NONLINEAR ANALYSIS OF REINFORCED CONCRETE VIADUCTS SUBJECTED TO SEISMIC LOADS USING 3D LATTICE MODEL

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Abstract: Analytical study for the seismic response of RC structures subjected to earthquake motion is presented. Dynamic analysis using the 3D lattice model is performed on the actual RC structures. The 3D lattice model can offer the reasonable prediction of the shear-carrying capacity of RC columns and beams. Analytical targets are two RC rigid-frame viaducts damaged at the 1995 Hyogo-ken Nanbu Earthquake. The results of dynamic lattice model analysis are compared with the actual damages of RC viaducts. The comparison reveals the reliability of the 3D lattice model at the structural system level. It is also found that the analysis can predict the damages including both the shear failure in the columns and the buckling of reinforcement in RC columns. In addition, the influence of the vertical motions on the maximum displacement and the instable behavior due to large vertical load is clarified.

1. INTRODUCTION

Hyogo-ken Nanbu Earthquake, which occurred at January 1995 in Japan, caused the destructive collapse to various structures, including reinforced concrete (RC) structures. The observations following this earthquake show that the main causes of severe damage were due to the overestimation of both shear carrying capacity and ductility of structures. It is also found that the structures had insufficient amount of transverse reinforcement. On the other hand, it is noticed that there were many RC structures without almost any damages. The difference of the degree of damage is influenced by several factors even if the dimensions of structures and the arrangements of reinforcement are almost identical.

Many investigations for the damage of structures and the damage analyses have been carried out (Ishibashi and Okamura 1998, Committee 311 2000). The seismic performance evaluation was also performed by the frame analysis based on the moment-curvature restoring characteristics or the analysis using the fiber technique (Tsuchiya et al. 1999, 2000). The shortcoming of these models is the difficulty in predicting the behavior in the post-peak region, especially, when RC members fail in shear. In these models, moreover, the shear deformation and torsional behavior cannot be directly predicted because the in-plane deformation is not taken into account.

In this study, the 3D lattice model is used. The 3D lattice model is the objective and simple procedure, which can explain the shear resisting mechanism for RC members appropriately. The seismic response of RC rigid-frame viaducts is evaluated using the 3D nonlinear dynamic lattice model analysis. Here, the RC rigid-frames in railroad viaducts are selected as the analytical targets. The input ground motions determined by FDEL (Sugito et al. 1994) are used. The analytical results are compared with actual damages of viaducts. The interaction between the soil and the structures is not considered in the analysis.

2. ANALYTICAL MODEL

2.1 Outlines of 2D Lattice Model

The 3D lattice model is developed based on the concept of the 2D lattice model. The 2D lattice model consists of members of concrete and reinforcement, as shown in **Figure 1**. The column subjected to load from left hand side is shown. With the RC column, the concrete is modeled into flexural compression members, flexural tension members, diagonal compression members, diagonal tension members, horizontal members and two arch members. The longitudinal and the transverse reinforcements are modeled into vertical and horizontal members, respectively. The characteristic of 2D lattice model is to incorporate the diagonal tension members and the arch members of concrete into the modeling. This point is different from the conventional modified truss model. The diagonal members are regularly located with the inclined angles of 45 and 135 degrees to the longitudinal axis of the column, respectively. The arch members connecting the nodes at the opposite diagonal corners between the loading point and the bottom of the column are arranged according to the direction of internal compressive stress flows.

Figure 2 shows a schematic diagram of cross section of RC column. The concrete is divided into the truss part and the arch part. When the value of t is defined as a ratio of the width of arch part to the width of cross section, b, the widths of the arch part and the truss part are given by $b \times t$ and $b \times (1 - t)$, respectively, in which $0 < t < 1$. The value of t is determined based on the theorem of the minimization of the total potential energy for the 2D lattice model with the initial elastic stiffness. The total potential energy is obtained from the difference between the summation of the strain energy in each element and the external work. The pre-analysis using the 2D lattice model is carried out for the small provided displacement at the loading point, where the value of t has changed from 0.05 to 0.95 with an interval of 0.05. The strain energy of the 2D lattice model is accumulated one of each lattice component.

2.2 Configuration of 3D Lattice Model

In the 3D lattice model, it is assumed that the shear resisting mechanism is divided into arch action and truss action. With the 2D lattice model, the 3D RC member is modeled into 2D based on the assumption of plane stress. As illustrated in **Figure 3**, in the 3D lattice model of an RC column, four arch members are arranged connecting between the top and the end of the column at the opposite corners. The resisting mechanism of RC column subjected to one certain load

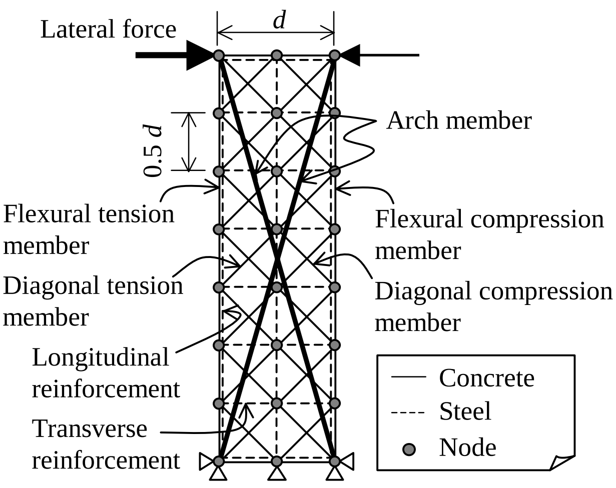


Figure 1 Outlines of 2D lattice model

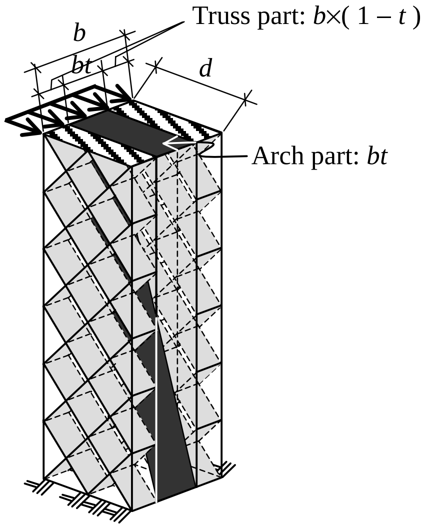


Figure 2 Cross section of RC column modeled by 2D lattice model

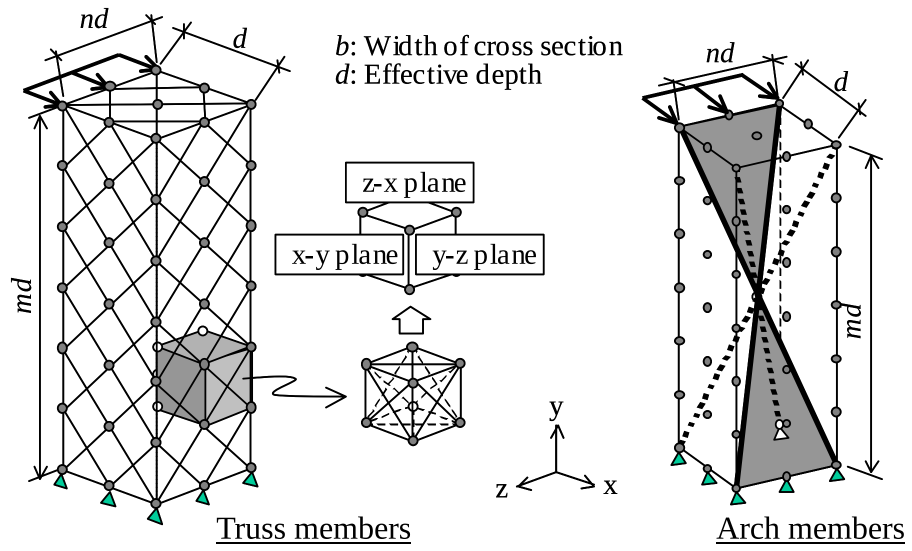


Figure 3 Discretization for 3D lattice model

consists of two arch members crossing each other. The stiffness of these arch members is assumed to be equivalent to that of two arch members in 2D lattice model. With the RC column subjected to load from the diagonal direction, it is assumed that the corner-to-corner arch action inside the RC column is idealized as a compressive strut that is represented by these arch members in the 3D lattice model.

To represent the truss action, it is assumed that 3D space is comprised of an orthogonal coordinate system that is defined by three planes, x-y plane, y-z plane, and z-x plane. Two crossed truss members are located on each truss plane so that unit element consists of 12 truss members in six truss planes as shown in **Figure 3**. In each truss plane, the in-plane 2D constitutive law of concrete, considering the softening of the compressive strength for diagonally cracked concrete depending on the transverse tensile strain, can be used.

In 2D lattice model, the value of t is defined by a ratio of the arch part width to the cross-section width of RC member as mentioned previously. Based on the assumption that the global stiffness of 3D lattice model is equivalent to one of 2D lattice model, the cross-sectional area of the arch member can be calculated. Here, the ratios of the arch part width to the width and the depth in the cross section of the column are defined as t_b and t_d, respectively. With the determination of values of t in 3D lattice model, not only the width of cross-section of the column but also the depth of cross-section of the column is varied in the pre-analysis. According to the 2D lattice model, in the 3D lattice mode the values of t_b and t_d are determined based on the theorem of minimization of the total potential energy.

2.3 Material Constitutive Models

In order to consider the effect of lateral confinement of concrete due to suitable arrangements of transverse reinforcement, the stress-strain relationship of confined concrete proposed by Mander et al. (1988) is applied to the diagonal compression members and the arch members. For cracked concrete, the compressive softening behavior of concrete proposed by Vecchio and Collins (1986) is considered. The ability of cracked concrete to resist compressive stress decreases as the transverse tensile strain increases.

With the flexural compression member, which is assumed to represent the cover concrete, the stress-strain relationship represented by the quadratic curve (Vecchio and Collins 1986) is used. With the concrete model under cyclic loading, the stress is assumed to decrease with the initial stiffness in the unloading path. In the reloading path, the stress is assumed to proceed to the previous maximum state along with same path as unloading.

For the flexural tension members of concrete, which are located near reinforcing bar, the concrete continues to contribute the tension force even after cracking due to the bond effect between the concrete and reinforcing bars. Therefore, the tension-stiffening model (Okamura and Maekawa 1991) is applied. On the other hand, for the diagonal tension members that consist of concrete far from reinforcing bar, after the crack occurs, the concrete can be assumed to show a tension strain-softening behavior. Hence, the tension softening curve, so-called 1/4 model (Rokugo 1989), is applied. The fracture energy of concrete G_F, which is the area under the tension softening curve, is assumed to be 0.1 N/mm as a standard value of normal concrete. In both tensile models of concrete, the unloading path is assumed to decrease directly to the origin and the reloading path is assumed to behave following to the unloading path.

The envelope stress-strain curve of reinforcing bar is modeled as bi-linear in which the tangential stiffness after yielding is $0.01E_s$, where E_s denotes Young's modulus. After yielding, the stiffness of the reinforcing bar decreases when the stress stage changes from tension to compression, while the similar behavior is observed when the stress stage changes from compression to tension. In the analysis, this phenomenon, so-called Bauschinger effect, is considered by the numerically improved model of reinforcing bars (Fukuura 1997). In addition, in order to evaluate the buckling behavior of longitudinal reinforcing bars, the buckling model proposed by Dhakal (2000) is used. This model is characterized as the spatially averaged material model that accurately takes into account the inelastic behavior of buckling.

3. TARGET STRUCTURES AND LATTICE MODEL

3.1 RC Viaducts

The seismic performance evaluation is performed for two RC rigid-frame viaducts. **Figure 4** shows the dimensions and arrangements in Shimokema R5 Viaduct. Hansui R5 Viaduct has similar dimensions and arrangements as Shimokema R5 Viaduct. They are beam-slab type rigid-frame with three-span. Actual damage conditions of two viaducts after the earthquake are shown in **Figure 5** (Committee 311 2000). It was observed that there was the slight damage with crossed diagonal cracks at the upper column in Shimokema R5 Viaduct. On the other hand, in Hansui R5 Viaduct, the severe shear failure with the collapse of columns and the drops of beams and slabs were observed. The compressive strength of concrete of Shimokema R5 Viaduct was slightly stronger than that of Hansui R5 Viaduct. The material properties used in these two viaducts are summarized in **Table 1**.

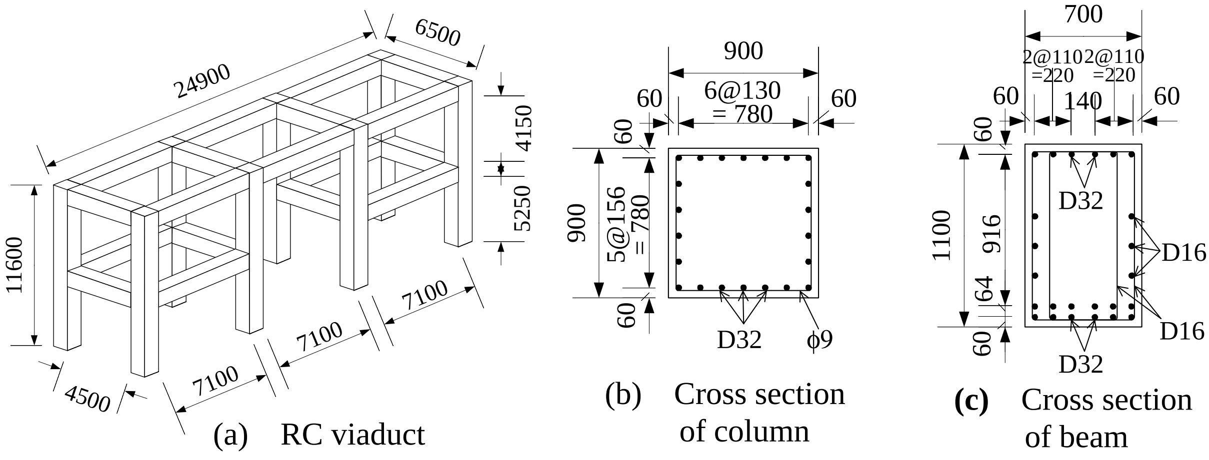
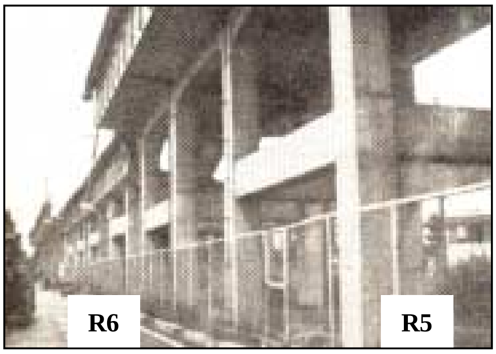
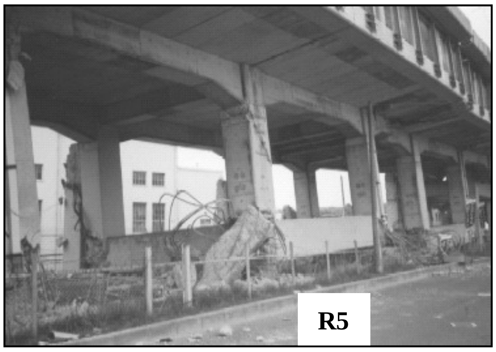


Figure 4 Dimensions and arrangements in Shimokema R5 Viaduct



(a) Shimokema R5 Viaduct



(b) Hansui R5 Viaduct

Figure 5 Damage conditions in Shimokema R5 Viaduct and Hansui R5 Viaduct

Table 1 Material properties of concrete and reinforcement

Name of RC viaducts	Concrete			Longitudinal reinforcement			Transverse reinforcement		
	f_c' (MPa)	f_t (MPa)	E_c (GPa)	f_y (MPa)	f_u (MPa)	E_s (GPa)	f_y (MPa)	f_u (MPa)	E_s (GPa)
Shimokema R5	31.7	2.43	16.8	349	533	197	296	436	204
Hansui R5	29.1	1.27	18.4	322	521	203	263	380	183

The input ground motions determined by Frequency-Dependent Equi-Linearized technique, FDEL (Sugito et al. 1994) are used. The weight of a superstructure including the slab is assumed to be 7200 kN. The self-weight of viaduct is calculated by multiplying the volume of columns and beams by the specific gravities of concrete and reinforcement.

3.2 Analytical Procedure

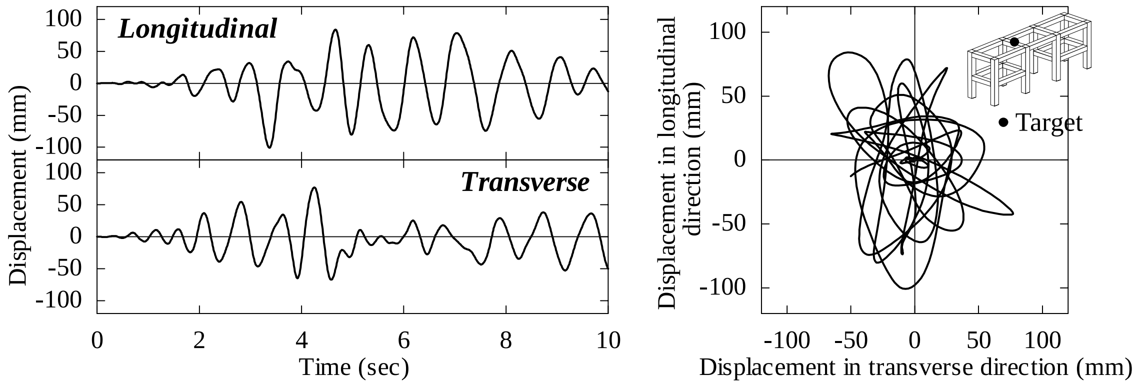
The three-span viaduct is treated as a unit of the analytical model. The model consists of columns and beams, while the slab is not included. In the analysis, it is assumed that the masses corresponding to the self-weight of viaducts are uniformly distributed over all nodal points, using the lumped-mass idealization. It is also assumed that there is a concentrated mass, which is equal to the weight of the superstructure and the slab, acting on the top nodes of columns and beams.

In the dynamic analysis, Newmark's method is used as a time integration scheme. For the damping, the numerical damping of Newmark's method (β = 0.36, γ = 0.70) is used. The Newton-Raphson iteration method is adopted to iterate the calculation until an adequately converged solution is obtained. With the termination of calculation, the out-of-balance force and the energy increment are compared with initial values during iteration. Here, the convergence tolerances for the out-of-balance force and energy are set to be 0.001 and 0.01, respectively.

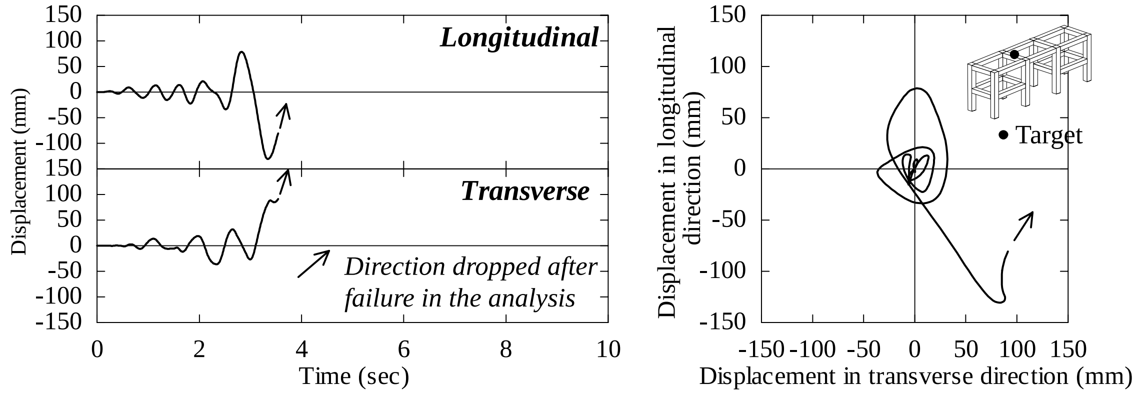
4. DAMAGE ANALYSIS OF ACTUAL RC VIADUCTS

4.1 Seismic Response of RC Viaducts

Figure 6 shows the analytical results by the dynamic 3D lattice model for Shimokema R5 Viaduct and Hansui R5 Viaduct. The analytical results of displacement time histories and particle traces of displacement at the mid-span of an upper beam in the transverse direction are illustrated in the figure. With Shimokema R5 Viaduct, it can be observed that the stable response during 10 seconds is predicted. In the analysis, the buckling of reinforcement takes place at the bottom of the lower column. It can be also observed that the buckling of reinforcement does not occur at the top of the column and the each end of the beam. The cracks occur in the diaconal members of



(a) Shimokema R5 Viaduct



(b) Hansui R5 Viaduct

Figure 6 Analytical responses at an upper beam by the dynamic 3D lattice model

concrete at the upper and lower columns. The behavior corresponds to the actual observation after the earthquake as shown in **Figure 5** (Committee 311 2000). On the other hand, the calculation is terminated due to the divergence in the case of Hansui R5 Viaduct as shown in **Figure 6(b)**. This is caused by the compressive softening of concrete in the diagonal and flexural members at the lower column. Actually, it was observed in Hansui R5 Viaduct that the severe shear failure at the lower columns took place that induced the drops of beams and slabs. It is also found that the direction, in which the beams and slabs dropped down after the lower column fails in shear, can be clearly estimated by the particle traces of displacement in **Figure 6(b)**. As can be judged from the analytical results, Hansui R5 Viaduct fails toward transverse direction during the displacement in longitudinal direction decreases. The information about the state of deformation after the structure fails in shear is beneficial in terms of the restoration after the earthquake.

4.2 Analytical Simulation of Shear Failure of RC Viaducts

To verify the response of Hansui R5 Viaduct where the calculation in the dynamic analysis is terminated by the divergence, the detailed analytical investigation is performed. With Hansui R5 Viaduct, the elements of the lower column in the 3D lattice model are focused on. **Figure 7** shows the stress-strain relationships of concrete flexural members and the strain time histories of concrete diagonal members. In the figure, the deformed shape of 3D lattice model for Hansui R5 Viaduct is also illustrated.

It can be observed that the analysis can predict the compressive softening behavior in diagonal and flexural members of concrete. The compressive softening behavior is observed in the elements D and E in the 3D lattice model, while the tensile strain of concrete increases in the element F. This is because that the externally applied energy by the earthquake is locally

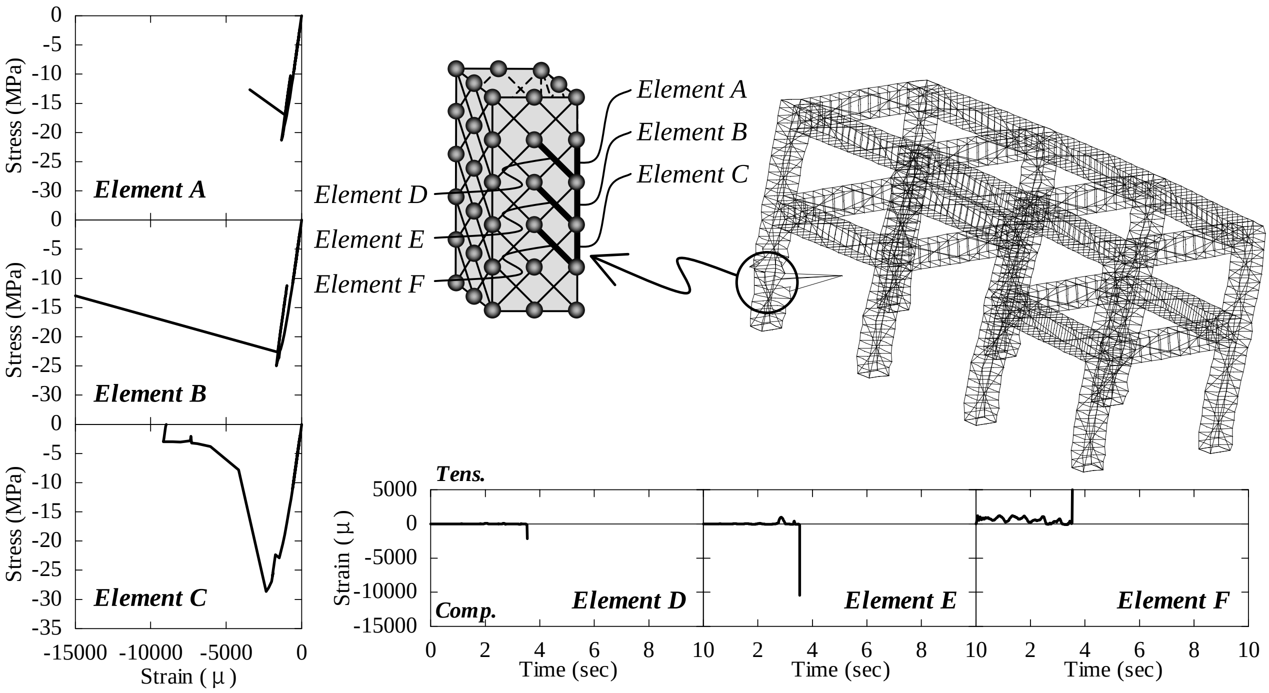


Figure 7 Elemental responses of concrete in 3D dynamic lattice model for Hansui R5 Viaduct

consumed by the elements D and E and the element F in the surrounding portion releases the stored energy into localizing elements. It is assumed that in the analysis the shear failure takes place at the column when the compression softening behavior of concrete in arch and diagonal members has been predicted. The shear failure is predicted in spite of the decrease in shear stresses transferred across diagonal cracks by the aggregate interlock. In addition, arch or diagonal members are assumed to include the severe diagonal cracks that have widely opened. Because of widely opened diagonal crack, the compressive strength of concrete shows the softening as the tensile strain perpendicular to the compressive direction increases. It is also found that the analytical response in the post-peak is milder than that in the actual brittle behavior.

4.3 Influence of Vertical Loads on Seismic Response of RC Viaducts

In order to investigate the influence of the self-weight of structure on the analytical response, two analyses are carried out. **Figure 8** shows the displacement time histories obtained by the dynamic 3D lattice model with and without vertical loads corresponding to the self-weight. It is

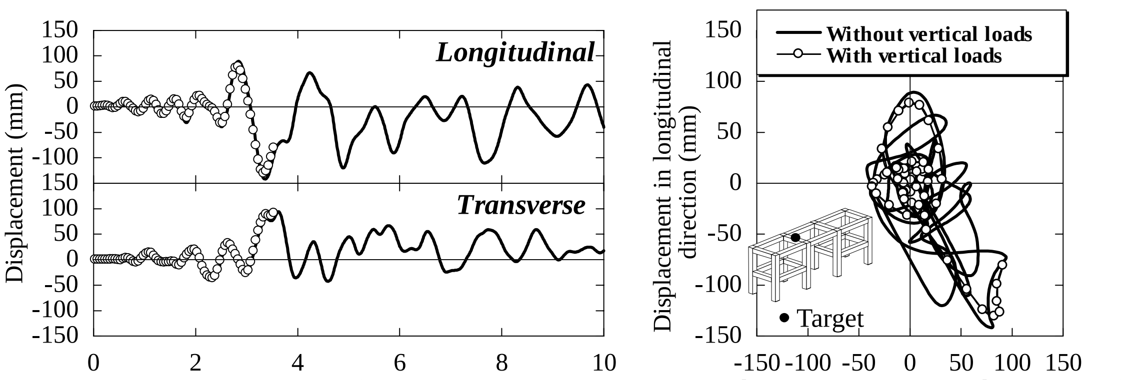


Figure 8 Analytical response of Hansui R5 Viaduct (Influence of vertical loads)

found that when the vertical loads are neglected, the analytical results of RC viaducts show the stable response during 10 seconds. The stable behavior can be observed even if the analysis considering the vertical loads predicts the shear failure. This is cased by the additional moment of $P-\Delta$ effect with the increase in the lateral displacement. It is found that the collapse mechanism takes place due to the geometrical nonlinearity, which is associated with large vertical loads. The analysis of RC viaducts shows that the flexural instability occurs with the vertical loads corresponding to the self-weight and the weight of the superstructure. Therefore, it is recommended that the geometrical nonlinearity due to the material nonlinearity at the large deformation range should be considered in predicting the allowable ductility of RC columns with heavier weight of a superstructure.

5. CONCLUSIONS

3D nonlinear analysis of actual RC rigid-frame viaducts is performed using the dynamic 3D lattice model. The comparisons between the results of dynamic analysis and the actual damages of RC viaducts prove the reliability of the 3D lattice model. It is also found that the analysis can predict damage conditions including both the buckling of longitudinal reinforcement in RC columns and their shear failure. In addition, the dynamic analysis is carried out on the verification with respect to the vertical loading, which represents the self-weight and the weight of a superstructure. The results of the analysis reveal the influence of the vertical loads on the maximum displacement. From the analysis, it is found that when the columns in RC viaducts are subjected to large deformation, the collapse takes place due to the geometrical nonlinearity. The analysis also shows that this instable behavior caused by a large vertical load appears when the lateral displacement significantly increases.

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