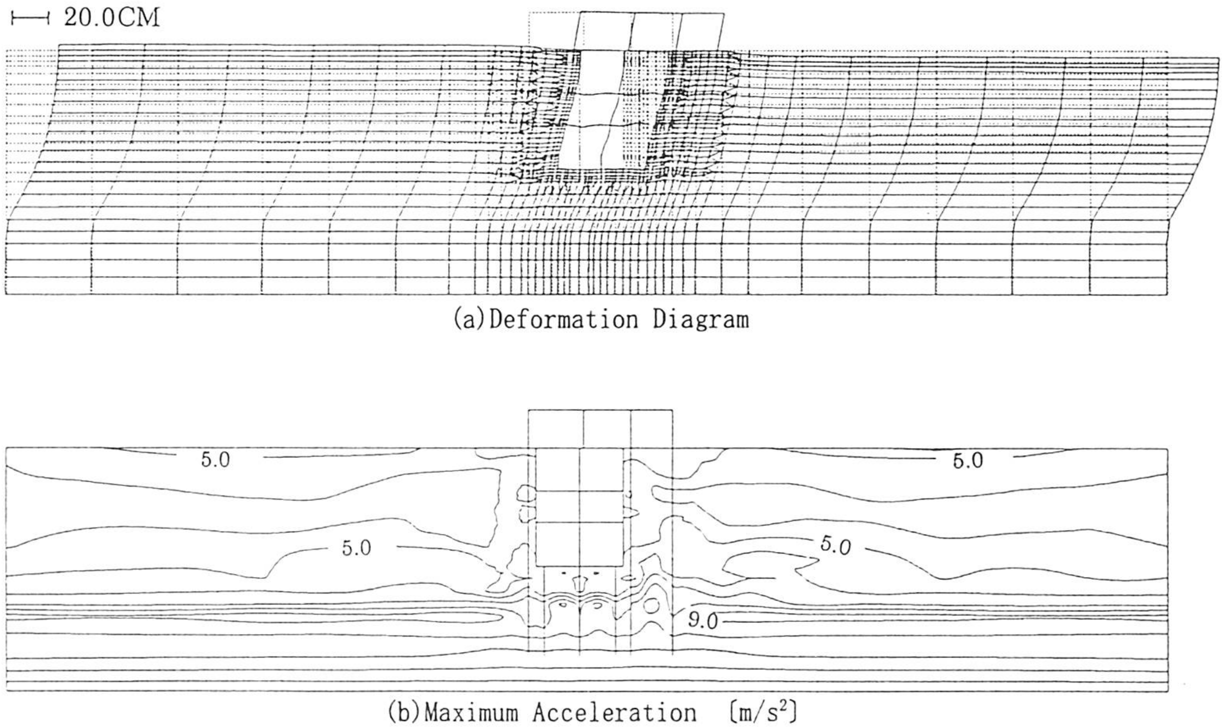


Fig.-6.1 Dynamic Analysis Model



(a)Deformation Diagram

(b)Maximum Acceleration [m/s²]

Fig.-6.2 Result of Dynamic Analysis

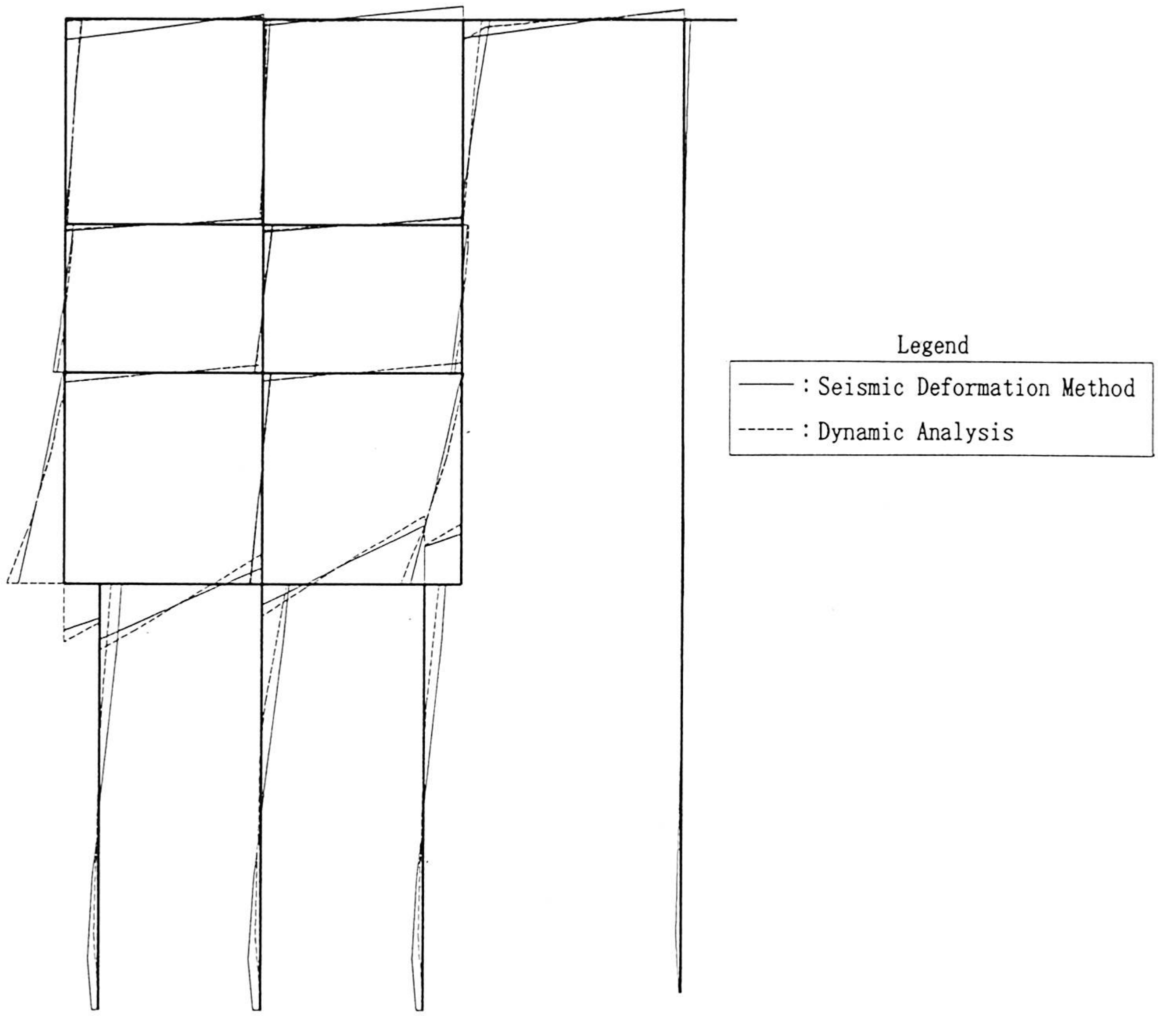


Fig.-6.3 Comparison of Bending Moment

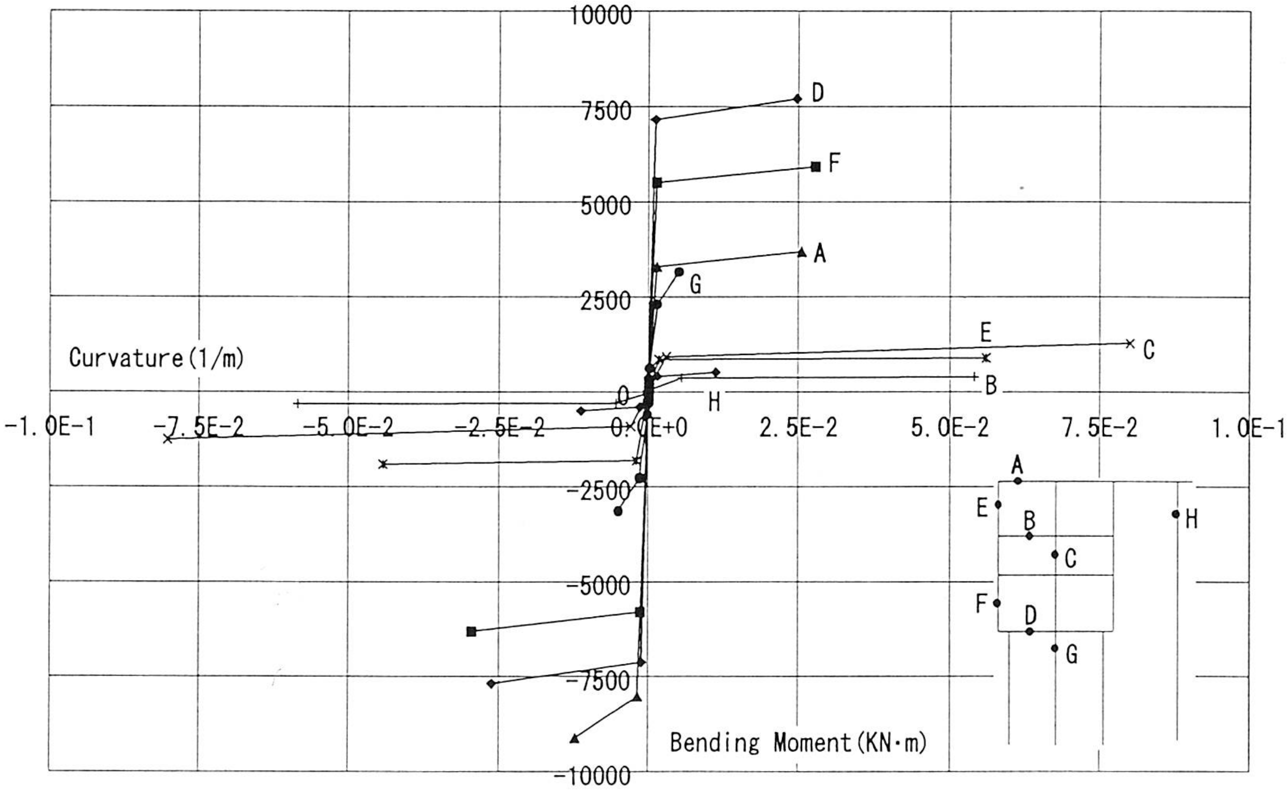
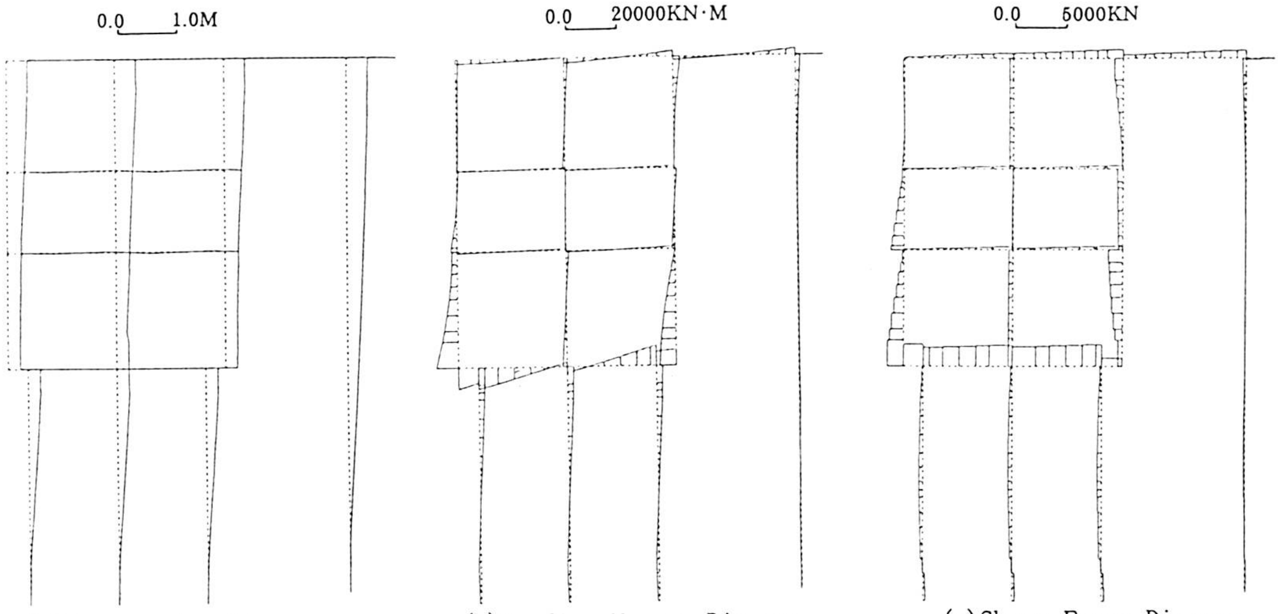


Fig.-7.1 The Characteristics for Bending Moment～Curvature



(a)Deformation Diagram (b)Bending Moment Diagram (c)Shear Force Diagram

Fig.-7.2 Result of Non-linear Seismic Deformation Method

Table-7.1 Results of Ductility Check

Location		Relative Displacement δud (cm)	Allowable Ductility μo	Yield Displacement δy (cm)	Allowable Amount of Displacement (μo·δy) (cm)
Left Wall	B 2 Fl.	0.7	10.0	1.6	16.0
	B 3 Fl.	2.0	4.3	0.3	2.2
	B 4 Fl.	0.9	6.8	1.0	6.4
Right Wall	B 2 Fl.	0.7	4.7	1.8	8.3
	B 3 Fl.	2.0	4.5	0.2	2.0
	B 4 Fl.	0.9	6.7	0.9	6.2

Location		Relative Displacement δud (cm)	Allowable Amount of Displacement (δu) (cm)
Center Column	B 2 Fl.	0.7	28.4
	B 3 Fl.	2.0	8.0
	B 4 Fl.	0.9	22.8

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