Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 816

COLLAPSE SIMULATION OF REINFORCED CONCRETE BUILDINGS WITH ASFI APPROACH

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ABSTRACT

We have carried out full-scale shaking tests on three-story reinforced concrete school buildings, one RC frame and the other strengthened, at E-Defense in 2006. A dynamic progressive collapse occurred associated with shear and axial failure of short columns in the RC frame specimen with fixed bases under an extreme level of earthquake motion. A method of simulating the collapse behavior of the specimen was developed with columns members, in which shear deteriorating behavior after flexural yielding was incorporated. The axial and lateral hysteretic responses of the inner column were measured at the column base in the dynamic test, which was compared with the analytical result to verify the analytical model. Nonlinear earthquake response analysis was carried out with three-dimensional frame model, taking account of shear deterioration of the columns. The effects of shear deterioration in the columns as well as the intensity and duration of the earthquake motion were discussed based on various cases of collapse simulations.

Introduction

It is necessary for collapse simulation of existing reinforced structures in Japan to estimate the ultimate deformation capacity of brittle short columns. However, The accuracy of the estimation is not adequate, because the deformation capacity depends on confinement, shear span ratio, axial stress, and material property. Japanese seismic evaluation standard for existing structures roughly estimates the deformation capacity of columns based on the ratio of shear strength to flexural strength, which is empirical and conservative lower bound of the test results.

We have carried out the full-scale shaking tests on two three-story reinforced concrete buildings with flexible or fixed foundation from September to November 2006 at E-Defense, the world largest three-dimensional earthquake simulator, as a part of a five-year national project on seismic safety of urban areas, referred to as DaiDaiToku project (Toshimi Kabeyasawa, 2006 and Toshikazu Kabeyasawa, 2006). One of the purposes in the shaking test on the first RC

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specimen is to simulate progressive collapse mechanism for Japanese existing school building structures. A progressive collapse mechanism of the first story occurred in the specimen starting from the brittle failure of the short columns in case of the fixed bases. The shear force and axial force were measured in the test at the bottom of the long internal column, which was next to the brittle columns and affected with the redistribution of the axial forces from these columns.

In this paper, the collapse of the RC specimen is simulated and the time history response of the internal column is compared with the test. The ultimate deformation capacity of the internal column is simulated with the estimation formula based on the AIJ guideline and Axial Shear Flexural Interaction (ASFI) Model (Hossein Mostafaei, 2006). The three- dimensional nonlinear frame analysis of the specimen is carried out including the shear failure, where the ASFI model was modified so that the model can be incorporated into the frame analysis.

Full-scale Specimens and Shake Table Tests

The shaking test specimens were three-story reinforced concrete structures modeling existing reinforced concrete school buildings. The specimens were designed following 1970' Building Code of Japan. The structural floor plans of the reinforced concrete specimens are shown in Fig.1. The structure has three spans in the longitudinal (Y) direction, two spans in the orthogonal (X) directions. The span length is 4m in Y-direction, 2m and 6m in X-direction. Note that total number of columns is 11 and a column at X2Y3 point is missing from regular location. Structural walls are located in 6m spans of the outer two frames in X-direction. The inter-story height is 2.5m each for the 1st to 3rd story, and the height of all girders is 0.5m in the longitudinal direction. Both outer frames in the longitudinal direction have spandrel walls or standing walls, which are also typical in Japanese school buildings.

The spandrel wall heights in these frames are different as 1.2m in X1-frame, as is typical on the north side, and 0.8m in X3-frame as on the south side. The section sizes and the reinforcement details are also inscribed in Fig.1. The standard columns section is 400(mm) square with 8-D19 (p_g=1.9%) except for Y3 columns (400(mm) square with 10-D22) and brittle shear columns in X1 frame (300×400(mm) with 8-D19). The column hoops are D10 for 100(mm) based on the minimum requirement of spacing, although the ratio (0.35%) is relatively high because of the scaled column section by five-sixth. The concrete strength of the first story was estimated from the cylinder test as 31Mpa. The strength of longitudinal bars was 384MPa, and that of hoops was 392MPa. Principal response and collapse direction of the structure is designed in longitudinal direction, because the two structural walls resist strong earthquake motion in transverse direction. The lateral and axial load redistribute after brittle shear failure of short columns in X1 and X3 frame. The load cell is implemented at the bottom of the X2Y2 column, where redistribution forces affected evidently to the response of the element.

The shake table tests were carried out six times by increasing the amplitude of the recorded motion (1995 JMA Kobe) up to collapse. Lateral strengths of the specimen deteriorate at the final shaking test with fixed base and under JMA KOBE 100% level. The all columns in the first floor suffered bending or shear failure, and a near collapse in the first story was observed. The responses in the final shaking test are analyzed in the following study.

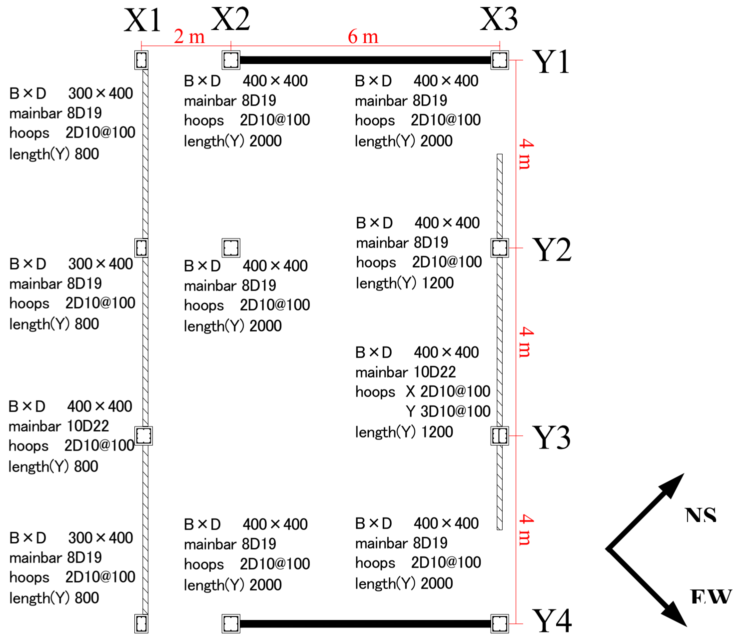

Figure 1. The typical floor plan of the 3rd-story full-scale test specimen

ASFI Model for Frame Analysis

In following analytical studies of the shaking test, a RC frame specimen was idealized with the two different type of frame model: Conventional fiber model and ASFI Model. Axial Shear Flexural Interaction (ASFI) model has been proposed for simulation of reinforced concrete columns under monotonic static loading by Hossein Mostafaei (Mostafaei, 2006). The model can evaluate the ultimate deformation capacity of the columns, even when the shear failure occurs after flexural yielding at the end of the column. The moment -curvature relation at the top and bottom section is evaluated with the stress-strain relation in each fiber element (fiber model), and the shear stress τ_s -strain γ_s relation of the column is represented on the inflection point, where the stress in principal direction is balanced with axial and shear stress (axial shear model).

In this study, the original ASFI model is modified and adapted to nonlinear frame analysis. The proposed model has been improved from the original ASFI model in the following points as:

(1) The varying axial force is applied on the column in the frame analysis
(2) The response in the opposite direction affects the strength deterioration
(3) The height of the inflection point is varied in the time history response

The model requires different numerical procedures from the original such as:

(1) The moment and shear are converged to be consistent based on the constitutive matrixes.
(2) The tangent stiffness method is applied to the incremental analysis in the frame model
(3) Modified Kent and Park model is used for the concrete material model in the fiber element.

The procedure of the modified ASFI model for the frame analysis is briefly described as follows with the formulas (Eq.1- Eq.6). (1) Firstly, the moments and balanced axial force is evaluated from the increment of the element force in the fiber model. (2) In the axial shear model, the average axial stiffness is derived from the difference between centroid strains at the inflection point and the end fiber section. (3) Pseud normal strains ε_x ε_y and shear strain γ_s are calculated from the axial shear flexibility matrix and balanced force in the fiber model. (4) The axial shear stiffness matrix is reconstructed from the stress-strain relation in the principal direction. (5) Shear stiffness (shear stress and strain) in the single shear spring is evaluated, which is balanced with the external force. (6) The flexural and shear stress are converged by the concrete softening factor β and average centoroid stiffness in axial shear model.

The minimum concrete softening factor β of compression strength is 0.3 for the maximum effect of the shear crack on the strength in this model, and the secant stiffness of shear spring is supposed to be constant value after the maximum shear crack has been occurred in orthogonal direction as well as in original ASFI model. The constant shear stiffness is quite smaller than flexural stiffness in fiber section. In this study, pull out effect and aggregate interlock mechanism are ignored for simplified approach. The ASFI model does not simulate the actual deterioration slope after crushing of cover concrete, because the buckling of the compressive rebar is not considered. The model can accurately simulate backbone curves until the ultimate deformation and restoring forces of columns are released with shear failure.

$$\textit{Fiber model (curveture } \Delta\phi_{xy1} \textit{ axial centroidal strain } \varepsilon_{01} \textit{ in end fiber section)}$$
$$\textit{fiber hysteresis model } (f_{c\max} = \beta f_{c0}\ (f_{c0} : \textit{concrete strength in cylinder test}))$$
$$\begin{pmatrix} \Delta\phi_{x1} \\ \Delta\phi_{x2} \\ \Delta\varepsilon_{01} \end{pmatrix} = \begin{pmatrix} \sum EAX^2 & \sum EAXY & \sum EAX \\ \sum EAXY & \sum EAY^2 & \sum EAY \\ \sum EAX & \sum EAY & \sum EA \end{pmatrix} \begin{pmatrix} \Delta m_{x1} \\ \Delta m_{y1} \\ \Delta N \end{pmatrix}, \quad \textit{Flexural Stiffness } K_f = (\Delta m_{x1} / \int \Delta\phi_{x1} dx) \tag{1}$$
$$m_x, m_y : \textit{Moment},\ N : \textit{axial force},\ E : \textit{modulus},\ A : \textit{fiber area},\ X, Y : \textit{fiber position}$$

$$\textit{axial stiffness } f_{xf} = (0.5\varepsilon_{01} + 0.5\varepsilon_{02} - \varepsilon_c)/\Delta\sigma_0, \quad \sigma_0 = f_{c0}\left[2(\varepsilon_c/\varepsilon_{c0}) - (\varepsilon_c/\varepsilon_{c0})^2\right] + \rho_x E_{sx}\varepsilon_{c0} \tag{2}$$

$$\textit{Axialshear model (in plain strain } \varepsilon_x\ \varepsilon_y, \textit{shear strain } \gamma_s \textit{ on inflection point)}$$
$$\begin{pmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma_s \end{pmatrix} = \left(\left(D_\theta^{\,t} \begin{bmatrix} E_{cp1} & & \\ & E_{cp2} & \\ & & G_{cp} \end{bmatrix} D_\theta + \begin{bmatrix} E_{sx}\rho_x & & \\ & E_{sy}\rho_y & \\ & & 0 \end{bmatrix} \right)^{-1} + \begin{bmatrix} f_{xf} & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \right) \begin{pmatrix} \Delta\sigma_0 \\ 0 \\ \Delta\tau_m \end{pmatrix} \tag{3}$$
$$D_\theta : \textit{principal direction trans matrix},\ E_{cp12} : \textit{concrete stiffness},\ G_{cp} : \textit{concrete shear},\ E_{sxy} : \textit{steel stiffness}$$
$$\rho_{xy} : \textit{reinforcement ratio}, \sigma_0 : \textit{axial stress}, \tau_m : \textit{flexural strss in fiber model}$$

$$\textit{principal stress} \quad \varepsilon_{p1}, \varepsilon_{p2} = (\varepsilon_x + \varepsilon_y)/2 \pm 0.5\sqrt{(\varepsilon_x - \varepsilon_y)^2 + \gamma_s^{\,2}} \tag{4}$$
$$\textit{softening factor} \quad \beta = 1/(0.8 - 0.34(\varepsilon_{p2}/\varepsilon_{c0}))$$

$$\textit{axail shear hysteresis model} \rightarrow \textit{reconstruct axail shear stiffneness matrix}$$
$$\textit{Shear Stiffness } K_s = (\Delta\tau_{new} / \Delta\gamma_s)) \tag{5}$$
$$\textit{if } \Delta\tau_{new} \neq \Delta\tau_m, \quad \beta, f_{xf} \textit{ changes in next covergence}$$
$$\textit{if } \Delta\tau_{new} \cong \Delta\tau_m, \quad [K] = 1/(1/[K_f] + 1/[K_s]) \tag{6}$$

Reaction Forces of the Internal Column

The time history response of the measured forces (axial loads and lateral load) at the base of the internal (X2Y2) column is shown in Fig. 2. The initial axial load on the column is 313(kN). The maximum response of the lateral force was 259(kN), which was attained at 9.54(s) under the large axial load of 660(kN), after the maximum response of the base shear coefficient at 6.75(s).

The shear at calculated at the flexural strengths with the two plastic hinges is 255 (kN) under the total axial load 962 (kN), which was the maximum value measured during shaking test. The shear strength calculated from the Japanese design equation (Arakawa's equation) is 258(kN) under an initial axial load. Those calculated strengths are almost equal to the maximum responses observed in the shaking test, which conforms to the flexure shear failure mechanism. Although there was no obvious increment for the stable axial load value, the varying axial load attained over 400 (kN) three times after the brittle shear failure occurred in the short columns in X1 frame (6.75(s), 7.56(s)). The residual axial load was finally 215 (kN) in tension after the flexural-shear failure.

The first yielding of the tension bars of this column occurred at 7.56(s) in the negative direction, and the hoops yielded at 11.12(s), which coincided with the reduction of the axial force. The backbone curve shows deterioration after the maximum shear strength at the drift ratio of 2/100. The lateral force oscillates three times with very high frequency in the first specimen, which might have been induced by the brittle shear failure of the short columns in X1 frame.

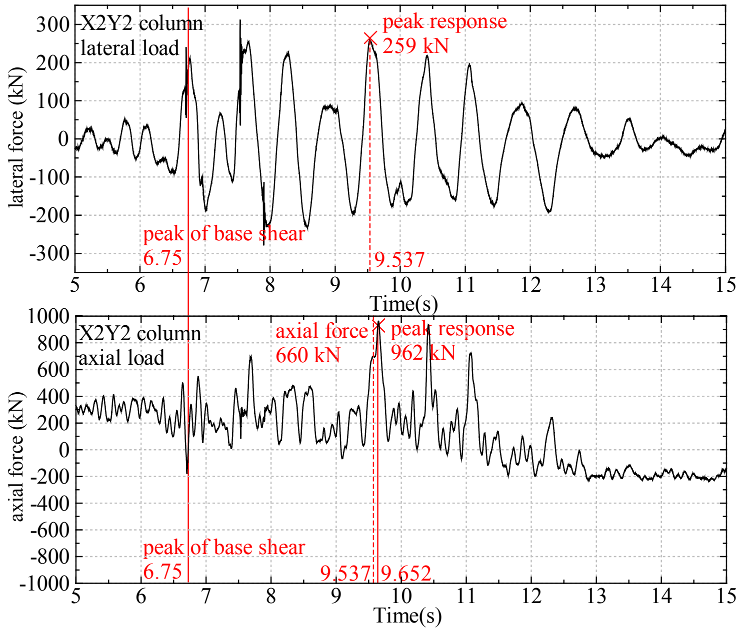

Figure 2. Observed time history response forces of the internal (X2Y2) columns

Static Loading Analysis of the Internal Column

The relations between the lateral force on the column and the 1st story drift ratio are shown in Fig. 3. The static loading analysis of the internal column was carried out with the original ASFI model, in order to estimate the ultimate deformation capacity as an independent column. The analytical results are compared with the shaking test results in Fig. 3. The inflection points were fixed at the mid-height of the columns, and the effect of the beam flexibility was ignored in the analysis. The first analysis was carried out under the constant axial load, while the second analysis with the axial load varied as measured in the test.

The equivalent stiffness degrades temporarily before the oscillation at the 1/100 drift ratio, and the strength deteriorated around 3/100-drift ratio in the shaking test. The first analysis result with the initial axial loads shows flexural behavior after yielding at 1/100 drift. The analysis results in the second model shows degrading skeleton similar to the shaking test result, due to the varying axial loads. The calculated ultimate deformation capacity is a drift ratio of 7/200, where the observed strength also shows obvious reduction in the specimen. It is verified that the analysis with ASFI model evaluates actual behavior and ultimate deformation of the column in the shaking test.

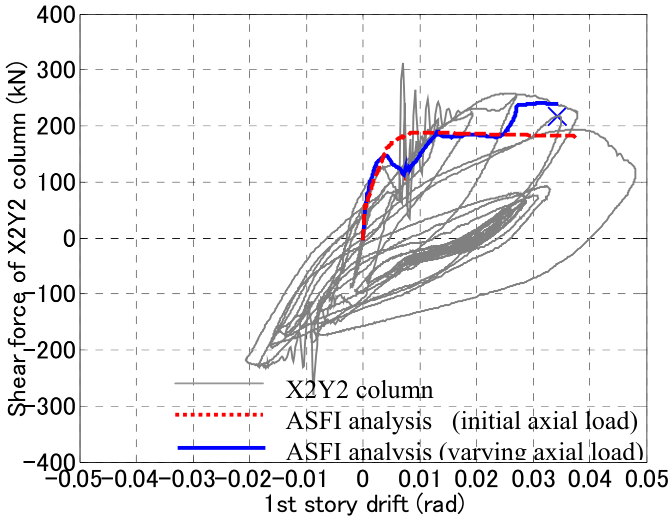

Figure 3. Hysteretic relations of the internal (X2Y2) columns compared with ASFI analysis

Analytical Model

The nonlinear static pushover of shaking table test specimen was carried out with frame model. Calculated floor mass for the analysis model are as follows: RF: 1103.7 (kN), 3F: 789.2 (kN), 2F: 789.2 (kN), 1F: 855.5 (kN). The structures are idealized using the one-component model for beams and wall side column, and the three-vertical line elements (TVL) model for the structural walls. The cracking forces, and yielding force of members in its member hysteresis model, Takeda model, are calculated in accordance with AIJ guidelines. The secant yielding stiffness is calculated by an empirical formula for beams and columns, 0.3 for bending spring, and 0.2 for shear spring of structural walls. The stiffness after yielding is 0.01 of elastic stiffness. Basically, the hysteresis models for bending, axial and shear spring are tri-linear model, axial-stiffness model and linear model, respectively. The effect of the slab on the flexural strength of the beam is considered for the effective width 10% of span length. The rebar and concrete section of the spandrel walls is taken into consideration in evaluation of stiffness, cracking and yielding strengths of the beam. The beam-column joint is assumed to be rigid zone.

Conventional fiber model and ASFI model are adapted to all of independent columns. The rigid zone of the short column is assumed up to the height of the spandrel wall attached to the beam. The actual strength of the concrete was 31 (MPa) during the shaking test. The yielding force of rebar was 392 (MPa) from the material test. The concrete model in fiber section is based on the modified Kent and Park model considering confinement effect for core concrete. The simple bilinear model is used for s hysteretic relation of steel bar. Elastic modulus of concrete is assumed to be 2.46e+07 (kN/m^2) based on AIJ guideline. Hysteretic models for steel bar are idealized with simple bilinear model. The stiffness after yielding is 0.001 of elastic stiffness.

The ASFI column model is to represent strength deterioration by incorporating the softening behavior in concrete constitutive law. The constitutive law for the concrete material model on inflection point is shown in Fig. 4. The compressive strain at the maximum strength is 0.2 percent in the model based on the cylinder test, while the ultimate strain is 0.6 percent. The maximum compression strength in principal direction was reduced based on tensile strain in orthogonal direction. Also, the lowest response strength in ASFI model is evaluated with the fiber model when the concrete strength is initially reduced into 30% from the material test.

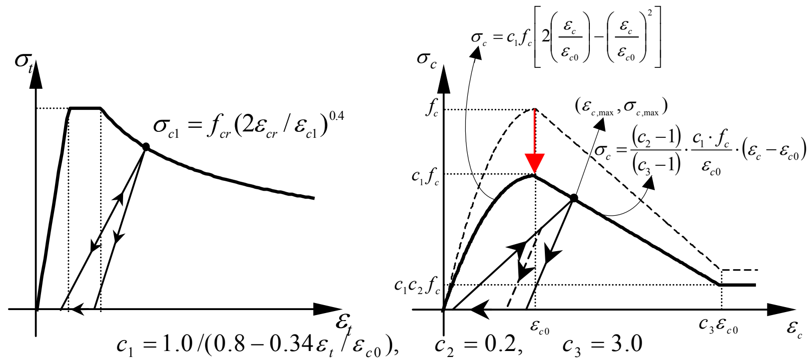

Figure 4. Concrete material model on inflection point

Static Loading Analysis

The ultimate deformation capacity of the structure in shaking test is evaluated in the nonlinear pushover analysis with the inverted triangle distribution forces. The relation between the base shear coefficient and the first story drift ratio is shown in Fig. 5. The three backbone curves are compared with the hysteretic behavior in the shaking test.

The analytical result showed similar backbone curve in the cyclic response before brittle shear failure of X1 columns in conventional fiber model. The strength of the analysis model keeps the maximum response after 1.0 percent drift, because the model cannot simulate the shear collapse. The actual strength also increases until 1.0 percent just after brittle failure of short columns. This result shows clearly the strength of those brittle shear columns deteriorated gradually for high transverse reinforcement, even though obvious diagonal shear crack were observed in center of two short columns before the maximum response of the structure.

The strength of the specimen decreases slightly until 2.5 percent, and prominently deteriorates for the shear failure of the short columns in shaking test. The lower bound strength with fiber models also showed constant base shear coefficient 1.1, which is a little smaller than the actual strength in the post peak response until 2.5 percent drift. The strength in ASFI model gradually decreases until 2.0 percent drift, and the stress does not finally converge between fiber and axial shear model when the response force is close to the lower bound strength in fiber model. This is because the ASFI model cannot simulate reducing the moment strength for buckling of the compression bars. The strength degrading occurred after 2.0 percent drift in ASFI model for compression failure of cover concrete for first damaged column X1Y2, where diagonal cracks were observed on short columns in the test as well. The strength in the shaking test shows obvious deterioration after the response force was smaller than the strength in the fiber model with lower bound concrete strength.

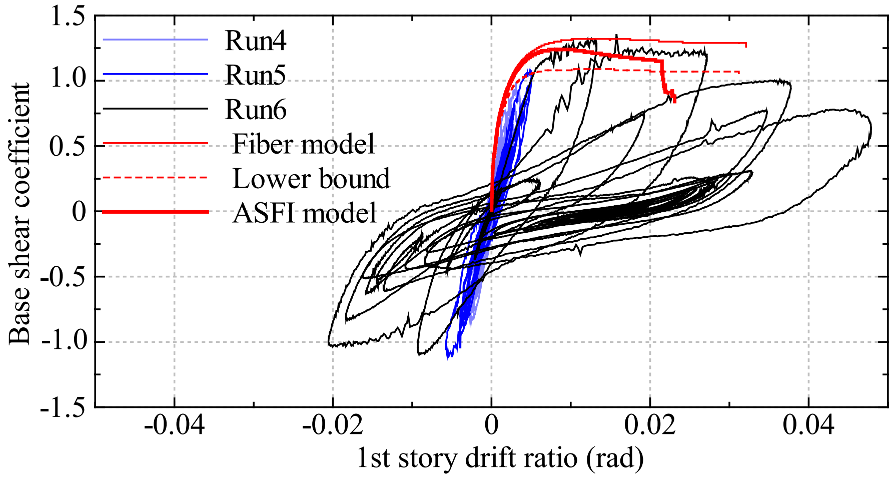

Figure 5. Hysteretic Relation between Base shear coefficient and 1st story drift ratio compared with backbone curves in nonlinear static frame analysis.

Responses of Internal Short Columns in Static Loading Analysis

In order to investigate the response of the element in ASFI model, Figure.6 shows force displacement relation of short columns in Y2 and Y3 frame with solid lines. Dotted lines show the analytical result with the fiber model, and tinted lines show the analytical result with lower concrete strength in fiber model.

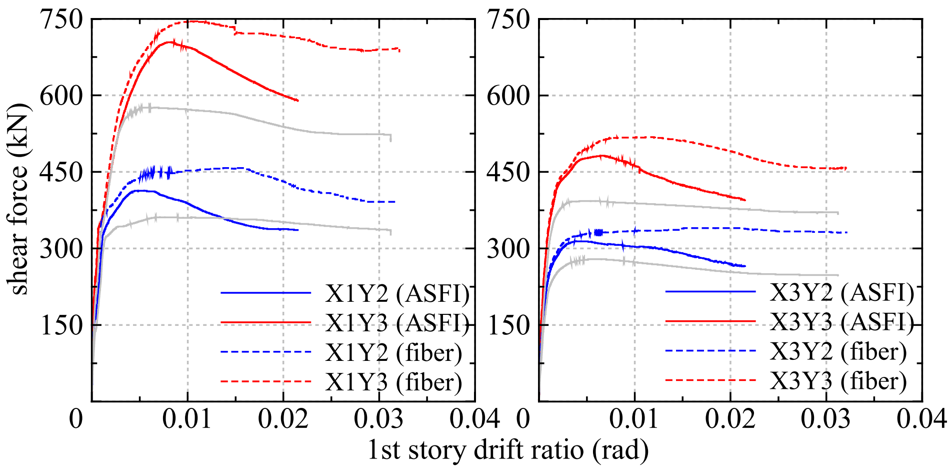

Figure 6. Force-Displacement relation of internal short columns on 1st floor

All of the short columns start deterioration of the response force before the maximum moment-shear strength with two plastic hinge mechanisms in fiber model, while the deteriorating slopes are gradual. This is because the ratio of axial force to the axial load carrying capacity is relatively low, and the ratio of the confinement hoops was enough in those column sections of low-rise building structures. The column strengths decrease up to 80 percent of the maximum response in ASFI model. The response of the X1Y2 short columns firstly approximated to the analytical result with lower strength concrete in fiber model, and the model does not converge between flexural and shear stress in the short column as well as the structure.

Nonlinear Earthquake Response Analysis

The nonlinear dynamic collapse analysis of the structure was carried out with two fiber model and ASFI models as compared with the test results in Fig. 7. The elongation of the initial period is reflected with the continuous response analysis from previous static cyclic loading until experienced displacement. The maximum base shear coefficient by the analysis with the conventional fiber model is almost equal to the actual maximum response in the shaking test, while the maximum response deformation is much smaller than the test results, because of stable and non-deteriorating behavior especially in the negative direction. The strength of the reloading point does not decrease, and the reloading stiffness is relatively high compared to the response of the structure in shaking table test and ASFI model. The hysteretic and displacement responses by ASFI model are similar to those of the test until 0.03 (rad), although the response force on the backbone curve is slightly smaller than the test and fiber model. The reloading stiffness in the opposite direction degrades with the incremental displacement in ASFI model.

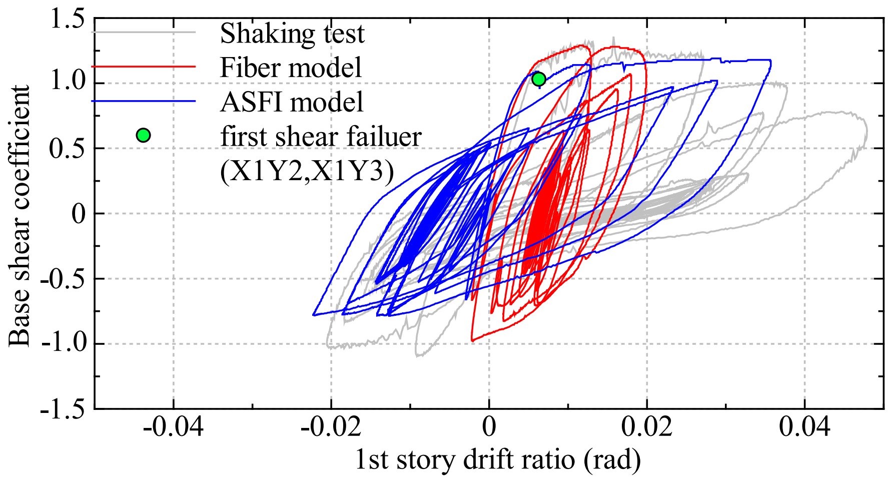

Figure 7. Hysteretic Relation between Base shear coefficient and 1st story drift ratio in nonlinear earthquake response analysis

Conclusions

The full-scale shake table tests on three-story reinforced concrete school buildings were analyzed with the original and modified ASFI model. The following conclusions may be drawn from the analyses compared with the test results:

The time history responses of the forces measured at the base of the internal (X2Y2) column in the specimen was shown and analyzed. The internal column showed flexural shear failure because the phase of the shear and the axial forces varied due to the brittle shear failure of the short columns. The backbone curve and the ultimate deformation capacity of the internal column were evaluated with the original ASFI model. The analytical results with the varying axial loads showed similar backbone curve observed in the test.

The original ASFI model was modified so as to be available as a member model in the frame analysis. The non-linear pushover analysis was carried out, and compared with the test results of the RC specimen. The stiffness and strength degraded obviously at 0.5 percent drift ratio, where the brittle shear failure of the short columns occurred in the shaking test. The non-linear earthquake response analyses were also carried out with the fiber model and the ASFI model. The analytical responses with the ASFI model simulated much better the hysteretic responses in the test with strength degradation than those with the conventional fiber model.

Acknowledgments

The full-scale test was carried out as a part of “National research project on mitigation of major disaster in major city/ Theme II Improvement of seismic performance of structures using E-defense/ Reinforced concrete structures (DaiDaiToku/RC project)” of MEXT. The full-scale test was carried out at E-Defense, NIED in Miki in 2006.

References

Hossein Mostafaei, 2006. Axial-Shear-Flexure Interaction Approach for Displacement-Based Evaluation of Reinforced Concrete Elements, *Doctor of Engineering theses*, The University of Tokyo, Japan.

Hossein Mostafaei, Frank F.J. Vecchio and Toshimi Kabeyasawa, 2009. Deformation capacity of reinforced concrete columns, *ACI Structural Journal*, Vol.106, No.2, 187-195

AIJ, 1975. *AIJ Standard for Structural Calculation on Reinforced Concrete Structures.*

Kim, Y., T. Kabeyasawa, and T. Kabeyasawa, 2006. Preliminary Response Analysis of a Full-Scale Structure for 3-D Shaking Table Test to Collapse, *Proceeding of the 8th National Conference on Earthquake Engineering*, Paper No. 479, San Francisco, USA,

Toshimi Kabeyasawa, Taizo Matsumori, Toshikazu Kabeyasawa, Toshinori Kabeyasawa, 2007. Plan of 3-D dynamic collapse tests on three-story reinforced concrete buildings with flexible foundation, *ASCE Structural Congress*, *Research Frontiers Track Session, Collapse Simulation and Experimental Studies for Reinforced Concrete Buildings Part II*, Long beach, USA.

Toshikazu Kabeyasawa, Toshimi Kabeyasawa, Taizo Matsumori, Toshinori Kabeyasawa, 2007. 3-D collapse tests and analyses of the three-story reinforced concrete buildings with flexible foundation, *ASCE Structural Congress*, *Research Frontiers Track Session, Collapse Simulation and Experimental Studies for Reinforced Concrete Buildings Part II*, Long beach, USA.