NONLINEAR RESPONSE ANALYSIS AND DAMAGE EVALUATION FOR UNDERGROUND STRUCTURES

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SUMMARY

To make clear the mechanism of damage factors of underground structures taking into account of nonlinear soil structure interaction, two-dimensional seismic nonlinear response analyses were conducted. The time integration type of FEM nonlinear analysis and the equivalent linear FEM were adopted. The nonlinear analysis code can take nonlinearity of concrete structures as well as soil layers into account. The hyperbolic type of constitutive low for soil layer was adopted, tri-linear type for concrete structure as well. The input motion was evaluated by Two-dimensional FEM response analysis based on the observed earthquake wave in Kobe City.
The results from comparisons between observed damage and analytical evaluation indicated that the equivalent linear analysis estimated larger extent of damage than observed results, however, the nonlinear analysis coincided with the observed results. The maximum plastic ratio at the center columns was about 2, which reveals that catastrophic damage has not occurred and coincides with the observed damage. It was shown that large shear force on center columns makes plastic hinge firstly and this plasticity of the center columns affects the reduction of stresses of the adjacent area such as walls and slabs. This failure pattern can predict using nonlinear response analysis code. And these analysis codes can play an important role for seismic and retrofit design for underground structures.

INTRODUCTION

RC underground subway structures were suffered severe damage caused by strong motions at the1995 Hyogo-ken Nambu earthquake. These sorts of damage of RC underground structures were induced by liquefaction or lateral flow of surrounding grounds so far. However, liquefaction phenomena such as boiling sand or boiling

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water were not observed around the most part of soil layers adjacent to the damaged underground structures in Kobe. Therefore, numerical analyses and model or element tests for the damage were carried out to investigate primary causes of the collapse. As results of these investigations, major damage is induced by horizontal motions or shear deformation of ground, and it was shown that damage mechanism could be predicted by the equivalent linear method for dynamic response analysis. However, secondary damage such as surface cracks or total collapse including settlement of upper ground can not be represented by the equivalent linear method. The nonlinear response analysis method using direct integration method can play important role on carrying out a precise prediction for complex damage behaviour during earthquake. On the contrary, the equivalent linear method has advantage of few analytical parameters and limited analytical time.

This paper describes the applicability of nonlinear response analysis method for underground RC structures to seismic investigation.

OUTLINE OF EARTHQUAKE DAMAGE AND DIMENSION OF TARGET STRUCTURE

Figure 1 illustrates profile of Sannomiya station. The station has 3-story and 2-span's RC structure. Center columns that are located at every 5m of longitudinal direction support upper and middle slabs. The primary damage concentrated on center columns of the first story (**Figure** 2), while 2nd and 3rd story's steel columns were not suffered any damage.

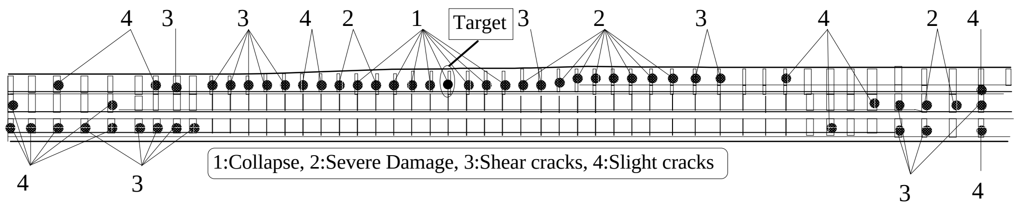

Figure 1: Profile of damaged station (Sannomiya station of Kobe municipal subway)

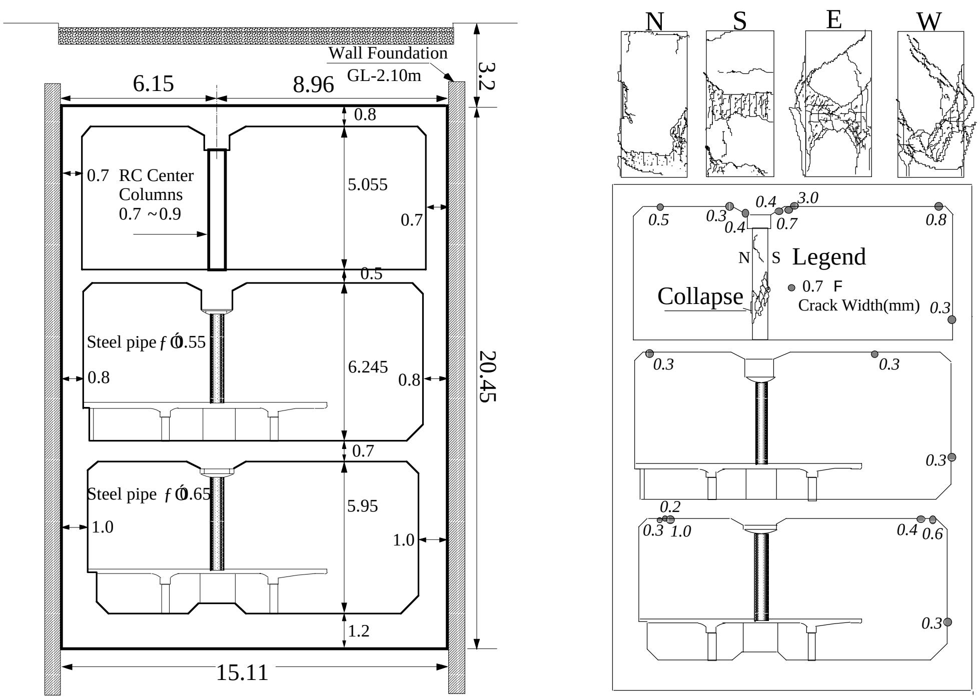

Figure 2: Damage pattern (Target section of the analysis)

Arrangement of steel bars is illustrated on **Figure** 3. Plan dimension of the center columns are 70cm width by 90cm length, and the major steel bar of center columns consists of deformed 25mm bar with 9mm round tie steel bar.

Soil properties of surrounding ground are tabulated on **Table** 1. These soil properties are evaluated by the as built document of the structures. Water table level is GL-3m, and comparatively stiff diluvial layer underlies under GL-5.2m level.

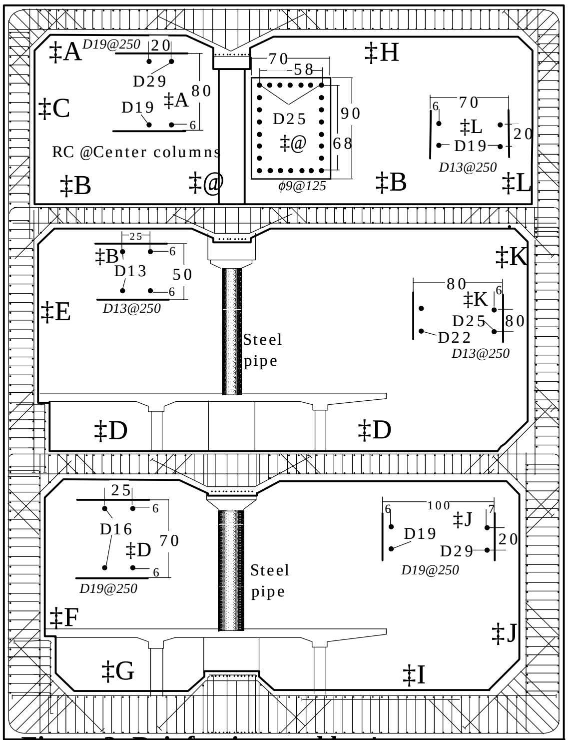

Figure 3: Reinforcing steel bar's arrangement

Table 1: Soil properties of horizontal layers

Layer	Thickness (m)	Depth (m)	N-Value	Soil	Vso (m/sec)	ƒÁ (tf/m^3)	ƒË	G$_0$ (tf/m^2)	
1	1.14	1.14	35		260	1.8	0.35	12400	
2	1.15	2.29	30		240	1.8	0.35	10500	
3	1.15	3.44	24		230	1.8	0.49	9700	
4	0.80	4.24	23		220	2	0.49	9800	
5	0.90	5.14	27		230	2	0.49	10700	
6	0.90	6.04	34	Sady Gravel	250	2	0.49	12700	Ag
7	0.90	6.94	44		280	2	0.49	16000	
8	0.91	7.85	56		300	2	0.49	18300	
9	0.90	8.75	78		340	2	0.49	23500	
10	0.90	9.65	44		280	2	0.49	16000	
11	0.65	10.3	41		270	2	0.49	14800	
12	1.01	11.31	40		270	1.9	0.49	14100	
13	1.01	12.32	44		280	1.9	0.49	15200	
14	1.01	13.33	34		250	1.9	0.49	12100	
15	1.01	14.34	33	Sand	250	1.9	0.49	12100	Ds
16	1.01	15.35	43		270	1.9	0.49	14100	
17	1.15	16.5	43		270	1.9	0.49	14100	
18	0.75	17.25	42		270	1.9	0.49	14100	
19	1.19	18.44	75		330	2	0.49	22200	
20	1.19	19.63	49	Sady Gravel	290	2	0.49	17100	Dg
21	1.18	20.81	44		280	2	0.49	16000	
22	1.19	22.00	90	Silt	350	1.8	0.49	22500	Dc
23	1.40	23.40	46		280	1.8	0.49	14400	
24	1.10	24.50	47		280	2	0.49	16000	
25	1.20	25.70	44		280	2	0.49	16000	
26	1.30	27.00	40	Sady Gravel	270	2	0.49	14800	Dg
27	1.50	28.50	83		340	2	0.49	23500	
28	1.50	30.00	60		310	2	0.49	19600	
29	2.00	32.00	44	Clay	350	1.8	0.49	22500	Dc
30	2.00	34.00	32		310	1.8	0.49	17600	
31	2.00	36.00	60		310	2	0.49	19600	
32	2.00	38.00	70	Sady Gravel	320	2	0.49	20800	Dg
33	2.00	40.00	80		340	2	0.49	23500	
34	2.00	42.00	80		340	2	0.49	23500	

ANALYSIS METHOD

Figure 4 shows flow of the present investigation consisted of following three parts.
(Input motion): 2-dimensional FEM deconvolution analysis using observed record at the 1995 Hyogoken Nambu earthquake (See **Figure** 6)

(Earthquake response analysis): Case-1= Equivalent linear method [FLUSH], Case-2= Direct integration method of nonlinear dynamic response analysis [EFECT; developed by Obayashi Corporation]
(Seismic stability evaluation): Ultimate states deign method based on the concrete design standard established by Japanese Civil Engineering Society.

FEM model for the analysis is illustrated in **Figure** 5. Two-dimensional plan-strain model with free field modeled by viscous boundary or energy transmitting boundary was adopted. The RC structure was represented by beam elements including center columns taking account of installation interval of 5m. Conjunction parts including hunches were represented by 100 times stiffer of normal elements.

Surrounding ground were assumed as horizontal layers, and initial soil properties were estimated using N-value listed on **Table** 1. G-γ and h-γ relationship for the equivalent linear method (Case-1) were determined using the results of element tests of soil sample logged at adjacent to the site (**Figure** 7). Modified Matsuoka model (a hyperbolic curve model) was used for nonlinear analysis (Case-1).

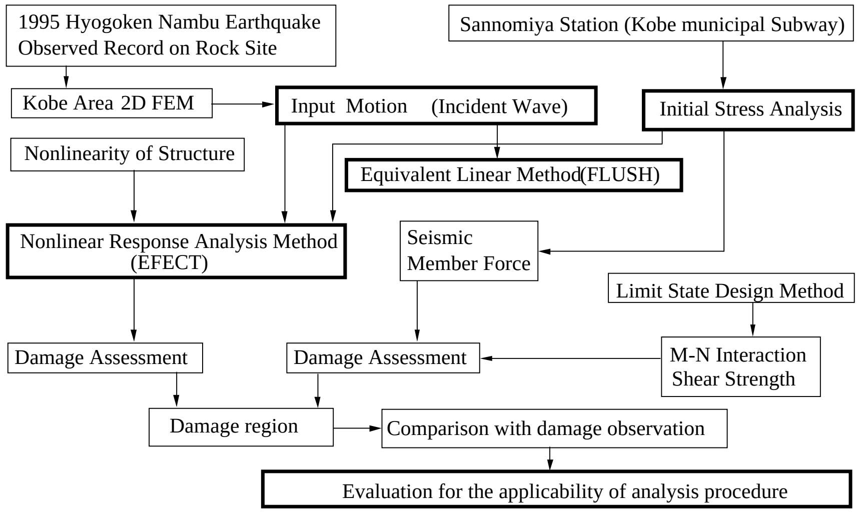

Figure 4: Analysis flowchart

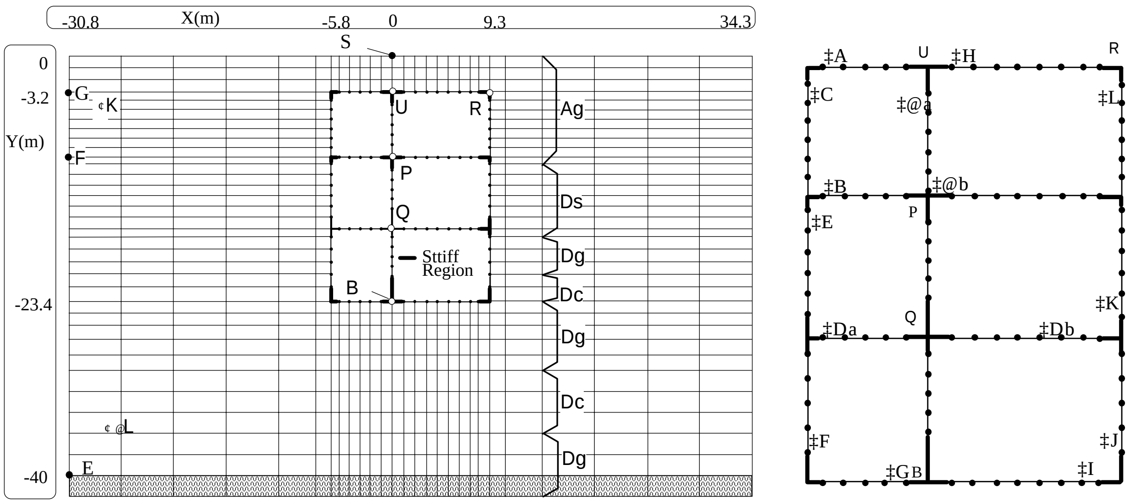

Figure 5: FEM model

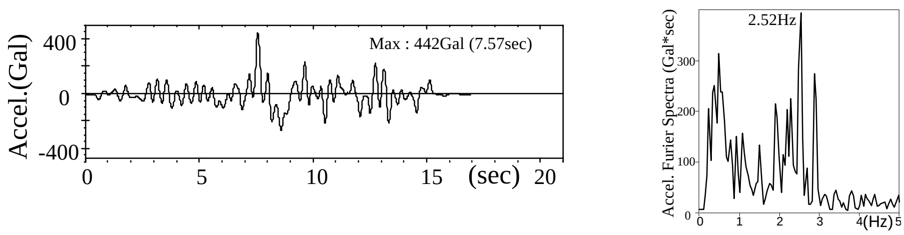

Figure 6: Input motion (2E)

Material properties of RC structure model are listed on **Table** -2. Critical damping ratio in Case-1 was set up to 5%, while the Rayleigh damping of 3% was assumed in Case-2. Nonlinear constitutive model of concrete member in the present method can take account of various axial forces with quadratic function for limit state moment (Eq.-1) and hyperbolic function for crack moment (Eq.-2).

$$Mu = -a_0 \times (N - N_0)^2 + M_0 \qquad (1)$$

$$Mc = b_0 \times N + M_1 \qquad (2)$$

Member forces computed by the dynamic analysis were incorporated into member forces of ordinary condition, which was assessed by initial stress analysis using same FEM model. Assessment of the seismic stability was estimated by comparison between total member forces of RC structure and the Limit State calculated by

Standard Specifications of Concrete established by Japan Society of Civil Engineers. For the limit state assessment, moment-axial force interaction (M-N curve, **Figure** 8) and shear strength were considered.

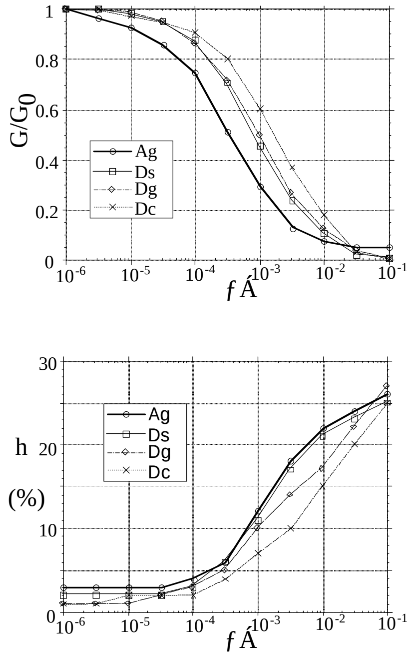

Figure 7: G-γ and h-γ relationship

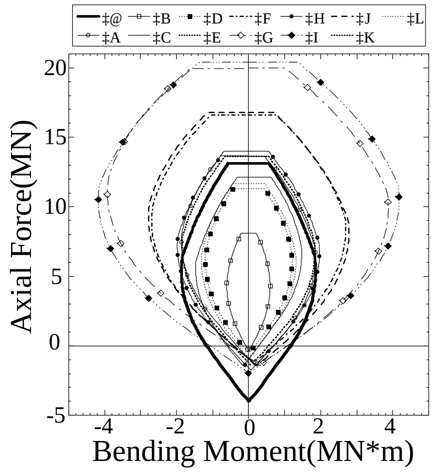

Figure 8: M-N interaction curve

Strain Criteria of concrete	ε'_{cu}	0.004
Compressive strength of concrete	f'_{cd0}	24 N/mm^2
Yeild Strength of steel bar	$f'_{\sigma yd0}$	3500 MPa
Elastic Modulus of steel bar	ε_{s0}	2.1*10^5 MPa

RESULT OF THE EQUIVALENT LINEAR METHOD

Figure 9 shows time histories of accelerations and displacement in Case-1. Maximum horizontal acceleration of the top slab is 703Gal, while maximum vertical acceleration is 26Gal. It is shown that the acceleration of structure indicates almost equal to that of same depth ground. **Figure** 10 shows the maximum distribution of acceleration and displacement. Relative displacement between top slab and bottom slab is 8.18cm and the average shear strain along the structure is 0.41%. It is noteworthy that deformation of the center columns is larger than that of sidewalls or surrounding ground, which induces damage. Maximum shear strain distribution of soil layers shown in **Figure** 11 indicates almost same as horizontal layer, and maximum value of shear strain of 0.51% is considered under the scope of applicability of the equivalent linear method.

Figure 12 shows maximum distribution of member forces. While dynamic component of axial forces at center columns accounts for 32% of ordinary condition, dynamic components of bending moments (M) and shear forces (S) predominate over those under ordinary condition. The relationship between member forces and relative displacement of upper slab from middle slab shown in **Figure** 13 indicates higher correlation coefficient, which gives a good example for the applicability of the response displacement method for underground structures.

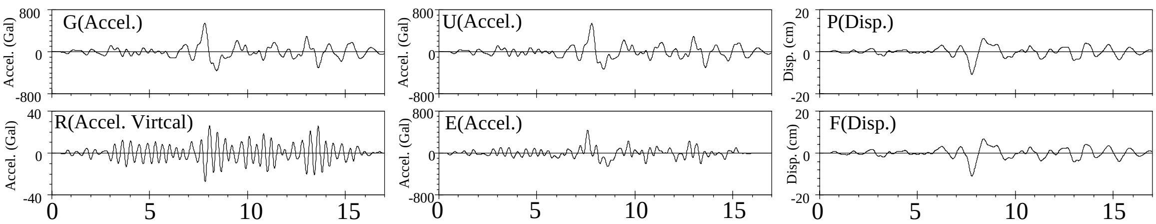

Figure 9: Time histories (Case-1, Equivalent linear method)

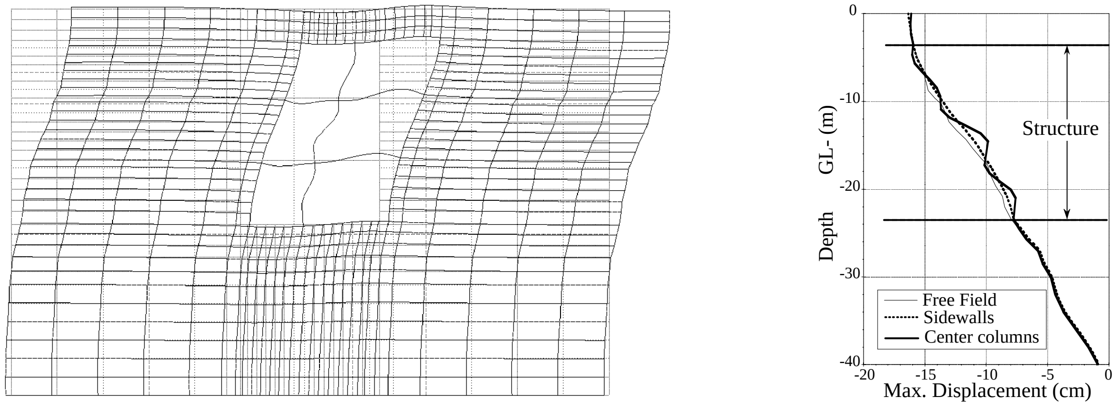

Figure 10: Maximum distributions of acceleration and displacement

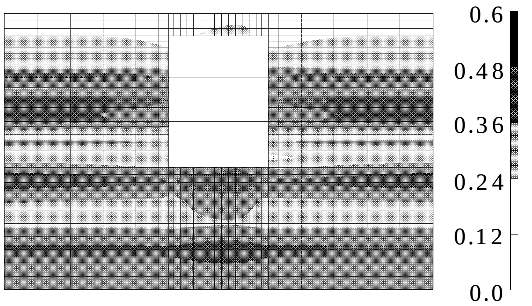

Figure 11: Maximum distribution of shear strain of the ground

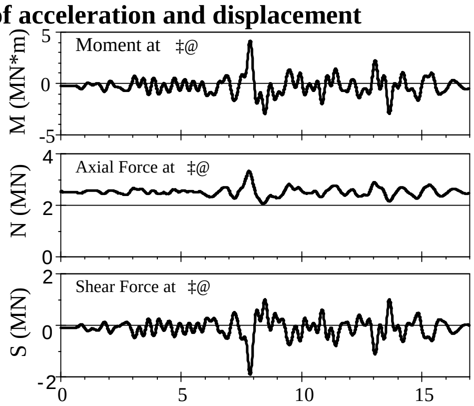

Figure 12: Time histories of member forces

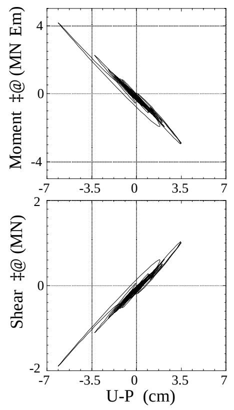

Figure 13: Relationship between member forces and relative displacement of structure

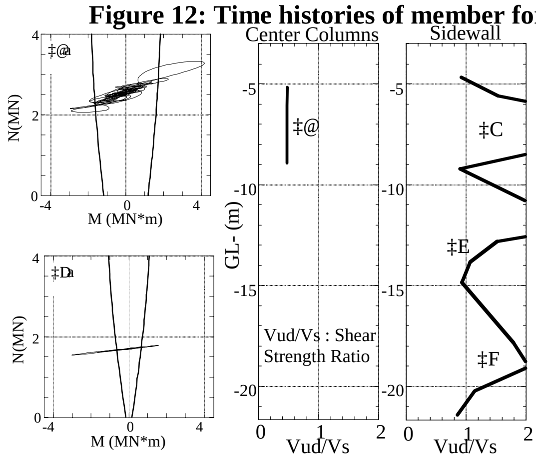

Figure 14: Shear strength ratio and M-N Interaction (Case-1)

M-N interaction shown in **Figure** 14 indicters that compressive failure of concrete due to axial forces did not occur while stress of steel bars exceeded over the tensile strength. **Figure** 15 shows the shear strength ratio Vud/Vs . Vud/Vs value of center columns falls below 1, which means possibility of collapse, the result of assessment of shear collapse coincides the damage observation.

Final failure assessment of the dynamic response analysis with the result of damage is shown in **Figure** 15. However, failure prediction result based on the equivalent linear response analysis shows tendency of overestimation, most damaged regions can be represented by the present method.

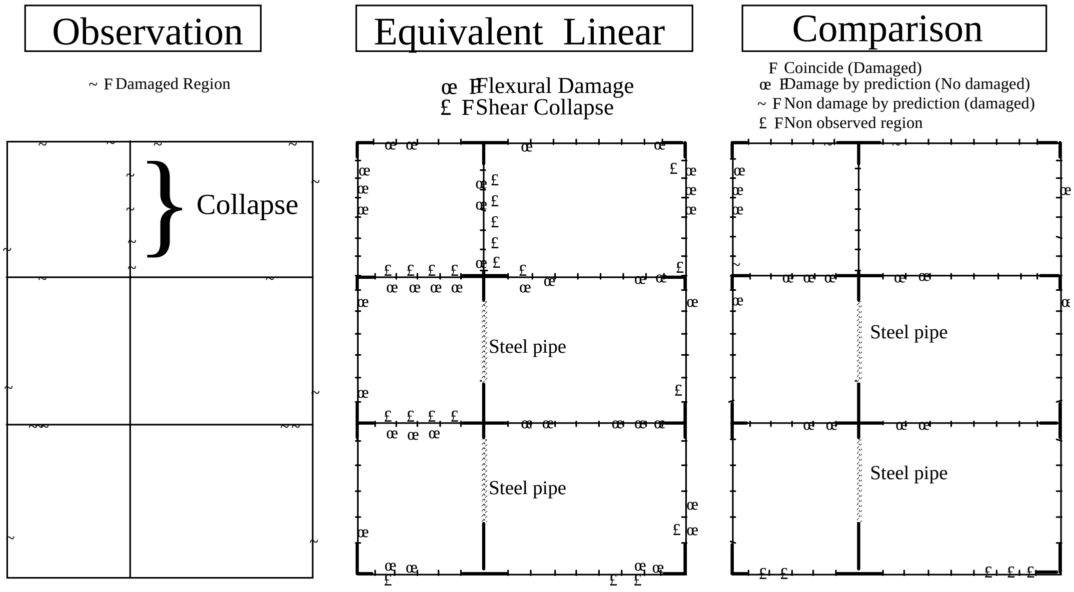

Figure 15: Final failure assessment (Case-1)

RESULT OF THE NONLINEAR RESPONSE ANALYSIS (CASE-2)

Figure 16 shows relationships between shear strain and stress of soil layers. Major skeleton of the nonlinear analysis coincides with equivalent shear rigidity of Case-1. However historical damping factor is larger than equivalent complex damping ratio, so that shear strains and stresses of Case-1 indicate larger values than those of Case-2.

Comparison between Case-1 and Case-2 of acceleration and displacement is shown in **Figure** 17. Difference between Case-1 and Case-2 is small, it is thought that nonlinearly or plastic deformations of structures hardly affect to responses of underground structures. **Figure** 18 shows the relationship between bending moment and rotation angle (M-ϕ). After the bending moment exceeds the Limit State, M-ϕ pass traces nonlinear loops, however, plasticity ratio dose not attain to that of 4 level.

Final failure assessment of the nonlinear response analysis with the result of damage is shown in **Figure** 19. Failure prediction ability of the nonlinear analysis is superior to that of the equivalent linear method. However, cracks on the upper slab adjacent to the columns can not be represented by both methods, it was thought that large deformation of center columns due to the collapse affects to those cracks. Nonlinear deformation analysis using FEM model (see **Figure** 5) was conducted to investigate the effect of plastic deformation of the columns (**Figure** 20). Forced displacement was subjected to the conjunction node. Fig-20 shows the final deformation pattern of the structure and **Figure** 21 shows member forces-settlement relationship, respectively. Before settlement reaches to 5mm, it is estimated that cracks appear on the upper slab, however, whole element does not attain to the Limit State.

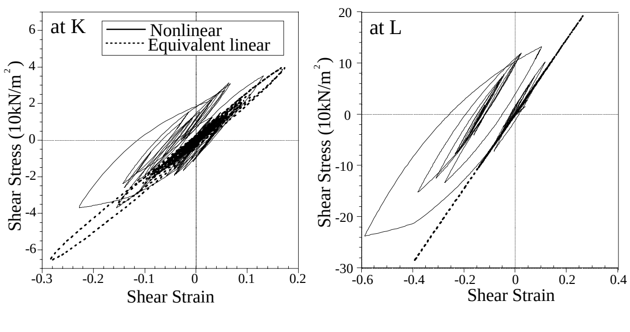

Figure 16: Shear strain vs. stress

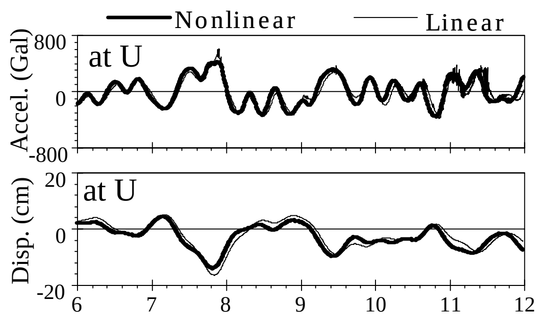

Figure 17: Time histories (Case-2)

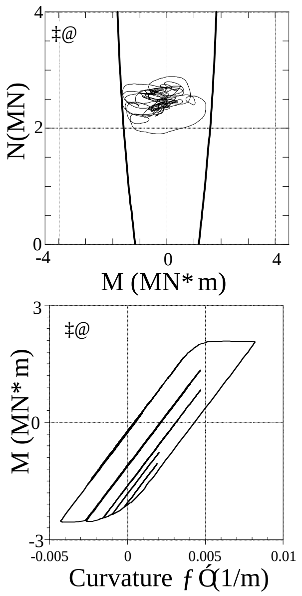

Figure 18: M-N and M-ϕ relationship (Case-2)

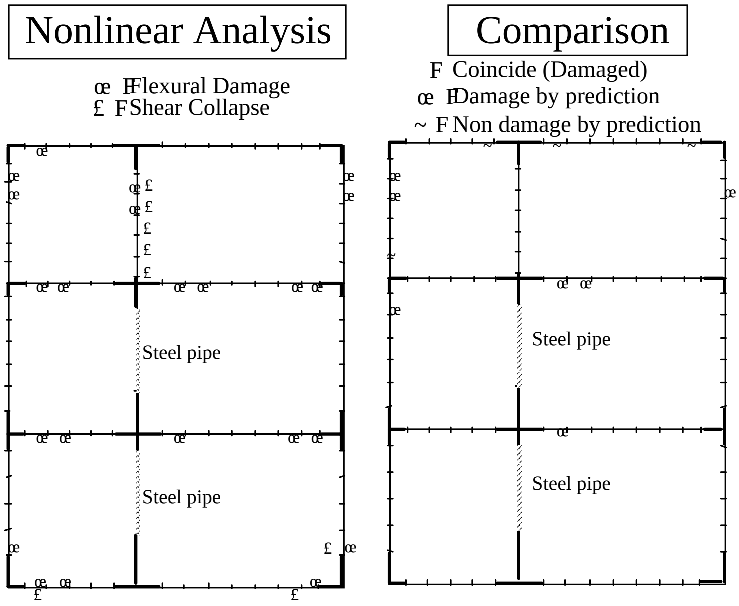

Figure 19: Final failure assessment (Case-2)

Two-dimensional FEM dynamic response analysis methods were adapted to predict seismic damage. The equivalent linear method is comparatively easy to treat, however, it has tendency to give an overestimation. On the other hand, the nonlinear analysis method can predict damage regions and extent precisely. Predominant damage can be represented by dynamic analysis, however, it is difficult to predict secondary damage such as cracks due to large seismic damage. In this case, forced deformation analysis is effective tool to estimate final damage.

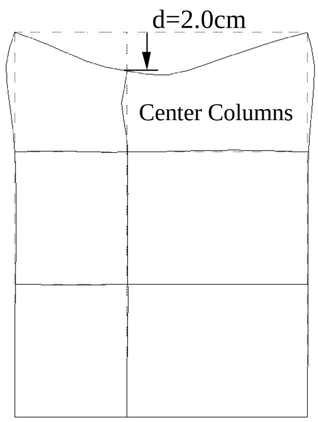

Figure 21: Relationship between member forces and settlements

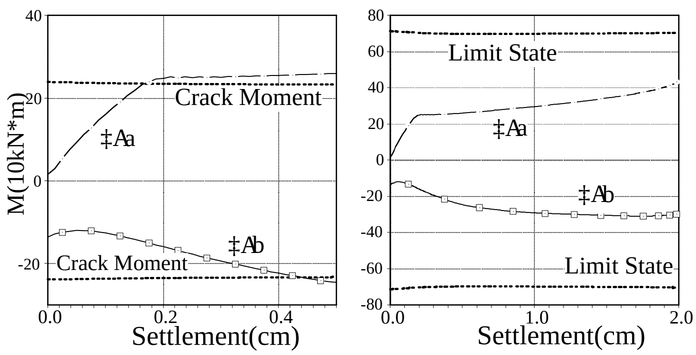

Figure 20: Nonlinear deformation analysis model and deformation (Case-2)

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