On Impact-like Failure of Reinforced Concrete Structures by Hyogo-ken Nanbu Earthquake (Kobe, 1995)[1]

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Synopsis: First, impact-like failure phenomena observed in Hyogo-ken Nanbu Earth-quake (Kobe, 1995) are described. Second, stress wave transmission characteristics within a multi-layered ground excited by vertical ground motions are discussed through one-dimensional stress wave propagation analyses. Third, a stress amplification phenomenon in the cross section of column in a bridge R/C pier under a pulse-like vertical stress induced at the bottom face of footing slab is examined by 3D FEM analyses. Last, 3D FEM analyses for a bridge pier model supported by multi-layered ground model are carried out, and it was made clear that high frequency characteristics of vertical ground motions during earthquake gave the most significant effect on an impact-like failure of R/C column due to large axial significant tensile forces.

***Keywords*:** *Hyogo-ken Nanbu earthquake, impact failure, civil engineering structures, 3D FEM analysis, stress concentration phenomenon*

1. Introduction

On 17th January 1995, a great earthquake hit Awaji and Hanshin area in Hyogo prefecture in Japan. Registered magnitude of the earthquake was M7.2 in the Richter scale, which was not the largest among recent earthquakes in Japan, but it gave severe and fatal damage to residential buildings, bridges, subway stations and so on. Since the focus of the earthquake was not deep (about 14km below the ground level), these structures had to undergo near field strong ground motions during this earthquake. Since just after this earthquake, the authors have been convinced that the vertical ground motions have a great effect on the damage of the above mentioned structures. After the earthquake, however, many structural engineers have investigated the failure mechanism through response analyses of the structures damaged using both horizontal and vertical strong ground motion records by the seismographs, and in consequence, at present, it has become widely accepted that the effect of vertical component of ground motions was not so large and the horizontal component was predominant for the fracture of structures.

But, the authors have suspected this conclusion, because conventional seismic response analyses for structures are based on a vibration theory for lumped mass models, but stress wave propagation characteristics between structural foundation and ground are not considered in these analyses. Stress waves travelling throughout ground starting from a deep base stratum during earthquake are refracted at an interface of structural foundation, and then refraction waves propagate within a structure and they also repeat reflection and refraction at its boundary planes. In consequence, if a large stress concentration occurs on a part of structural members due to its geometrical condition, impulsive failure such as spall fracture in tension, impact buckling in compression and so on may be caused.

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This paper is intended for revealing interaction problems between ground and structure during strong vertical ground motions and for examining the possibility of impulsive failure of structural members during earthquake.

2. Impact-like Failure Phenomena Observed

Photo 1 shows the horizontal cracks observed in the columns of a reinforced concrete bridge pier, which seemed to be caused by impact tensile forces [1]. Though the cracks of large width are not many, a number of parallel small width cracks can be seen over the whole region of the column. The degree of damage in bridge piers very varies from the earlier stage of only small horizontal cracks to the fatal stage of collapse of the column. In the columns suffered from a small damage, some of horizontal cracks were only seen, but in the columns damaged more, the axial reinforcing bars buckled by the repeated action of compressive forces after the tensile cracks occurred and the concrete cover flew out (see **Photo 2**).

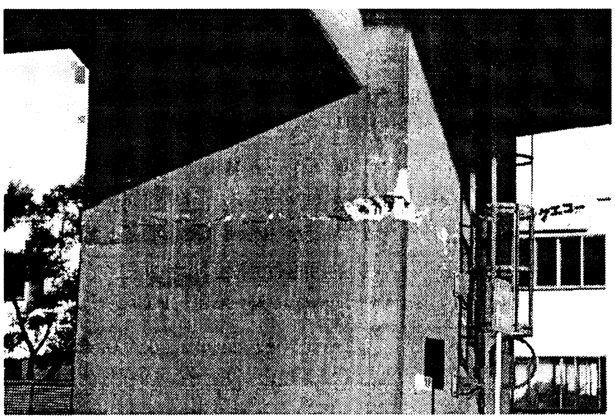

Photo 1 Tensile failure mode due to impulsive force in a bridge R/C pier

Photo 2 Complex failure mode by tensile and compressive forces

On the other hand, some of steel tube bridge piers buckled by undergoing strong vertical compressive forces. For example, **Photo 3** shows a buckling mode for circular steel tube piers. In these damaged piers, an appreciable horizontal deformation of the columns was not seen, it is therefore considered that the vertical impact forces at the earlier stage of earthquake is a main factor to cause the fatal damage of the bridge columns in cooperation with following horizontal shaking.

Other phenomena seemed to be impulsive failure mode were also observed in the collapse mode of an intermediate R/C column at subway stations, the brittle fracture mode of ductile iron tube of bridge piers, tensile cracks of R/C piles, the fracture mode of shoes to support bridge girders and so on.

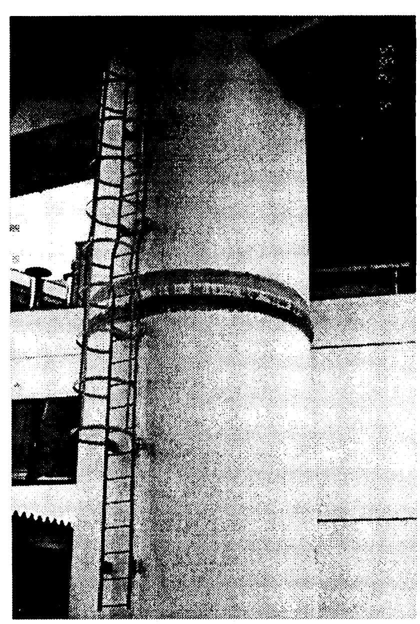

Photo 3 Buckling mode of circular steel tube bridge pier

3. Vertical Component of Observed Ground Motions

Seismographs were installed at ten and several stations within Kobe-Osaka area. The maximum acceleration recorded was 818gal in N-S component at Kobe Ocean Meteorological Observatory station which was located about 20km north east from the epicenter. The maximum acceleration and velocity of vertical component at the same station were 332gal and 40kine, respectively. **Figure 1** shows the response spectra of N-S, E-W and U-D components [2]. The seismograph used at the station was of an acceleration type, in which sampling time interval of digital data was 0.01sec and cut frequency of low-pass filter was 10Hz. As seen from **Fig. 1**, predominant frequencies of both N-S, E-W and U-D components are almost the same being around 1Hz, and so much difference among characteristics of these components can not seen.

However, there were two important evidences [3], one was presented by an official of the above mentioned observatory who was working at his office just at the earthquake. For an interview of Kobe Newspaper, he answered that, with other two officials staying the office, they were initially thrust upward and then were horizontally shaken, and they saw falling down of heavy instruments of 29kg. The other was presented by a woman who was living in front of the above mentioned observatory. She answered to an interview of a magazine that she and her parents were initially jumped up by an explosive sound of "dohoon" and then they went out during strong shaking. These evidences seem not to match the ground motions recorded by the seismograph. Therefore, the authors have thought that the seismograph might lose high frequency components of vertical ground motions due to its frequency characteristics.

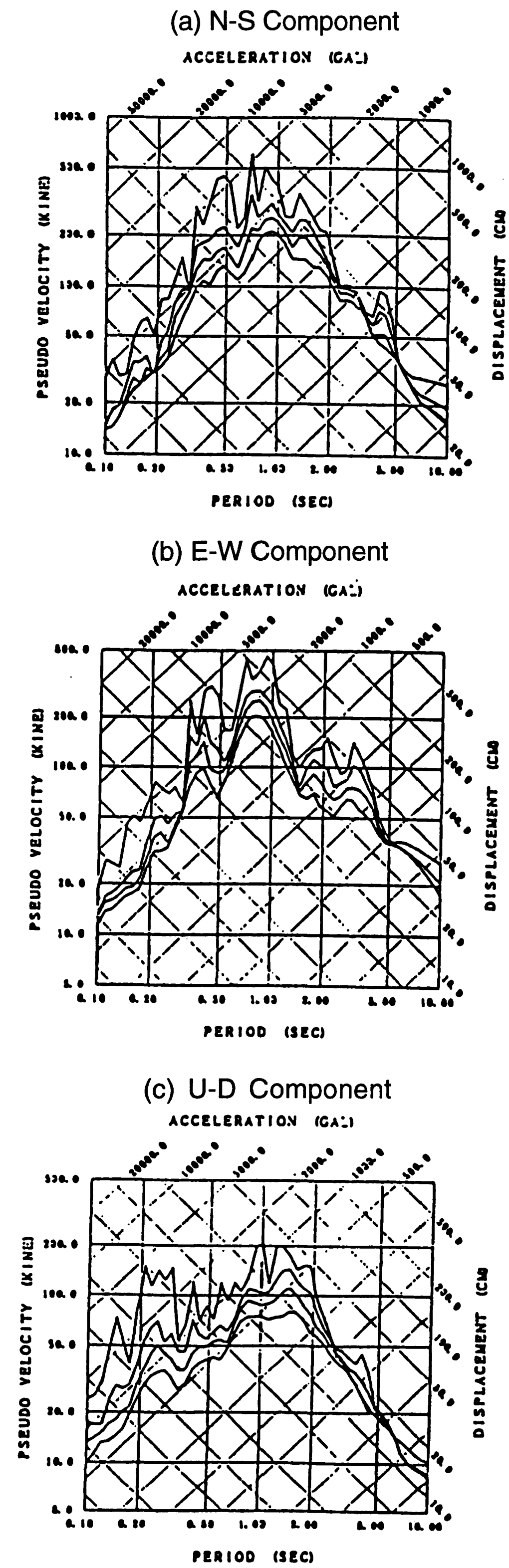

Fig. 1 Response spectra (Kobe Ocean Meteorological Observatory station) [2]

4. Stress Wave Propagation Characteristics Within Multi-layered Ground

First, when a multi-layered ground model is subjected to a vertical ground motion at its bottom layer, how much intensity of vertical stress is developed within each layer would be examined. If the bottom layer is sufficiently so deep that a stress wave within ground has not an influence of reflection from its boundary layers, relationship between particle velocity v and vertical stress σ_z at any point of ground can be given by

$$\sigma_z = -\rho C v \qquad (1)$$

where C is velocity of longitudinal elastic wave and ρ is mass density, and ρC is called characteristic impedance. But, at a point near ground level, where a seismograph is usually installed, Eq. (1) can not be generally applied, because of an influence of wave reflection at the ground level. Generally, a footing of structures such as pile and caisson is supported on a diluvium deposit such as sand or gravel stratum through an alluvium deposit such as soft clay near ground level. Therefore, it is important to evaluate a stress propagation characteristics of these strata to study an interaction problem between ground and structures.

Figure 2 shows a model of multi-layered ground, in which the first and second strata are assumed to consist of alluvium deposit and diluvium deposit, respectively and the base stratum is a rock. Now, let us consider that the bottom base of this model is subjected to a pulse-like vertical motion given by

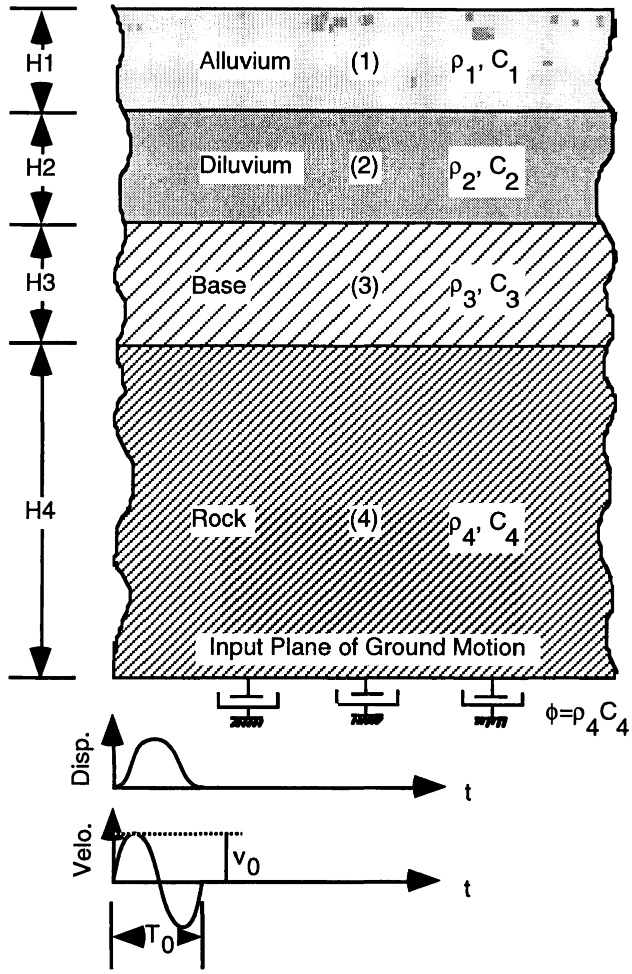

Fig. 2 Multi-layered ground model

Displacement:

$$w = \frac{T_0}{2\pi} v_0 \left[1 - \cos\frac{2\pi t}{T_0}\right], \quad 0 \le t \le T_0 \qquad (2)$$

Velocity: $$v = v_0 \sin\frac{2\pi t}{T_0}, \quad 0 \le t \le T_0 \qquad (3)$$

where v_0 is the maximum intensity of input vertical velocity, t is time, and T_0 is duration of incident displacement (also equal to period of input velocity), and denoting C as longitudinal wave velocity, CT_0 corresponds to wave length. Namely, the bottom base is enforced by Eqs. (2) and (3) in a range of $0 \le t \le T_0$, and in a range $t > T_0$ it is dealt with as a boundary plane with viscosity factor of $\rho_4 C_4$ to eliminate reflection effect as much as possible.

Numerical calculations are carried out under the following conditions for each layer:

Alluvium deposit: H_1 =20m, ρ_1 =1.6t/m^3, C_1 =362m/sec

Diluvium deposit: H_2 =20m, ρ_2 =2.0t/m^3, C_2 =1,024m/sec

Base stratum: H_3 =20m, ρ_3 =2.0t/m^3, C_3 =3,240m/sec

Rock stratum: H_4 =200m, ρ_4 =2.0t/m^3, C_4 =3,240m/sec

Figures 3a show time histories of vertical stress and velocity at the center of base stratum (point P3),

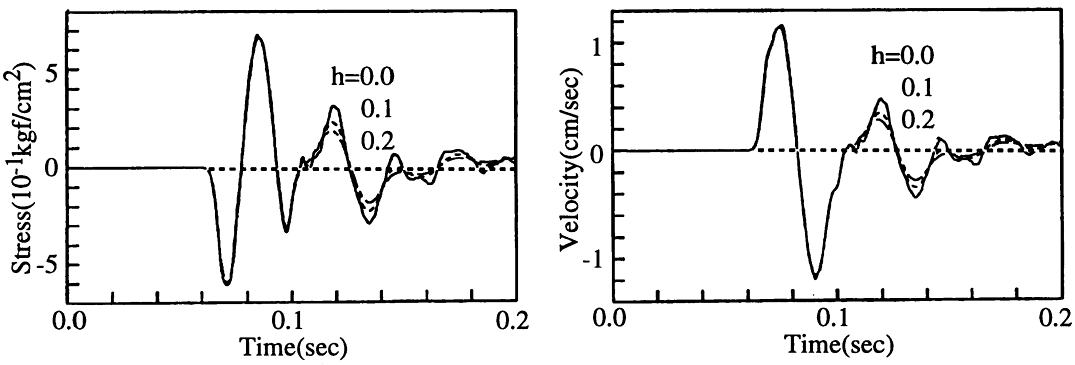

Fig. 3a Time histories of vertical stress and velocity at the center of base stratum (point P3)

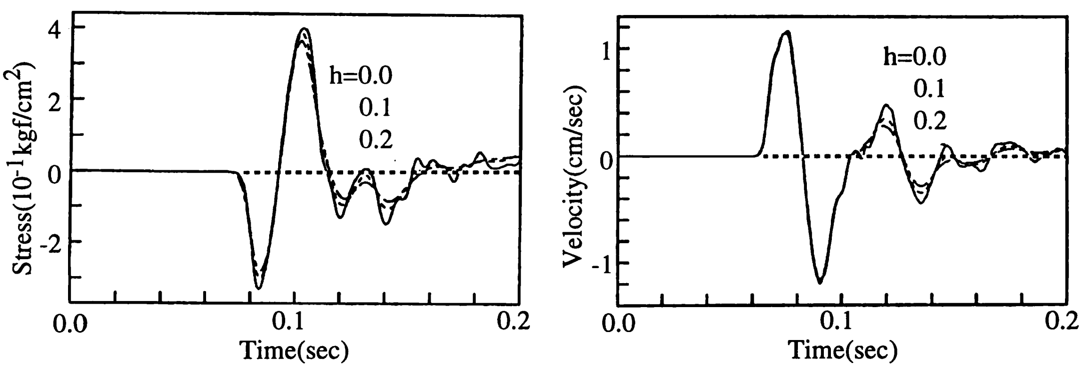

Fig. 3b Time histories of vertical stress and velocity at the center of diluvium deposit (point P2)

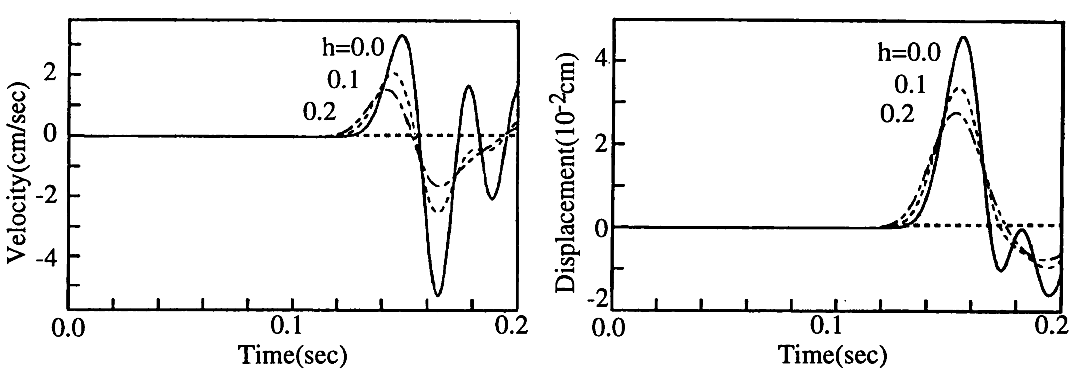

Fig. 3c Time histories of velocity and displacement on the ground surface

Figures 3b show them at the center of diluvium deposit (point P_2), and **Figures 3c** show time histories of velocity and displacement at the ground level. All the results were obtained under v_0 =1cm/sec and T_0=0.032sec. The viscous damping is taken into account by a dashpot in Kelvin Model, and its factor h is taken as a ratio to $\sqrt{2}\ \rho_i c_i$. As shown in the time histories obtained under h=0.0, 0.1 and 0.2, almost the same response at points P_3 and P_2 is obtained in both the stress and the velocity. The effect of viscous damping is larger in the high frequency range than in low frequency range. The maximum intensity of vertical stress obtained almost becomes the value estimated by Eq. (1) at point P_3, but at point P_2 it becomes somewhat greater than that. On the other hand, the response of velocity and

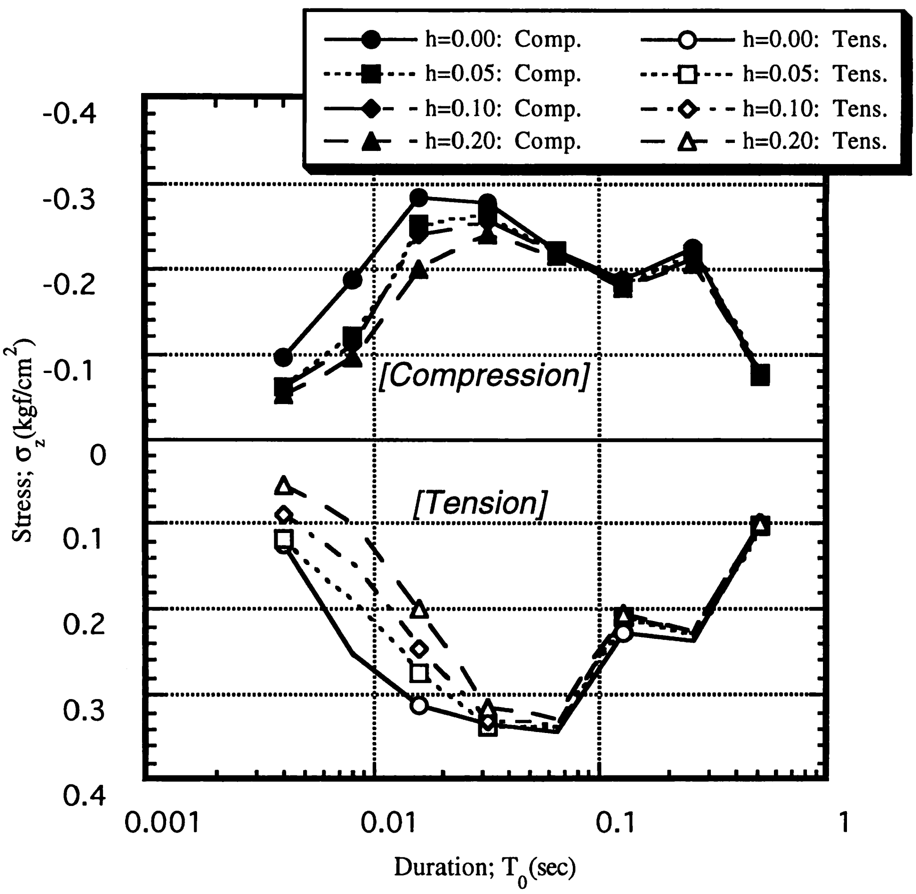

Fig.4 Maximum stress response spectra at the center of diluvium deposit

displacement at the ground level are significantly influenced by the viscosity damping, as about a half reduction of the maximum values can be seen in comparison of the both cases of $h=0.0$ and $h=0.2$.

Figure 4 shows relationships between the maximum vertical stress intensity and the duration of input displacement enforced, T_0, in which the stress intensities are normalized to a unit velocity at the center of base stratum, P_3. The maximum response appears at $T_0=0.016\sim0.032$sec in compressive stress and at $T_0=0.064$sec in tensile stress. If natural period of multi-layered stratum, T_e, is approximately calculated as

$$T_e \fallingdotseq 4\sum_i \frac{H_i}{C_i} \tag{4}$$

where H_i is depth of each stratum and C_i is longitudinal wave velocity, $T_e=0.594$sec for 5-layer model of **Fig. 2** and $T_e=0.2$sec for the first layer only considered in **Fig. 2**. Therefore, it can be known that the predominant frequency range of vertical stress within a near ground surface considerably differs from that of vertical ground motion, and this range is much higher than the predominant frequency observed by a seismograph as shown in **Fig. 1**.

5. Stress Concentration Phenomena in a R/C Column of Bridge Pier

Here, a stress concentration phenomenon in a column of bridge pier would be discussed from a viewpoint of stress wave propagation. **Figure 5** is a simple model of bridge pier with only a top slab and a footing slab. In the model, our concern is placed on how much stress concentration occurs in the column section under a unit vertical stress induced at the bottom of the footing slab. For the sake of this, 3D Finite Element elastic analyses with an explicit time integral scheme were done. The incident stress at the bottom face of footing is of a pulse-like sinusoidal half wave with a unit maximum intensity as shown in **Fig. 6**. The duration of incident stress, $T_0/2$, varies from 0.008sec to 0.25sec.

Figure 7 shows the amplification factor of vertical stress at the center of column, in which the factor is significantly depending on T_0 and the maximum factor at $T_0 \fallingdotseq 0.08$sec approaches to the value of 36, which may correspond to the ratio of the area of footing slab to that of column cross section. Deviating from $T_0 \fallingdotseq 0.08$sec, the factor of tensile stress amplification rapidly decreases, but the factor of compressive stress amplification approaches to a constant value being a half of 36, when T_0 becomes very larger than 0.08sec. The latter tendency can be regarded as a predominant influence of rigid motion. On the other hand, an eigenvalue analysis for the bridge pier model of **Fig. 5** was done, and the first order eigenvalue is found to be 0.0123rad/sec in angular velocity which corresponds to 0.0775sec in period.

From the above mentioned observations, we can conclude that there exists a predominant period for the amplification of stress in a column of bridge pier, which might be regarded as a phenomenon of resonance related with an extensional vibration mode of column.

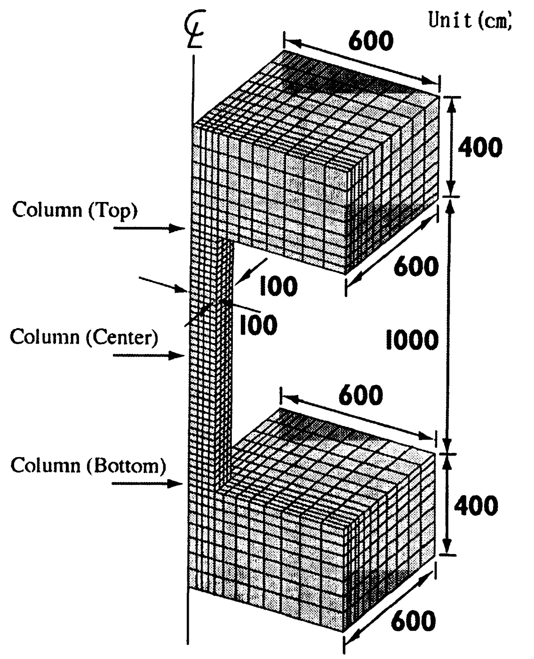

Fig. 5 Simplified bridge pier model and 3D FEM mesh division (1/4 space)

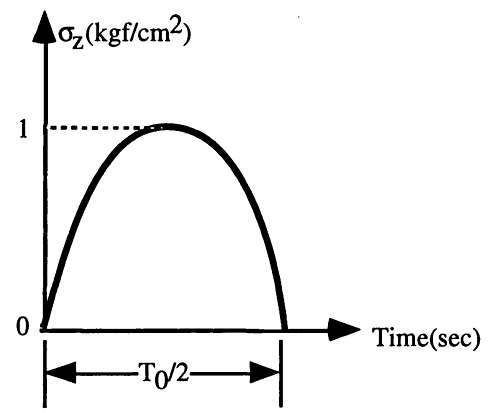

Fig.6 Sinusoidal half wave stress induced

6. Response Analyses of Bridge Pier Considering Ground Motion

Here, we deal with stress transmission problems to a bridge pier due to a vertical ground motion. A bridge pier model consists of a R/C top slab, a R/C column, a R/C footing, R/C piles and a multi-layered soil ground. **Figure 8** shows 3D FEM mesh used. An incident ground motion at the bottom face of base stratum is a pulse-like vertical ground motion corresponding to a sinusoidal velocity given by Eqs. (2) and (3). The material constants used for the bridge pier and ground are as follows:

Base stratum: $\rho = 2\text{t/m}^3$, $C = 4{,}180$m/sec

Diluvium deposit: $\rho = 2\text{t/m}^3$, $C = 1{,}320$m/sec

Alluvium deposit: $\rho = 1.63\text{t/m}^3$, $C = 148$m/sec

Concrete: $\rho = 2.3\text{t/m}^3$, $E = 3\times10^5\text{kgf/cm}^2$ (29.4GPa), $\nu = 1/6$

where ρ, C, E and ν are mass density, longitudinal wave velocity, elastic modulus and Poisson's ratio,

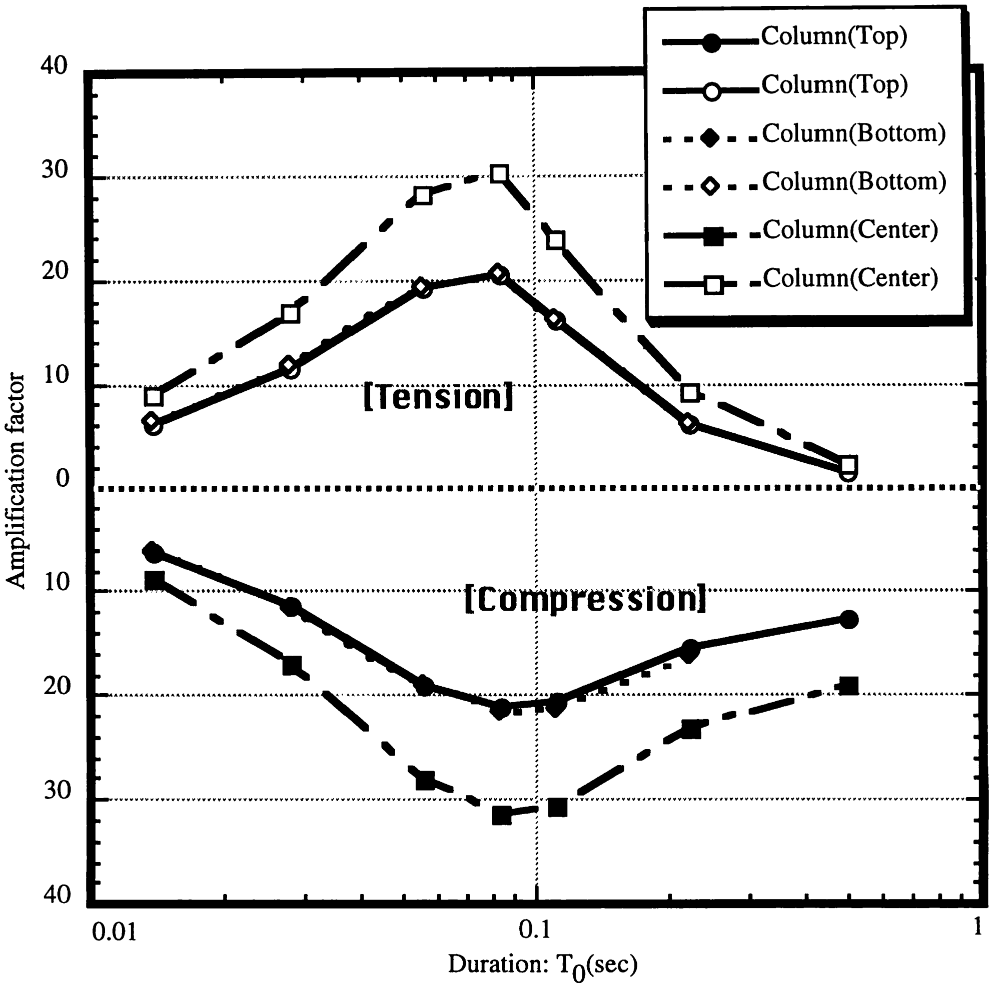

Fig.7 Amplification factor of vertical stress intensity in column

respectively.

Figure 9 shows maximum vertical stress response spectra obtained at the center of the diluvium deposit under unit maximum incident velocity, v_0=1cm/sec in Eqs. (2) and (3). The maximum stress intensity can be seen at the duration of enforced vertical displacement, T_0=0.1sec for compressive stress and T_0=0.2sec for tensile stress. Since the incident stress intensity calculated by Eq. (1) is equal to 0.85kgf/cm^2, the maximum stress intensity in the diluvium deposit is somewhat less than the incident stress intensity. Comparing this fact with the results in **Fig. 4**, we can say that a heavy bridge pier loaded on the ground surface does not so much influence on the maximum vertical stress response in the diluvium deposit. On the other hand, **Fig. 10** shows the maximum vertical stress response spectra in the piles and the column under the same incident velocity, v_0 =1cm/sec. The maximum stress intensity can be seen at T_0=0.08～0.09sec in these figures. The maximum stress intensity within the column appears at the edge of bottom end cross section, whose value attains to 17kgf/cm^2 (1.7MPa) in tensile range and 13kgf/cm^2 (1.3MPa) in compressive range, though the average stress in the whole cross section do not vary between the center and the bottom of column. From these results, it can be pointed out that if the tensile strength of concrete is assumed to be 30kgf/cm^2 (3.0MPa), an initial crack occurs in the R/C column under the incident ground velocity, v_0= 1.7cm/sec and the duration, T_0=0.09sec.

From the recorded ground motion in this earthquake which was already mentioned before, we can not read the high frequency characteristics of the vertical ground motions, whose period is less than 0.1sec, because ordinary seismographs installed were designed for the frequency range being of 0.1~10Hz. Therefore, we can not conclude that the R/C bridge columns placed within the meizoseismal region collapsed by the impact-like vertical ground motions with high frequency, but we can point out the possibility of that, considering the before mentioned evidences by a lot of people living within the meizoseismal region.

7. Concluding Remarks

In Hyogo-ken Nanbu earthquake, a lot of phenomena seemed to be an impact-like failure mode were observed, but these phenomena can not be explained by an ordinary seismic response analysis based on the ground motion waves which were read from the seismographs.

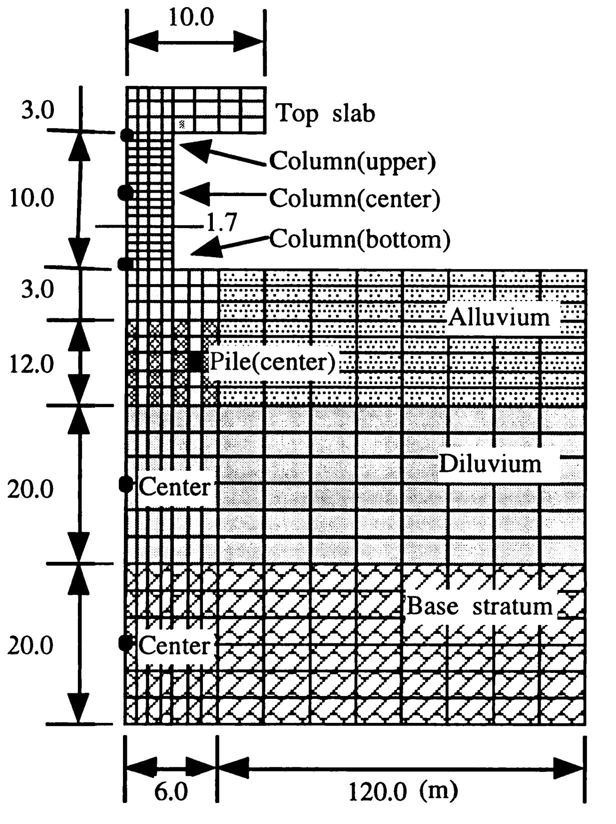

Fig.8 Bridge pier model supported by a multi-layered ground and 3D FEM division (1/4 space)

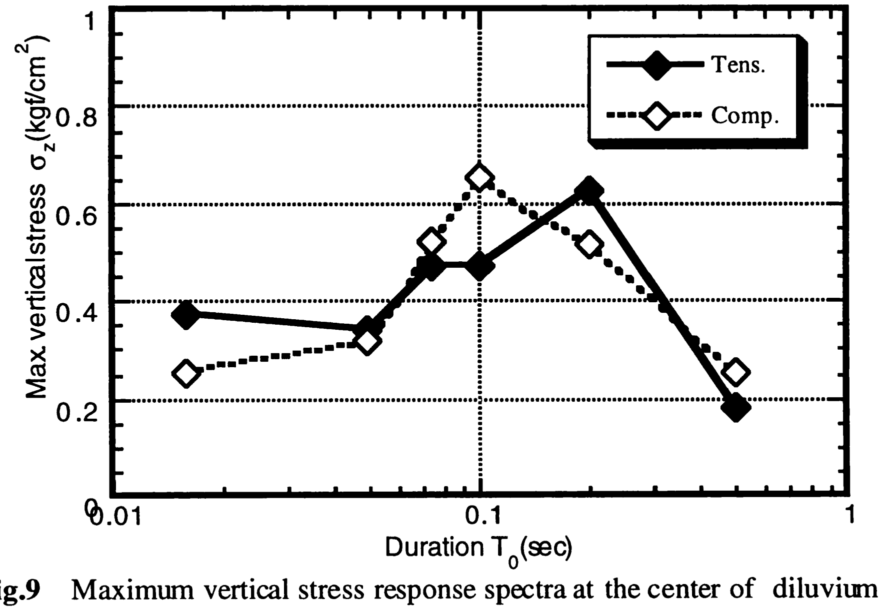

Fig.9 Maximum vertical stress response spectra at the center of diluvium deposit

This paper has challenged to make these phenomena clear and pointed out the impotance of high frequency characteristics of vertical ground motions for this problem, though those remains unclear in the ground motions recorded by the seismographs. Since impact-like failure phenomena of R/C bridge piers had first appeared in this earthquake, there have been a lot of problems to be solved in the future. We hope that this paper becomes a motivation forfurther discussions on these problems.

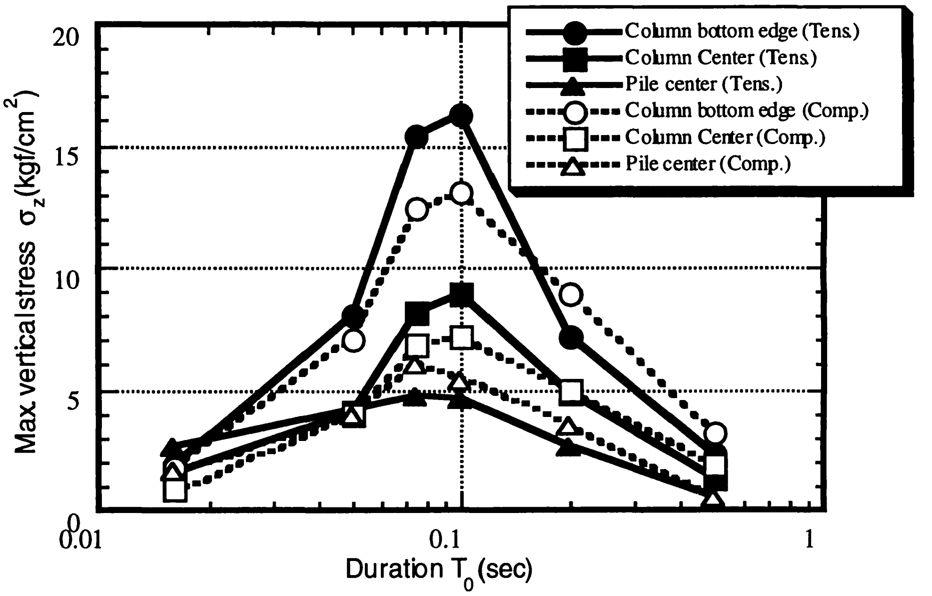

Fig.10 Maximum vertical stress response spectra at the column and pile

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