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ABSTRACT

Under destructive earthquake motions, structures behave heavily in nonlinear manner that is quite different from the linear response computation. The underground structures such as tunnels, ducts are susceptible to the ground motions so that the nonlinear interaction analysis should be performed for the reliable design. In case of irregular soil profile, due to the surface wave generation, the vertical motions come out to a less negligible extent compared to the horizontal motions. The present paper has concerned with such nonlinear response evaluation of a duct structure under strong motions. The effect of the transient characteristics in the inland type and ocean type earthquake motions is investigated. The effect of the vertical component in the ground motions is also evaluated.

INTRODUCTION

After the Hyogoken Nanbu earthquake (1995), the ground motion of the inland earthquake type is specified for the severe earthquake loading (Level 2) which is considered for aseismic design of structures. The new seismic code for railway structures[1] specifies the response spectrum as Spectrum I for ocean type earthquakes and the Spectrum II for inland type earthquakes.

Duct structures, embedded in ground, are severely subject to the surrounding ground motions. The soil deformation method that imposes the seismic load due to the horizontal soil response is commonly used for the practical design procedure. A heavy damage was reported at the Daikai Station in the Hyogoken Nanbu earthquake[2]. The seismic design requirement is therefore intensified since then. The failure check is requested in two steps that depend on the degree of seismic loading. In view of the importance of the vertical seismic motion at irregular base formation, the effect of combined input motions of horizontal and vertical components is often investigated in the nonlinear soil and structure interaction behavior.

In this paper, focusing on the duct structure as shown in Fig.1, the dynamic response is investigated with respect to the phase effect and direction characteristic of input motions. This figure shows the double-box type cross section of a duct embedded shallowly in soft ground. The cross section in Fig.2 is design based on the soil deformation method in view of the Spectrum I for the seismic motion of Level 2 for railway structures. The Spectrum I is based on the ocean type earthquakes whose characteristic in comparison to Spectrum II is depicted in Fig.3.

The soil condition at site is described as in Table 1. The top layer is composed of soft sandy silt so that the nonlinear characteristic is accounted by modeling Ramberg-Osgood type hysteresis. This is given in Fig.4 for the equivalent properties.

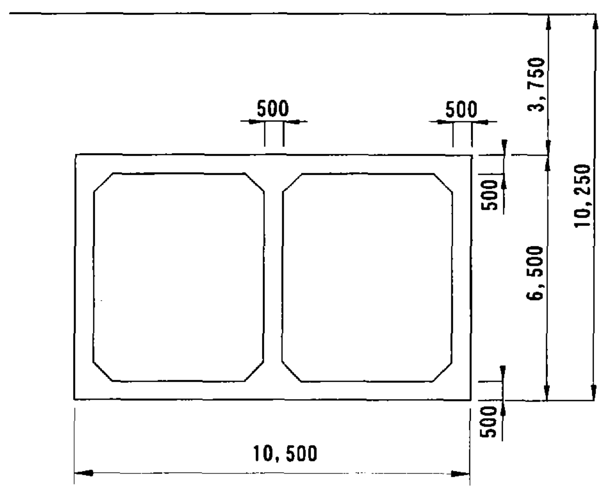


Fig.1 General cross section of an embedded duct structure

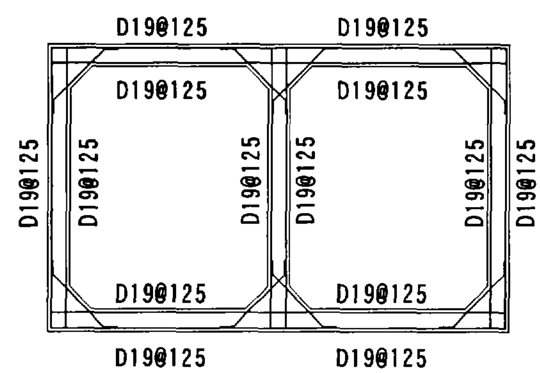


Fig.2 Design cross section of an embedded duct structure

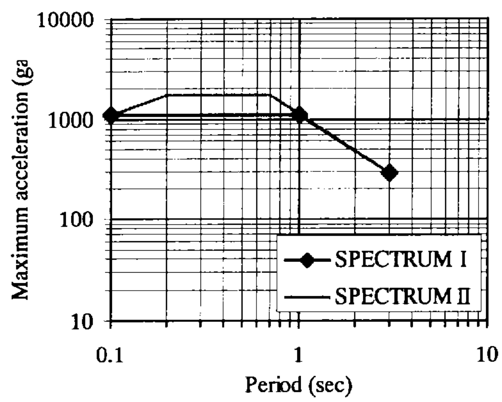


Fig. 3 Response spectrum for railway structures

Table 1 Soil properties

Layer	Thickness (m)	Shear velocity Vs (m/s)	Unit weight γ (kN/m^3)	Poisson ratio ν
Layer1	20	126	18	0.49
Layer2	28	400	19	0.49

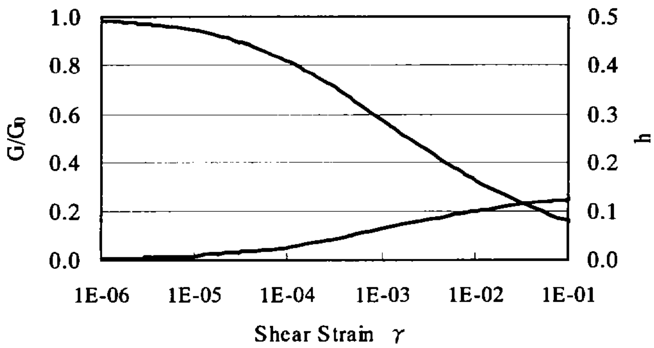


Fig.4 Nonlinear soil characteristic

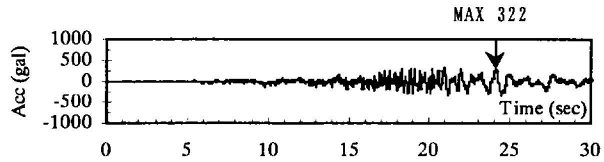
1-D DYNAMIC SOIL RESPONSE ANALYSES

Seismic analyses are conducted for different input motions by using typical earthquake records. These are classified in Table 2. The acceleration input motions, as adjusted to show fitting the Spectra curves of Spectrum I, and Spectrum II in the seismic code for railway structures are depicted in Fig. 5. The Hachinohe record has relatively long period motions because of the deep sediment site. The Port Island has very long period by the liquefaction at site. For use of the vertical motion, time histories are halved.

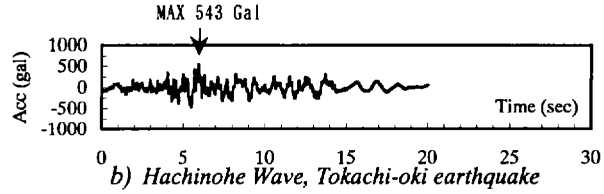
Table.2 Cases of a dynamic analyses

Case	Amplitude	Phase	Vertical effect
1-A	Spectrum I	Specified	–
1-B	ibid	Hachinohe	–
2-A	Spectrum II	Specified	–
2-B	ibid	Port Island	–
2-C	ibid	Specified	Considered

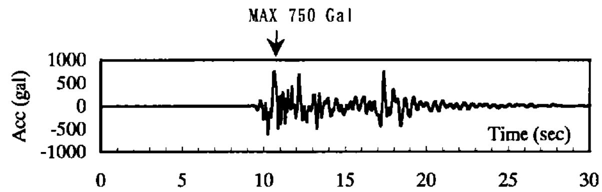
First, one-dimensional dynamic analysis is conducted in order to make clear the effect of the respective input motions. Fig. 6 is the maximum response profiles. It is noted that the order of the input motions that give rise to the greater response is case 1-A < case 1-B ≈ case 2-b < case 2-A.



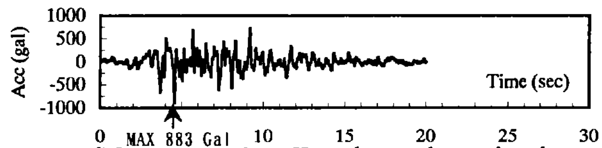
a) Standard Wave (Spectrum I)



b) Hachinohe Wave, Tokachi-oki earthquake (Spectrum I)

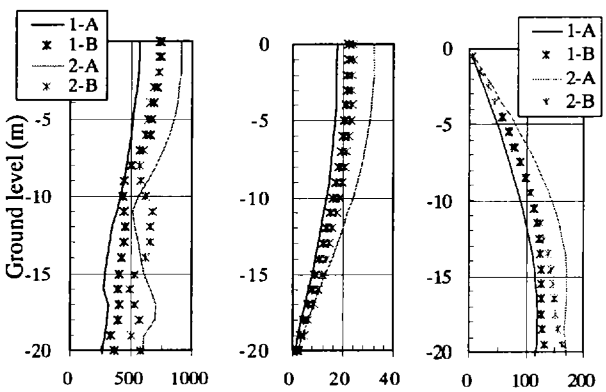


c) Standard Wave (Spectrum II)



d) Port Island wave, Hyogoken nanbu earthquske (Spectrum II)

Fig.5 Input Acceleration time histories



a) Acceleration (gal) *b) Displacement (cm)* *c)Shear force (kN/m^2)*

Fig.6 Maximum response profiles from 1-D analysis

2-D DYNAMIC SOIL-STRUCTURE ANALYSIS

The 2-dimensional model of plane strain condition is used as illustrated in Fig. 7 for the FEM analysis. The nonlinear behavior is taken into account by the Ramberg-Osgood hysteresis model. Only the updating of the shear modulus is searched by iteration for meeting the shear strain level. (The considered duct was shown in Fig. 1 and Fig. 2. The duct wall is modeled by a beam with the tri-linear concrete whose properties are specified as in Fig.13 by ○for the concrete.

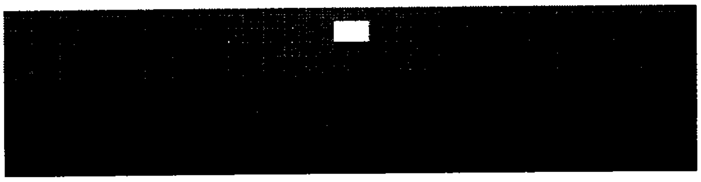


Fig. 7 FEM model for analysis

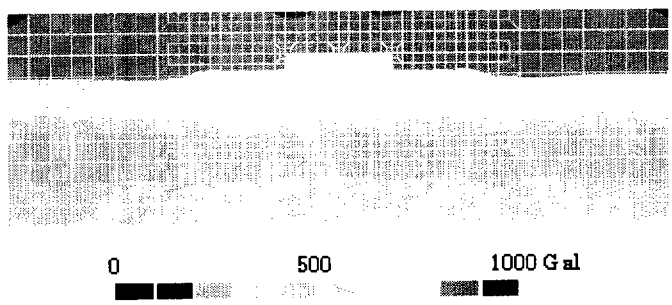


Fig.8 Maximum horizontal acceleration of case 2-A

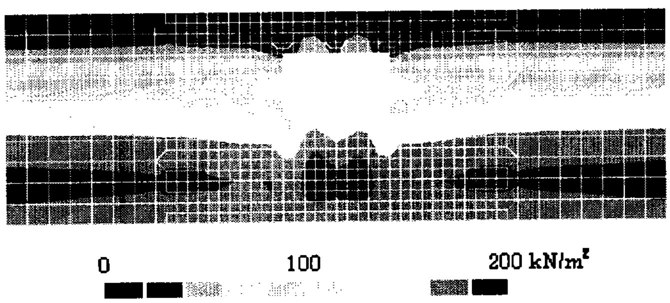


Fig.9 Maximum shear stress of case 2-A

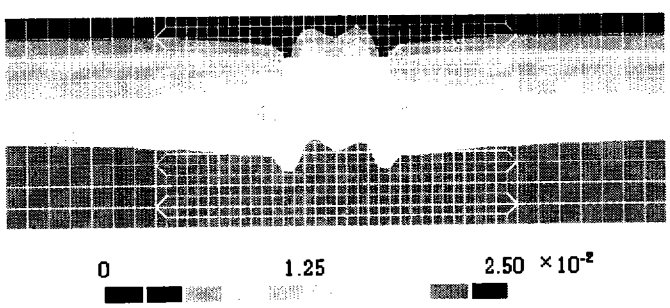
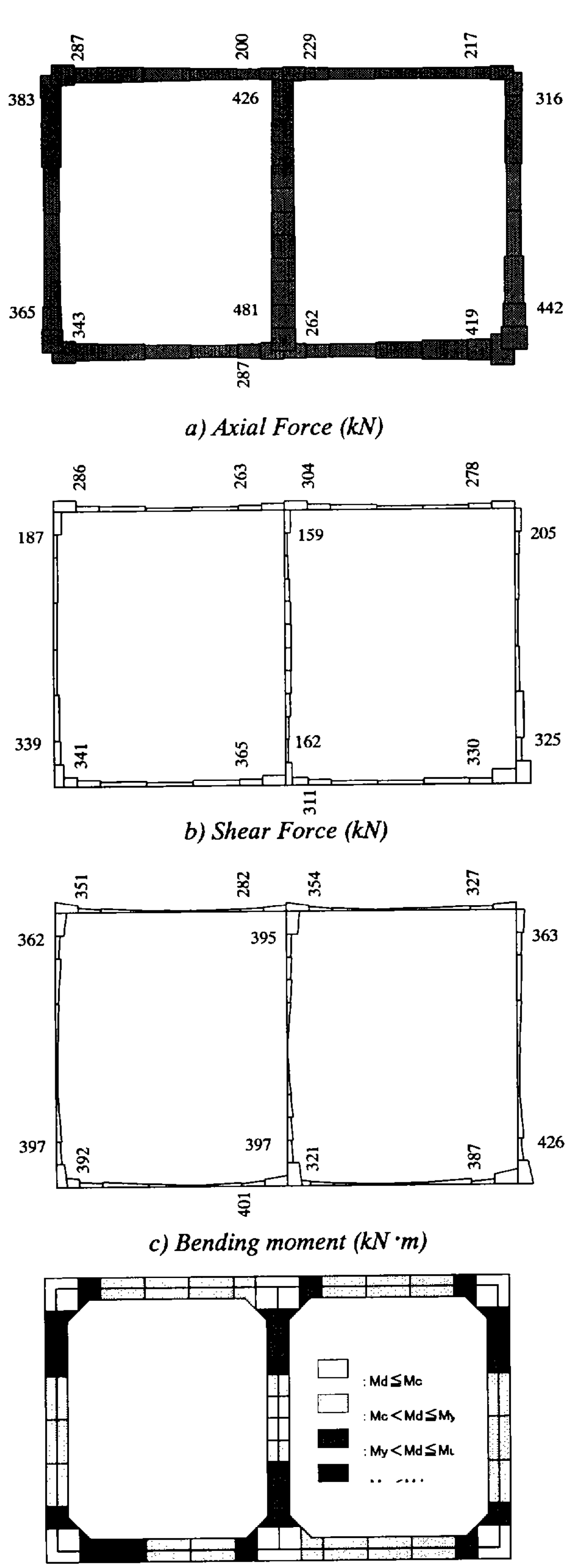


Fig.10 Maximum shear strain of case 2-A



a) Axial Force (kN)

b) Shear Force (kN)

c) Bending moment (kN·m)

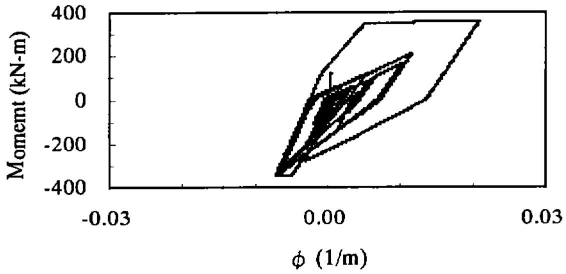
d) Bending moment resistance

Fig.11 Maximum internal forces (case 2-A)

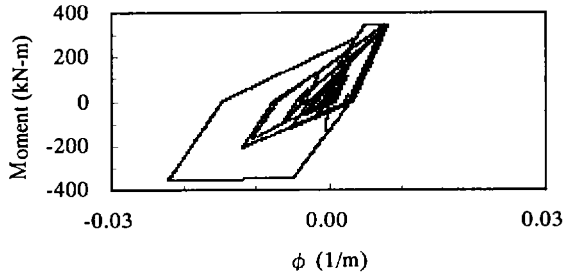
crack, reinforcement yielding and member collapse

According to the results of analyzed cases, the maximum results correspond to the case 2-A; therefore, only its results are interpreted below. The maximum horizontal acceleration, shear stress and strain intensities are depicted in Fig. 8, Fig.9 and Fig.10, respectively. The influence zone by the duct structure is limited within the double of the structural dimensions in size.

The maximum internal forces are indicated in Fig.11. The maximum values appear at corners of the structures. The concrete cracking occurs almost every section except the middle portion of the inner wall. The bending moment exceeding the yield moment is noted at corner haunches but yet to reach the failure moment. Fig. 12 depicts the bending moment-curvature hysteresis loop.



a) Inner Wall (Top)



b) Inner Wall (Foot)

Fig.12 Moment-curvature loop of case 2-A

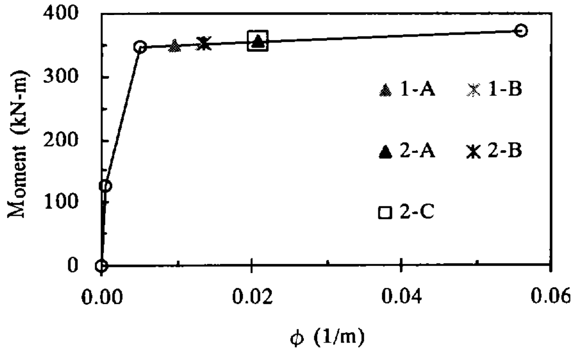
COMPARISON BETWEEN DYNAMIC ANALYSIS AND PSEUDO-STATIC ANALYSIS

The soil deformation method resulted in the internal forces that exceeds the failure value at the inner wall for the case 2-A. However, the dynamic analysis gave the smaller value than it. The difference is interpreted that the former method is based on the 1-D equivalent linear soil analysis that give the larger soil deformation than the 2-D analysis.

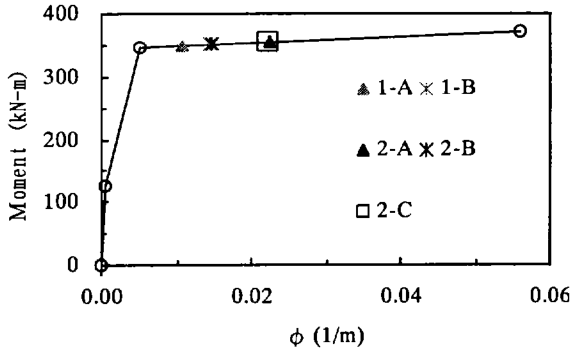
Table.4 Maximum internal forces from soil deformation method

	Case 1-A			Case 2-A		
	N	S	M	N	S	M
Top slab	221	277	340	273	284	352
Bottom slab	470	397	415	672	545	457
Outer wall	495	400	413	707	540	455
Inner Wall	496	173	432	499	191	428

Note: N(kN), S(kN), M(kN·m) in units



a) Inner Wall (Top)



b) Inner Wall (Foot)

Fig.13 M- ϕ hysteresis loop(2-A) (???)

CONCLUSION

In this paper an embeded duct structure is analyzed by taking account of the nonliear characteristics of soil and RC members. The design values based on common procedure, namely by the soil deformation method are compared with those computed from the 2-D dynamic analysis. The effect of the vertical seismic motion resulted in the larger axial forces while the bending and shear forces remain less affected. .

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3) Yamato, T., Umehara,T., Oki,H., Nakamura,S., Ezaki,J. & Suetomi,I.: Damage to Daikai subway station of Kobe rapid transit system and estimation of its reason during the 19565 Hyogoken-nanbu earthquake, Proc. JSCE 537/I-35, 3030-320, 1996.

Table.3 Maximum internal forces

	1-A			1-B			2-A			2-B			2-C		
	N	S	M	N	S	M	N	S	M	N	S	M	N	S	M
Top slab	220	290	351	245	296	351	295	304	354	249	297	352	318	303	354
Bottom slan	354	330	392	378	352	390	419	365	424	368	344	396	492	364	427
Outer wall	368	321	393	395	362	395	442	339	426	382	360	392	515	347	432
Inter Wall	466	227	397	480	287	399	481	346	417	483	268	399	677	347	417

Note: N(kN), S(kN), M(kN·m) in units.