0596

# 3-D ANALYSIS OF A RC FRAME-WALL BUILDING DAMAGED IN THE 1995 HYOGOKEN-NANBU EARTHQUAKE

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## SUMMARY

Some of the building structures damaged in the 1995 Hyogoken-Nanbu Earthquake have not yet been thoroughly investigated for the structural irregularities requiring full 3-dimensional analysis. This paper presents a 3-dimensional analysis method to simulate the response of the structure with multiple segments of floor slabs subjected to tri-axial input motions. The method is applied to analyze a reinforced concrete frame-wall residential building that was severe damaged during the earthquake. The building has opening slabs through all floor levels so that it is segmented into four blocks with uneven stiffness ratio. Two structural models are used in the analysis for comparison: (1) single rigid diaphragm connecting all nodes in floor level and (2) multiple rigid diaphragms for the segmented blocks. The analysis results of the second model show distinct responses among the segmented blocks and simulate well the observed damage.

## INTRODUCTION

Many building structures damaged in the 1995 Hyogoken-Nanbu Earthquake have been analyzed by numerical simulation to investigate the damage mechanism (Tasai and Kitada, 1998, Teshigawara, et al., 1998, and Hayashi, et al., 1999) using relatively simple structural models. Some severe-damaged building structures in the area during the earthquake may still have not been thoroughly investigated for the structural irregularities that require full 3-dimensional analysis. For example, a building structure may have multiple segments of floor slab in a floor level. Assuming a whole floor slab as rigid diaphragm is reasonable for the building structure, especially for frame-wall structure, if the floor slab is well integrated. However, it may result in less reliability if applying the assumption to a structure with multiple segments of floor slab.

This paper presents the investigation on a RC frame-wall building that has floor slab in multiple segments. The five-story high residential building was constructed and occupied just before the earthquake. It has a long rectangle floor plan with four segments of floor slab. That is, the building is in four blocks connected through beam elements. According to the damage observation, it was thought that significant torsional response might have been induced because of the damage concentrated in the outside frames. However, the building was well proportionally designed and the structural eccentricity was not considerable. Therefor, the analysis shall be carried out using sophisticated element model and structural model to investigate the damage mechanism. In this paper, multi-spring model (MS model, Li and Otani, 1993) and fiber model (Li and Kubo, 1998 and 1999) are used to idealize the column and shear wall to allow for the interactions among biaxial bending and axial loads. And the segmented floor slabs are treated and represented by multiple rigid diaphragms. The input motions are evaluated making use of nearby seismometer records. Thus the analysis is performed to simulate the building response in accuracy and to predict the damage in a reasonable agreement with the real damage.

## THE BUILDING AND DAGAME DATA

The building, 5-story high, long rectangle floor plan, 2 by 11-span or 10.5 by 81 square meters, was constructed in Higashinada ward, Kobe-city in 1994 and damaged in the 1995 Hyogoken-Nanbu earthquake. Figure 1 shows the typical floor plan and member sections, and Figure 2 illustrates the frame elevation. The longitudinal frame

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directs near East-West and transverse frame in about North-South. The first story is open frame (piloti) for residential parking except the shear wall placed in the center stair and elevator room (frame X6 and X7). The upper stories are residential units with boundary shear walls in transverse direction. Note that the wall in longitudinal direction is secondary element not considered in design to carry lateral loads. From the second floor up to the roof floor level, openings in the floor slab makes it four slab segments, in frame X1 to X2, X3 to X6, X7 to X10 and X11 to X12. The outside segments (X1-X2 and X11-X12) are relatively weak in transverse direction for no shear walls in that direction. The material properties of the structure are listed in Table 1.

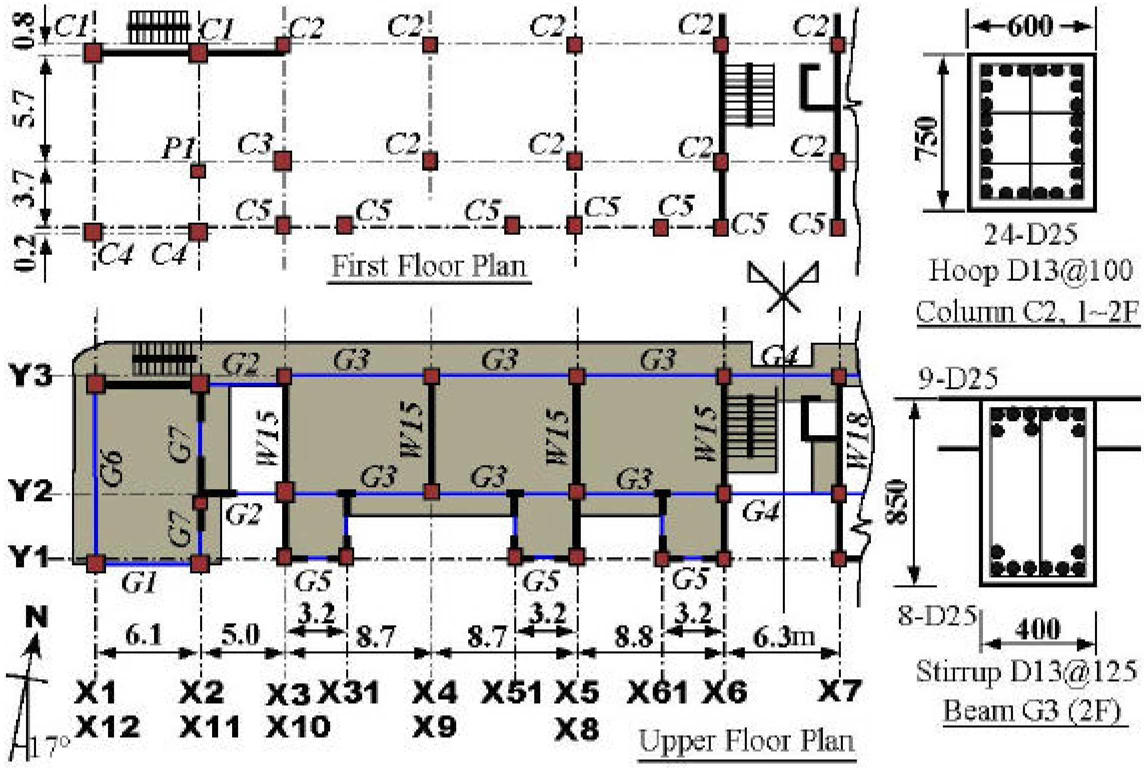



**Figure 1: Typical floor plan and member sections of the analyzed building structure**

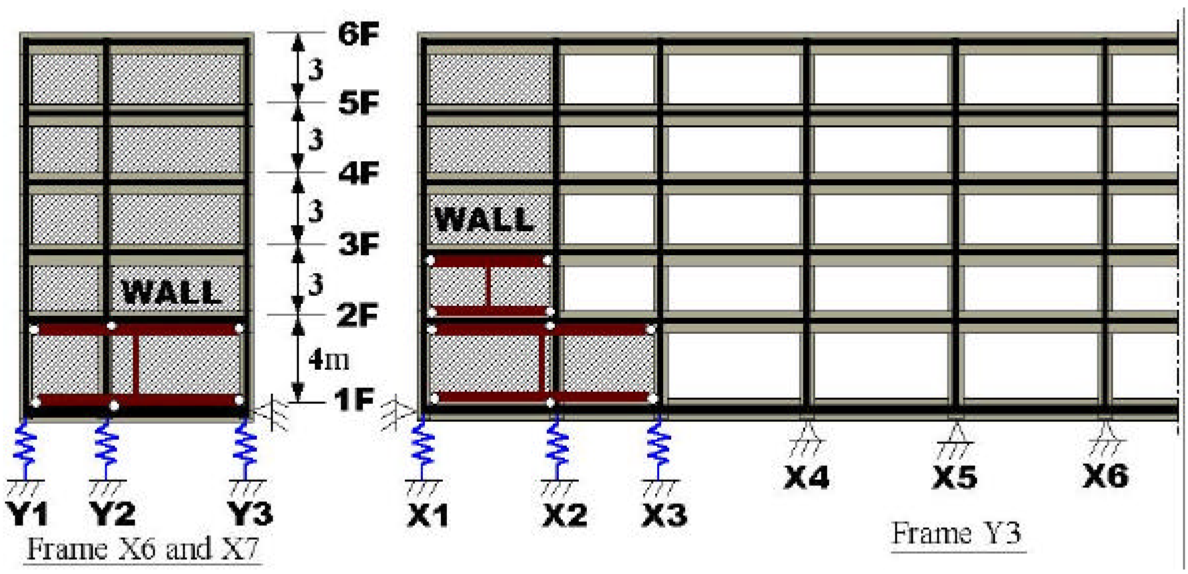
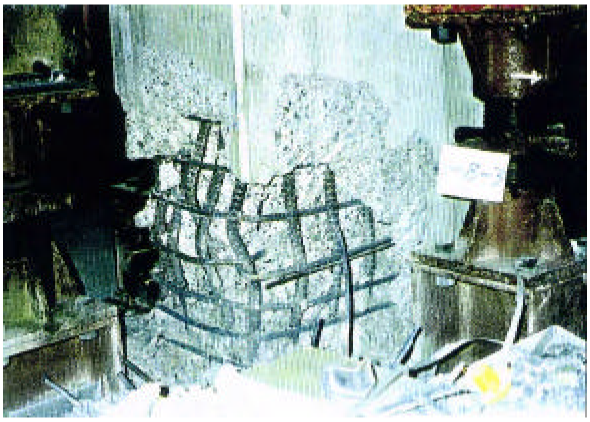



**Figure 2: Frame elevation and frame models for the analysis**

The earthquake damage to the lateral load carrying system was severe that made the structure nearly collapse. According to the second author's observation, damage was mainly found in the columns in the first story. Concrete crushed and steel bar exposed and buckled at both column base and top, as shown Figure 3, in almost columns in the West-side from frame X1 to X3, as well as some East-side columns (frame X10~X12). The column of middle blocks (frame X3~X10) were damaged too but less than the side columns. X-shape shear cracks were found in the shear walls in frame X6 and X7 as well as all the secondary walls. Flexural and shear damage of columns and beams were also found in upper stories except top story where the damage was slight.

**Table 1: Material properties for determining the element properties used in the analysis (unit: N, mm)**

| Concrete materials | Column | Beam | Shear wall | Secondary wall |
|---|---|---|---|---|
| Young modulus | 24500 | 24500 | 22900 | 11450 |
| Compression strength and strain | 23.5, 0.002 | 23.5, 0.002 | 20.6, 0.002 | 10.3, 0.002 |
| Shell ultimate strength and strain | 0, 0.012 | 0, 0.01 | 0, 0.01 | 0, 0.01 |
| Core ultimate strength and strain | 23.5, 0.02 | 0, 0.01 | 0, 0.01 | 0, 0.01 |
| Tension strength | 2.0 | 2.0 | 2.0 | 0.0 |
| Shear strength | 1.13 | 1.13 | 1.03 | 0.52 |
| Steel materials | Es | $\sigma_y$ | $\sigma_{max}$ | Ultimate strain |
| D19 and above D19 | 205800 | 441 | 551 | 0.2 |
| D16 and below D16 | 205800 | 382 | 477 | 0.18 |



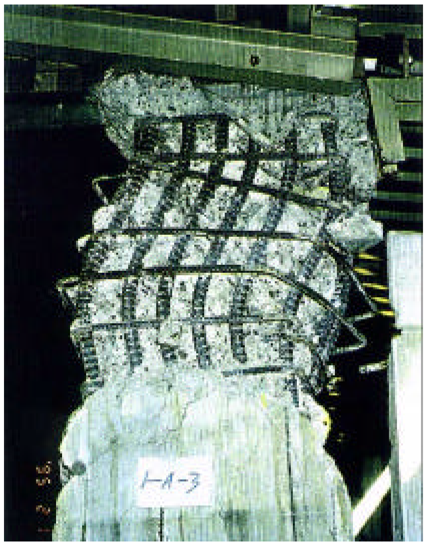



**Figure 3: Sample of the damage of 1st-story columns (left: column base, right: column top)**

## COMPUTER PROGRAM

A computer program CANNY is developed for 3-dimensional analysis of frame and frame-wall structures by the first author. Both static and dynamic loads are treated and multiple analysis functions (mode analysis, pushover, static cyclic analysis and dynamic response) are made available. MS model and fiber model are used in the program to idealize flexural elements to allow for multi-axial load interactions. Numerical method is proposed and applied to maintain the axial force equilibrium in the elements idealized by the MS model and fiber model (Li and Kubo, 1998). The program also includes uniaxial model for one-component elements. The uniaxial model represents the relations of moment-rotation, moment-curvature, shear force-deformation, and axial force-deformation.

For the analysis of various types of structures, the program also includes rigid element (rigid link, rigid diaphragm and rigid body) and spring element to represent boundary conditions and constraints among structural nodes. It is made available using rigid diaphragm to simulate the rigid-body movement of multi-segment floor slabs in the horizontal plane of floor level.

## MODELING OF THE STRUCTURE AND ELEMENT

The frame model for the analysis is illustrated in Figure 2. The beam, column and shear-wall are idealized as line element with proper rigid zone at element-end to represent the beam-column joint.

The beam is idealized using uniaxial model (one-component line element) to represent moment-curvature relation in frame plane. The beam-end rotation then is calculated from curvature distribution along beam axial line that is based on assumption of linear-distributed flexibility. Elastic shear deformation of beams is included. The column is idealized by MS model to simulate the interactions among bi-directional bending and axial loads. The number of steel springs and concrete springs for the column in MS model is so determined that one steel spring represents a single steel bar, and the governing area of each concrete spring is made about 75 by 75 square mm. The hoop bar confinement to column section core concrete is considered (Park and Paulay, 1975). The column shear deformation in nonlinear behavior is also taken into account and is represented by two uniaxial shear springs in the column two bending directions. The shear wall is idealized as uniaxial bending column in the panel plane. The wall base and top sections are kept in plane section and the shear-wall element is made to resist the vertical and horizontal displacements of the element-relevant corner nodes and middle-span nodes. This is ensured by placing rigid beam on shear-wall base and top sections and using pin-connection to the relevant structural nodes, as shown in Figure 2. The flexural and axial deformation of panel and the interaction between them are simulated by fiber model that gives the relations of axial force-deformation and moment-curvature at the panel base and top sections. The rotation of the top and base rigid beams is calculated from the curvature and based on the assumption of linear distribution of flexibility along the shear-wall axial line. The number of steel and concrete fibers is determined in similar way of column. Uniaxial shear spring is added to simulate the shear deformation of shear-wall in nonlinear behavior.

The bending capacity of beam is calculated using the material properties listed in Table 1 and based on plane section assumption and taken into account of slab contribution (maximum 1.2-meter slab in one side). The shear capacity of column and shear-wall is evaluated in AIJ recommended equations. The axial stress effect on the shear capacity is ignored.

The floor slabs are assumed having rigid body movement in the floor plane. To investigate the effect of multi-segment floor slabs on the structural responses, two models treating floor slab are considered. First model simply assumes the whole floor area in one rigid diaphragm. Second model treats each segment of the floor slab as a rigid diaphragm, so it makes four rigid diaphragms in a floor level, and each has three displacement degrees of freedom in the floor plane, as shown in Figure 4. Then the beams that connect between the rigid diaphragms are idealized as two uniaxial-bending elements in horizontal and vertical planes and with axial stiffness.

The structural nodes are assumed to have five displacement degrees of freedom, independent vertical translation and rotations in two vertical planes (X-Z and Y-Z plane) and two lateral translational displacements (in X and Y directions) determined by the movement of rigid diaphragm. The torsional rotation (in horizontal plane) at structural node is made free because the torsional stiffness of elements is ignored.

The foundation of the building is continuous footing in two meters deep below ground floor level. Elastic footing beam is included in the frame model, and pin support is placed under the beam and to the first-story column base (Figure 2). Vertical support spring is used under the first-story shear wall and corner column to represent the uplifting of shear wall and corner column under tension. The initial compressions of the support springs are counted the structure self-weight and the footing weight.

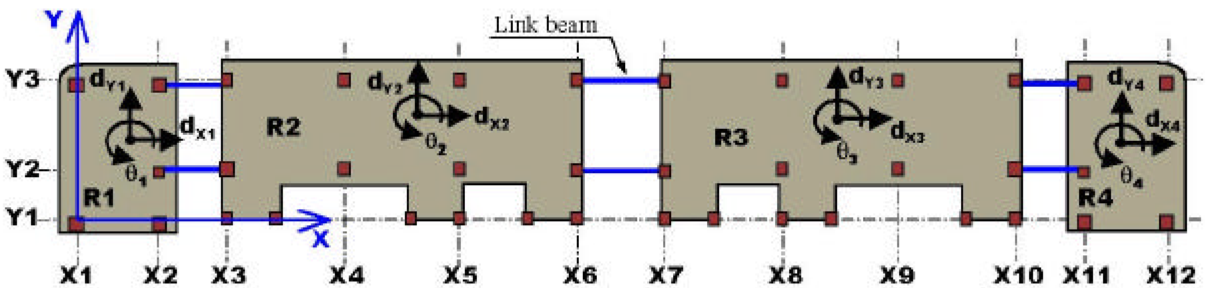



**Figure 4: Multiple rigid diaphragms to represent the effect of segmented floor slabs**

## ANALYSIS METHOD

The response analysis is carried out to the input ground motions in two lateral directions as well as in vertical direction at the ground floor level. The input accelerations are shown in figure 5, that are evaluated from the nearby records (about 350 meters between the record location and the building) and are transformed to match the building longitudinal and transverse directions. The input time duration is 25 seconds (start at 5 sec and end at 30 second of the records).

The structural self-weight plus 300 N/m$^2$ live load are counted to form mass matrix, and the mass is concentrated yielding vertical inertia load at structural nodes and inducing lateral load including rotational inertia at rigid diaphragm gravity center-point. Rotational inertia load at structural nodes is ignored. The gravity center-point (X, Y) of rigid diaphragms are listed in Table 2 that are measured to the origin at structural joint (X1, Y1) in ground floor level and are found based on the mass concentrated at structural nodes. The gravity center-point is almost incident with the geometrical center-point of each floor level and of each slab segment for the building is in good proportion.

**Table 2: Weight and gravity center-point of floor level and slab segment (unit: kN, m)**

| Floor level | Total weight | Whole floor area | | R1 (X1~X2) | | R2 (X3~X6) | | R3 (X7~X10) | | R4 (X11~X12) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | X | Y | X | Y | X | Y | X | Y | X | Y |
| 6F | 9369 | 40.45 | 6.172 | 3.604 | 4.949 | 24.73 | 6.664 | 56.07 | 6.683 | 77.30 | 4.949 |
| 5F | 8960 | 40.44 | 5.964 | 3.567 | 5.464 | 25.05 | 6.129 | 55.81 | 6.161 | 77.33 | 5.464 |
| 4F | 8722 | 40.43 | 5.973 | 3.595 | 5.537 | 24.90 | 6.130 | 56.12 | 6.161 | 77.30 | 5.537 |
| 3F | 8979 | 40.43 | 5.960 | 3.574 | 5.525 | 25.01 | 6.119 | 56.01 | 6.149 | 77.33 | 5.525 |
| 2F | 9254 | 40.44 | 6.120 | 3.542 | 5.657 | 25.10 | 6.290 | 55.74 | 6.324 | 77.36 | 5.657 |

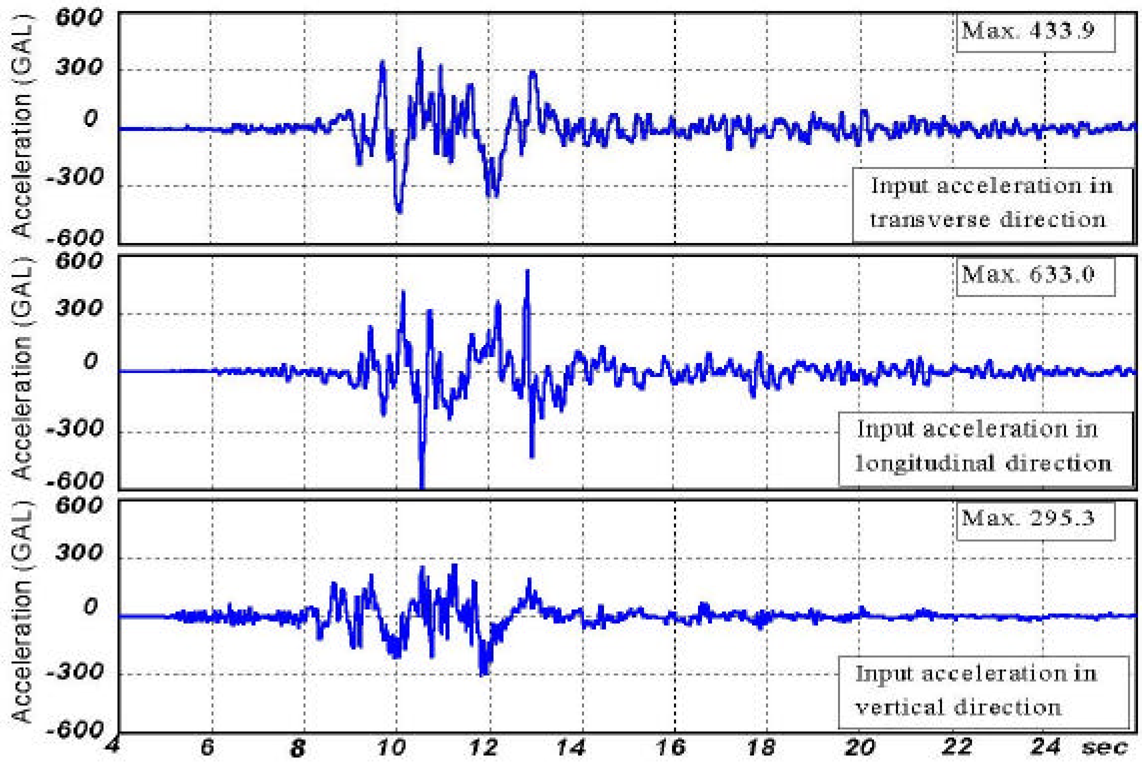



**Figure 5: Input accelerations evaluated from nearby records during the earthquake**

The step-by-step numerical integration is carried out at time interval 1/200 sec and the equations of motion are solved by Newmark $\beta$-method ($\beta = 0.25$). Internal viscous damping (damping constant 5 %) proportional to instantaneous stiffness matrix is assumed. The damping is calculated based on the frequency of fundamental mode. The fundamental period is estimated as 0.3114 sec for the structural model with whole floor rigid diaphragm and 0.3174 sec for the structural model with multiple rigid diaphragms. Therefore, the damping matrix is calculated as [C] = 0.004975[K] and [C]=0.005052[K] for the structural models, respectively.

The element initial forces are taken into account by a first-step static load analysis to the structural self-weight and live load, then followed by step-by-step dynamic response analysis. The step-by-step analysis checks the force equilibrium at the end of every step. Iteration may be carried out to maintain the equilibrium if any unbalanced force components over 0.5 % to the structural total weight (about 230 kN) or if any unbalanced moment components over 0.5% of the structural weight times element average dimension (about 970 kN·m).

## COMPARISON AND DISCUSSION ON THE ANALYSIS RESULTS

The structural vibration mode before the earthquake is estimated using the structural models. The two structural models produce quite different vibration modes. The fundamental period is almost identical, while the period of higher modes is not. The third mode period is 0.244 sec and 0.281 sec, and fourth mode 0.099 sec and 0.183 sec for the two structural models, respectively. That is because the second structural model with multiple rigid diaphragms has relatively lower stiffness and has different stiffness ratio in X- and Y-direction among the segmented blocks. This implicates different responses between the two structural models.

Looking at the time history of displacement responses at the gravity center-point of top floor level rigid diaphragms, the results of the second structural model shows completely distinct responses in transverse direction but almost similar in longitudinal direction among the segmented blocks, as shown in Figure 6. This can be seen clearly by the illustration of Figure 7 for the peak responses at 10.25 sec. The different responses among the blocks in transverse direction and between the transverse and longitudinal directions are attributed to the connection beams among the segmented floor slabs. In transverse direction, the beam to hold two adjacent blocks is subjected to bending and shear that is relatively weak, while in longitudinal direction the beam is relatively stiff subjected to axial tension and compression. As aforementioned the structure is in good proportion so that the torsional response is not considerable. The distinct responses among the segmented blocks are the attribution to the damage concentrated in the ends of the building.

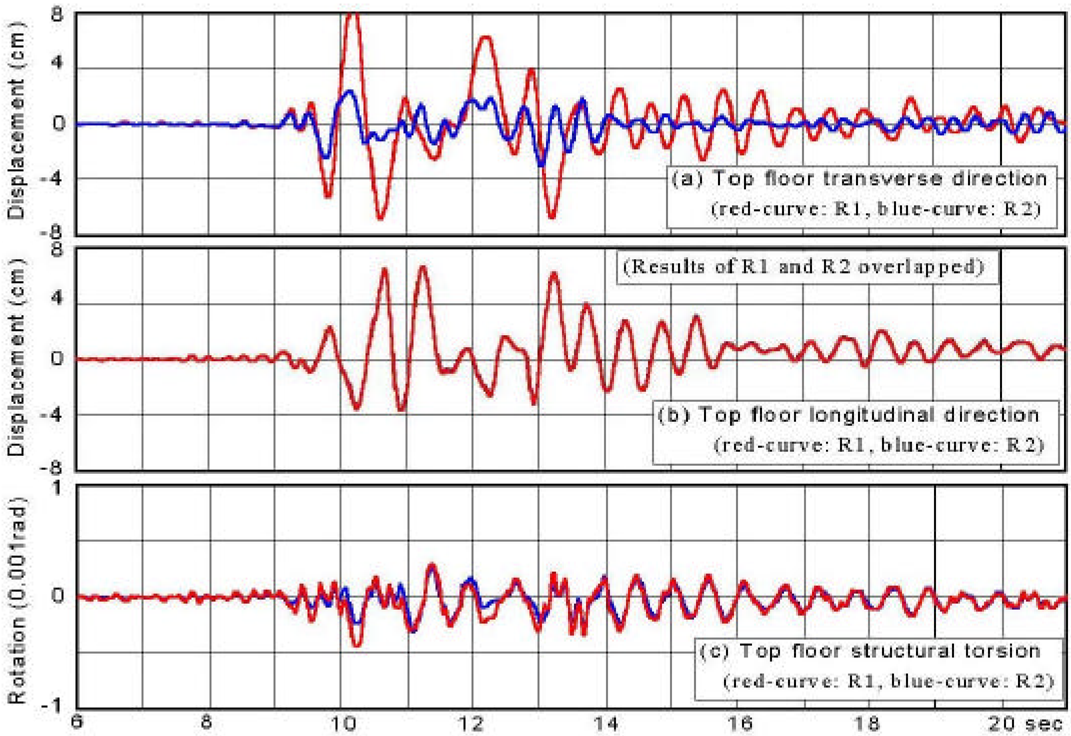



**Figure 6: Displacement responses at top floor level of the multi-rigid diaphragm structural model**

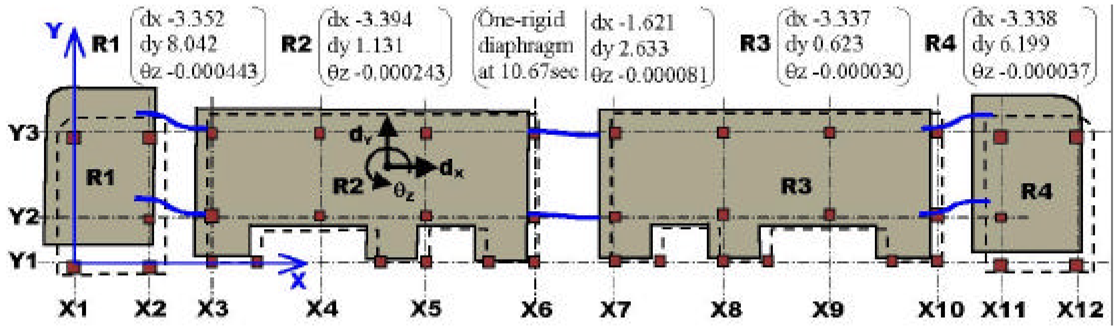



**Figure 7: Top floor displacement at peak responses (at 10.25 sec)**

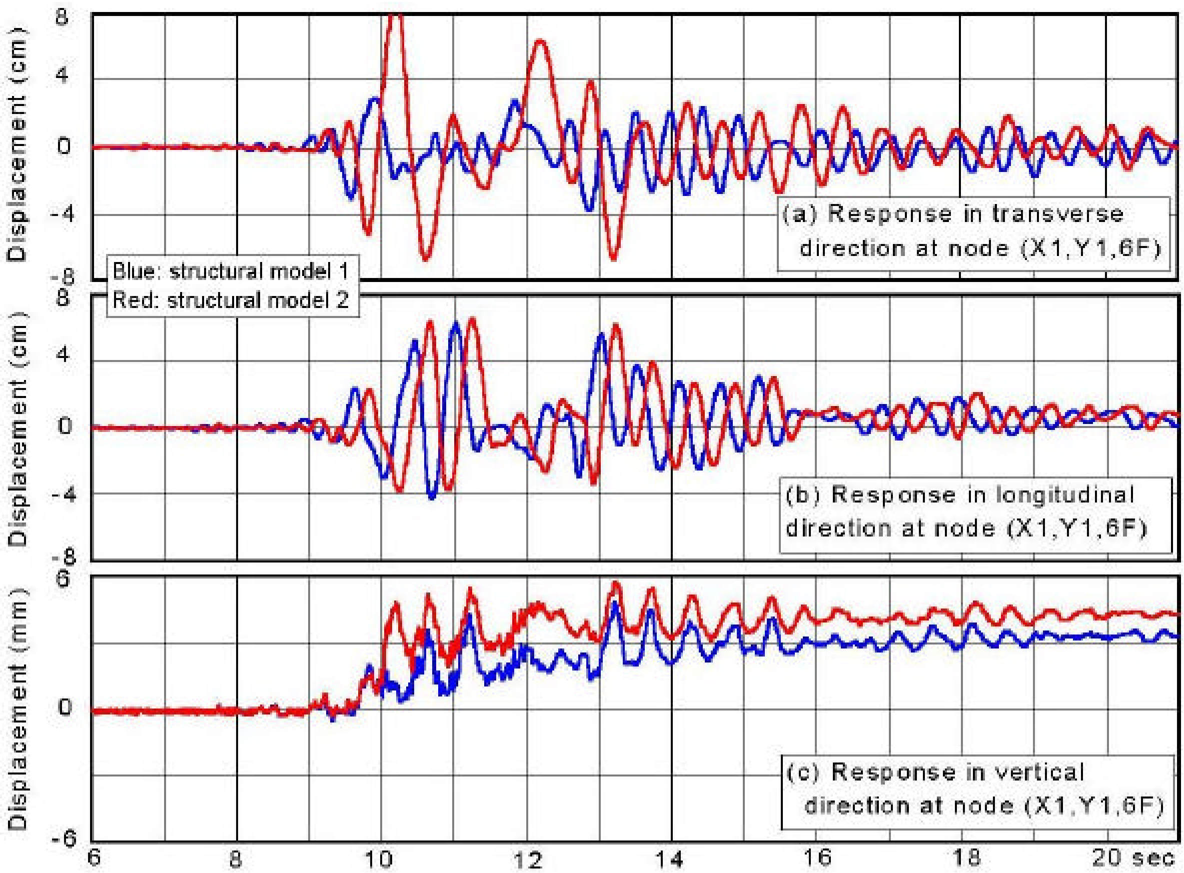



**Figure 8: Comparing the responses at structural node predicted by the two structural models**

Comparing the responses predicted by the two structural models, Figure 8 shows the responses at a corner node (X1,Y1, 6F) in top floor level. The blue curve is of whole floor in co-rigid movement (structural model 1) and the red curve is of multiple rigid diaphragms (structural model 2). The nodal lateral displacements are determined by the relevant rigid diaphragm movement including the contribution of structural torsion, while the vertical displacement is the response to the vertical input ground motion. The nodal vertical displacement of positive value is in tension and negative in compression. It indicates shift to tension side because of the interaction among the bending and axial deformation in vertical column elements. The results show significant differences both in amplitude and in frequency between the two structural models.

Finally looking at element damage as shown in Figure 9. The damage distribution is predicted in the structural model with multi-slab segments. All columns in the first story are tension yielded and also shear walls shear failed. To the outside blocks (frame X1~X2 and X11~X12) the columns are predicted undergone steel-bar compression yielding. That may implicate steel bar buckling and concrete crushing. Also a number of columns, beams and shear walls in upper stories are predicted undergone cracks and yielding. Generally the damage predicted by the analysis in the structural model with multiple rigid diaphragms agrees with the observation from site. Especially, it can explain the over-concentrated damage in the building-end blocks.

The structural mode with whole floor rigid diaphragm also predicts tension yielding and shear failure but almost no compression yielding. Meanwhile, the damage predicted by the structural model is relatively even distributed. Figure 9 compares the material ductility or the spring deformation factor related to the yielding displacement for some corner columns and middle shear walls. The number in parentheses is of the structural model 1. It results in less damage in the outside blocks but more damage in the middle blocks because of whole floor area is treated as a rigid diaphragm.

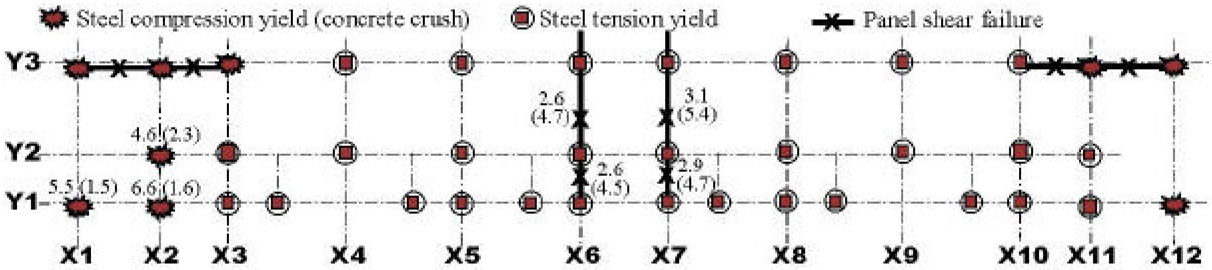


**Figure 9: Predicted damage distributions in structural model with multi-rigid diaphragms**

## CONCLUSIONS

A RC frame-wall building damaged in the 1995 Hyogoken-Nanbu earthquake with long floor span and segmented floor slabs is analyzed using 3-dimensional model and treating the floor slab in multiple rigid diaphragms. The calculated results show distinct responses among the floor segments for different stiffness ratios and vibration modes.

The calculated damage distribution agrees well with the observed damage from site. The damage over-concentrated in the building out-side blocks is attributed to the different responses among segmented floor blocks. The building is in good proportion so that the structural torsion has less effect on the responses and is not considered the cause of the over-concentrated damage.

Using simple structural model by assuming whole floor level in co-rigid movement may result in less reliable prediction in the case when floor slab is not well integrated.

## REFERENCES


Hayashi, Y., Tamura, K., et al. (1999), "Simulation analyses of buildings damaged in the 1995 Kobe, Japan, Earthquake, considering soil-structure interaction," *Earthquake Engineering and Structural Dynamics*, Vol.28, No.4, pp371-391.

Li, K.N. and Otani, S. (1993), "Multi-spring model for 3-dimensional analysis of RC members," *International Journal of Structural Engineering and Mechanics*, Vol.1, No.1, pp17-30.

Li, K.N. and Kubo, T. (1998), "Reviewing the multi-spring model and fiber model," *Proceedings of the 10th Japan Earthquake Engineering Symposium*, Vol.2, pp2369-74.

Li, K.N. and Kubo, T. (1999), "Analysis of circular RC member in MS/fiber model," *Summaries of Technical Papers of Annual Meeting* (Hiroshima), St.II, AIJ.

Park, R. and Paulay T. (1975), *Reinforced Concrete Structures*, John Wiley & Sons, New York.

Tasai, A. and Kitada T. (1998) "Analysis of damaged reinforced concrete building failed in flexure of columns by 1995 Hyogoken-Nanbu earthquake, (in Japanese)" *Proceedings of the 10th Japan Earthquake Engineering Symposium*, Vol.1, pp345-348.

Teshigawara, M., Kitagawa, Y., et al. (1998) "Estimation of strong ground and damage of building in 1995 Hyogoken-Nanbu earthquake, (in Japanese)" *Proceedings of the 10th Japan Earthquake Engineering Symposium*, Vol.1, pp315-320.