This paper presents a 3D shake table experiment on a large scale reinforced concrete bridge column using E-Defense. The model was a typical reinforced concrete column which was built in the 1970s in Japan. Collapse of this type of column was one of the major causes of the extensive damage during the 1995 Kobe, Japan earthquake. A 7.5 m tall, 1.8 m diameter column model was excited twice by a near-field ground motion which was recorded during the 1995 Kobe earthquake. Both experimental response and analytical correlation are presented.

1. INTRODUCTION

E-Defense is a full-size, three-dimensional shake table of the National Research Institute for Earth Science and Disaster Prevention, Japan1 that is the largest in the world. The table is 20 m long and 15 m wide with a full load of 1 200 tf. Peak acceleration, velocity and displacement that can be excited by E-Defense under full load are 0.9 g, 2 m/s, +/−1 m, respectively, in the two lateral directions, and 1.5 g, 0.7 m/s and +/−0.5 m in the vertical direction.

It was built to advance the scientific knowledge in earthquake engineering as a consequence of the extensive damage of urban infrastructures in the 1995 Kobe, Japan earthquake. “Why did structures suffer such extensive damage during the Kobe earthquake?”, “what was the mechanism of collapse?”, and “to what extent do structures fail under near-field ground motions?” are the basic motivations of constructing E-Defense.

A large scale bridge experimental program was initiated in 2005 as one of the three US-Japan cooperative research programs based on NEES and E-Defense collaboration. The program was formulated based on discussions among Japan and US researchers since April 2004. It was decided at the 4th planning meeting in August 2005 in Miki to conduct experiments on two model types; 1) component models and 2) system models. They are called hereinafter as C1 experiment (C1 models) and C2 experiment
The purpose of C1 experiment is to clarify the failure mechanism of single reinforced concrete columns using models with as large size and section as possible. The objectives of the C1 experiment are to clarify 1) the failure mechanism of reinforced concrete columns which failed during the 1995 Kobe earthquake, 2) the effectiveness of standard seismic retrofit measures for existing RC columns, 3) the seismic performance of reinforced concrete columns designed in accordance with the current design requirements, 4) the seismic performance of reinforced concrete columns designed based on the current design requirements under ground motions stronger than the specified in the current code, and 5) the effect of new damper technology.

On the other hand, C2 experiments were proposed to clarify the system failure mechanism of a bridge consisting of decks, columns, abutments, bearings, expansion joints and unseating prevention devices. The objectives of C2 experiment are to clarify 1) the progress failure mechanism of a bridge system due to the combination of poundings and rupture of expansion joints, bearings, restrainers and columns, 2) the seismic performance of critical columns and columns utilizing advanced materials and, 3) the effectiveness of advanced dampers and energy dissipating units, and 4) the effectiveness of advanced unseating prevention devices.

After preliminary experiments and analyses in 2005 and 2006, the first shake table experiment was conducted in December 2007 for a large scale reinforced concrete column that represents typical columns built in the 1970s (C1-1 column). This paper introduces the experimental evidence and analytical correlations on the C1-1 column.

2. COLUMN MODEL

C1-1 column is a typical column which was built in the 1970s. Collapse of this type of column was one of the major causes of the extensive damage during the 1995 Kobe, Japan earthquake. C1-1 column is a 7.5 m tall, 1.8 m diameter reinforced concrete column as shown in Fig. 1. It was anchored to E-Defense by a 1.8 m thick square footing. It was designed as a full-scale model based on a combination of the static lateral force method and the working stress design (seismic coefficient method) which were specified in the 1964 Design Specifications of Steel Road Bridges, Japan Road Association (JRA). Combination of the lateral seismic coefficient of 0.23 and the vertical seismic coefficient of $+/−0.11$ (upward and downward seismic force) was assumed in the design.

The column had eighty 29 mm diameter longitudinal bars in three layers, i.e., 32, 32 and 16 bars at the outer, center and inner layers. The longitudinal bars have a nominal yield strength of 345 MPa (SD345). A series of loading tests were conducted for SD345 bars. For example, Fig. 2 shows the stress vs. strain hysteresis of an 18 mm diameter coupon which was fabricated by grinding ribs. The coupons were taken from the same rods with the bars used for the column.

Deformed circular ties with 13 mm diameter were provided at 300 mm interval, except the outer ties at the top 1.15 m zone and the base 0.95 m zone where they were provided at 150 mm interval. Ties were lap spliced with 30 times the bar diameter. Lap splicing was a common practice in the mid 1980s.

Consequently, the longitudinal reinforcement ratio is 2.02% and the tie volumetric reinforcement ratio $\rho_t$ is 0.32% except the top 1.15 m and base 0.95 m zones where $\rho_t$ is 0.42%.

Design strength of concrete is 27 MPa. Averaged strength of concrete based on the cylinder test at the day of experiment was 33.0 MPa. The measured strength was used in the analysis shown in Section 7.

Table 1 shows the evaluation of the seismic per-
Table 1 Evaluation of C1-1 and C1-5 column based on 2002 JRA.

| Demand and Capacity | Model Column | C1-1 | C1-5 |
|---------------------|--------------|------|------|
|                     | Direction    | LG   | TR   | LG   | TR   |
| Lateral Force       | Design response acceleration $S_A$ (m/s$^2$) | $1.75 \times 9.8$ | $21.2$ |
|                     | Force reduction factor $R = \sqrt{2\mu_d - 1}$ | 1.58 | 1.53 | 2.56 | 2.42 |
|                     | Acceleration demand $S_A/R$ (m/s$^2$) | 10.83 | 11.24 | 6.70 | 7.09 |
| Demand              | Lateral force (kN) | 3.27 | 2.31 | 2.02 | 1.46 |
|                     | Lateral displacement $u$ (m) ($\mu = u/\mu_y$) | 0.328 | 0.319 | 0.168 | 0.170 |
| Capacity            | Lateral force $P_u$ (kN) | 1.614 | 1.388 | 2.341 | 1.992 |
|                     | Yield displacement $u_y$ (m) | 0.046 | 0.062 | 0.045 | 0.060 |
|                     | Design displacement $u_d$ (m) ($\mu_d = u_d/\mu_y$) | 0.081 | 0.103 | 0.169 | 0.208 |
|                     | Ultimate displacement $u_u$ (m) ($\mu_u = u_u/\mu_y$) | 0.099 | 0.123 | 0.231 | 0.281 |

3. EXPERIMENTAL SETUP

The C1-1 column was set on the shake table as shown in Fig. 3 and Photo 1. Two simply supported decks were set on the column and on the two steel end supports. The decks were set up to fix the mass blocks to the column however they were not designed with the stiffness and strength of real decks. Each deck was supported by a fixed bearing on the column and a movable bearing (friction bearing) on the end supports as shown in Fig. 4. Two side sliders (friction bearings) were set beside the fixed bearing for preventing rotation of the decks around their axis. Two more side sliders (friction bearings) were also set beside a movable bearing for preventing excessive rotation of the deck over 10 degree. However, they were not in action during the excitations because the deck rotation did not reach 10 degree. This complex support condition was adopted so that col-

![Fig. 3 Set up of the model on E-Defense.](image-url)
Fig. 4 Supporting condition of two decks on the column and two end supports.

The table was excited using a near-field ground motion recorded at the JR Takatori Station during the 1995 Kobe, Japan earthquake. It included long-pulses which resulted in large structural response. Consequently it was one of the most influential ground motions to structures. However the duration was very short.

It is well known that the radiational energy dissipation of a column anchored to a shake table is extremely smaller than the real energy dissipation of a column embedded in the ground7). Taking account of the soil structure interaction effect, a ground motion with 80% the original intensity of JR Takatori record was imposed as a command to the table in the experiment8). This ground motion is called hereinafter as the 100% E-Takatori ground motion. NS, EW and UD components of the E-Takatori ground motion were imposed in the longitudinal, transverse and vertical directions, respectively, of the column.

The model was first subjected to a 10%, two 20% and six 30% E-Takatori ground motion excitations to check the response and measurement. No visible cracks occurred on the column during those excitations. The second 100% E-Takatori ground motion excitation was conducted to clarify whether damage developed during the first 100% E-Takatori ground motion excitation would progress if the column was subjected to a much longer duration near-field ground motion or strong aftershocks.

Pulse excitations were conducted prior to and after
each excitation to estimate the natural period of the column. A number of white noise excitations with small peak table accelerations (nearly 0.03 m/s²) were conducted to tune up the control of E-Defense so that the table motion can be as close to the command as possible.

**Fig. 5** shows the 100% E-JR Takatori ground motions. Both the target table acceleration and recorded table acceleration during the first 100% E-Takatori ground motion excitation are shown here for comparison. The measured table accelerations are fairly in good agreement with the commands. As shown in **Fig. 6**, the E-Takatori ground motion has over 15 m/s² response accelerations at 0.3-0.5 s and 1.2 s.

A total of 741 channels of accelerations, displacements, strains, curvatures, forces and control signals are monitored as well as video and LED monitoring during the excitation.

### 4. VARIATION OF THE NATURAL PERIODS

**Fig. 7** shows the variation of the natural periods of the column in accordance with the progress of excitations. They were estimated based on the pulse excitation tests prior to and after each excitation. Computed natural periods of the column assuming the uncracked section stiffness and the yield stiffness are shown here for comparison. The natural period of the column was 0.486 s and 0.576 s in the longitudinal and transverse directions, respectively, after small intensity level excitations were completed. The measured natural periods are between the computed natural periods assuming the uncracked stiffness and the yield stiffness.

The natural periods of the column in the longitudinal and transverse directions increased to 0.765 s and 0.880 s after the first 100% E-Takatori ground motion excitation and 0.894 s and 1.024 s after the second 100% E-Takatori ground motion excitation. The natural periods of the column after the second 100% E-Takatori ground motion excitation are longer than the natural periods at the initial stage by 1.84 and 1.78 times in the longitudinal and transverse directions, respectively.

### 5. SEISMIC PERFORMANCE DURING THE FIRST 100% EXCITATION

(1) Overall response and failure mode

**Figs. 8** and **9** show the response accelerations and displacements, respectively, at the top of the column. The peak displacement at the column top was 0.173 m and 0.134 m in the longitudinal and transverse directions, respectively. Combined displacement of the two lateral components had a peak of 0.195 m (2.56% drift) at 6.9 s. Because the computed ultimate displacement at the top of the column is 0.091 m, the peak response of the combined displacement is 2.1 times the computed ultimate displacement. Residual
displacement of 0.018 m (0.24% drift) and 0.017 m (0.23% drift) occurred at the end of the excitation in the longitudinal and transverse directions, respectively, but they are sufficiently small. The peak acceleration was 8.79 m/s$^2$ and 9.38 m/s$^2$ in the longitudinal and transverse directions, respectively as shown in Table 2.

### Table 2 Peak responses during the first 100% E-Takatori ground motion excitation.

(a) Peak response accelerations at the top of column (m/s$^2$)

| Directions | LG | TR | UD |
|------------|----|----|----|
| + Peak     | 8.76 | 8.24 | 4.41 |
| – Peak     | –8.79 | –9.38 | –3.56 |

(b) Peak response displacement at the top of column (m)

| Directions | LG  | TR  | Combined |
|------------|-----|-----|----------|
| + Peak     | 0.173 | 0.134 | 0.195 |
| – Peak     | –0.170 | –0.119 |          |

### Photo 2 Progress of failure of the column during the first 100% E-Takatori ground motion excitation at SW surface.

**Photo 2** shows the progress of column failure at the plastic hinge on the SW surface where damage was most extensive. It should be noted that NS and EW directions correspond to the transverse and longitudinal directions, respectively, of the model. At an instance of 6.9 s, covering concrete started to spall at the SW surface between the base and 0.6 m from the base of the column due to compression. At least two outer longitudinal bars from S to W locally buckled between the ties at 200 mm and 500 mm from the base of the column during the excitation. Buckling of the center and inner bars did not occur or at least was not significant because they were still
covered by the covering concrete. At the opposite side (NE), covering concrete spalled between the base and 0.2 m from the base, but damage of covering concrete was much less without exposure of any bars.

(2) Deformation of longitudinal bars

Fig. 10 shows the strain distribution of the longitudinal bars in the vertical direction at 6.9 s when the combined two lateral components took a peak displacement. Strains at the eight surfaces (N, NE, E, SE, S, SW, W and NW) as well as the displacement orbit of the combined two lateral components at the column top are shown in Fig. 10. It should be noted here that strains of the outer, center and inner longitudinal bars were measured at N, E, S and W surfaces, while only strains of the outer longitudinal bars were measured at NE, SE, SW and NW surfaces. It should also be noted that measurement of bar strains was difficult due to damage of sensors, thus exact identification of true and distorted strain values was difficult. Consequently, overall strain distribution not the single point values has to be evaluated.

Based on Fig. 10, strains in the longitudinal bars were over 10,000 µ in tension at the SE, E, NE, N and NW surfaces while they were over 5,000 µ in compression at the SW and W surfaces. The fact that large compression strains were developed in the longitudinal bars implies that the core concrete had already been damaged allowing local buckling of longitudinal bars to occur. Strains in the longitudinal bars are extremely large between 0.25 m below and 1.5 m above the base of the column. Because the plastic hinge length is 0.9 m based on the design code⁶, it is important to note that longitudinal bars extensively yielded at the zone above the plastic hinge region.

(3) Deformation of circular ties

Fig. 11 shows the strains of circular ties at 6.9 s during the first 100% E-Takatori ground motion
excitation. In particular, strain distribution along ties at 50 mm, 350 mm, 650 mm and 950 mm from the base of the column are shown. Because strains of ties at 1 250 mm, 1 550 mm and 1 850 mm were small, they were not shown here. It should be noted that strains of outer, center and inner ties were measured at 50 mm, 350 mm, 650 mm and 950 mm, while only strains of outer ties were measured at 1 250 mm, 1 550 mm and 1 850 mm. Strains of tie bars reached nearly 2 000 μ at 350 mm and 650 mm, slightly larger than the yield strain and nearly 1 000 μ at 50 mm and 950 mm from the base of the column. Between 1 250 mm and 1 850 mm, the strains were only nearly 500 μ. Consequently it is important to note that the ties were still in the elastic or slightly inelastic range. It is also important to note that strains in the outer ties are larger at the SW and W surfaces where the section is subjected to compression. As will be described later, this resulted from the local buckling of longitudinal bars at the SW surface.

It should also be noted in Fig. 11 that the lateral confinement by ties is very complex. The lateral confinement is not uniform around the ties as it is generally assumed when the lateral confinement is evaluated in design9. The tie strains are not the same among the three ties. For example, outer bars yielded at the SW and W surfaces while strains of center and inner ties are still less than 1 000 μ at 350 mm.

Fig. 12 shows how strains of three ties at 350 mm from the base of the column vary between 6.9 s and 8.0 s. Strains are generally larger in the outer ties than the center and inner ties. It is noted that strain of a tie in a layer (outer, center or inner) becomes large independently with ties in other layers. For example, strain of an outer tie at 7.6 s is largest at NE (1 513 μ), but strains of center and inner ties are small. On the other hand, strain of an inner tie at 7.6 s is largest at S (1 925 μ) but strains of center and outer ties are small. Based on the current design code, the volumetric tie reinforcement ratio ρ is evaluated in C1-1 as

\[
\rho_s = \rho_{sO} + \rho_{sC} + \rho_{sI}
\]

where, ρ_{sO}, ρ_{sC} and ρ_{sI} are volumetric tie reinforcement ratio of the outer, center and inner ties, respectively. However Fig. 12 shows that estimation of the volumetric tie reinforcement ratio by Eq. (2) can be overestimated for evaluating the lateral confinement of the core concrete. Mechanism of the lateral confinement by multi-layered ties should be critically clarified.

Fig. 13 shows the strains in the outer, center and inner layers of both longitudinal bars and tie bars. As described earlier, it was observed after the excitation that the outer longitudinal bar locally buckled while the center and inner longitudinal bars still did not buckle. Because a compression strain over 15 000 μ developed in the outer longitudinal bar at 6.9 s, buckling of the outer longitudinal bar must have occurred at this time. It is important to note that strain of the outer tie reached 2 300 μ at the same time. This suggests a mechanism that the outer tie restricted the local buckling of the outer longitudinal bar, and that this resulted in a sharp increase of strain of the outer tie.

Fig. 14 shows the interaction of a longitudinal bar 300 mm from the base and a tie bar 350 mm from the base at the W surface. Fig. 14 (a) shows the

![Diagram](image1)

Fig. 11 Strains of ties at 6.9 s during the first 100% E-Takatori ground motion excitation.

![Diagram](image2)

Fig. 12 Strains of Ties at 350 mm from base.
(a) Longitudinal bars                                    (b) Tie bars

Fig. 13 Strains of longitudinal bars at 300 mm and tie bars at 350 mm from base at W surface.

Fig. 14 Hysteresis of strain of longitudinal bars at 300 mm and tie bars at 350 mm from base at W surface.

hysteresis of strains of the outer longitudinal bar and the outer tie. An increase of strain in the outer tie which resulted from restraining the local buckling of the outer longitudinal bar under high compression strain is clearly seen. On the other hand, such an increase of strain in the outer tie is not seen in the center and inner bars as shown in Fig. 14 (b) and (c) because longitudinal bars did not yet buckle.

(4) Curvature and moment capacity

Fig. 15 shows the curvature responses at 180 mm, 380 mm and 580 mm from the base and Fig. 16 shows the curvature distribution at an instance of 6.9 s. Curvatures were computed from relative displacement measured between heights \( h_i \) and \( h_{i+1} \) by LVDT, and an averaged curvature between \( h_i \) and \( h_{i+1} \) is shown here as a curvature at a height of \( (h_i + h_{i+1})/2 \) in Figs. 15 and 16. Because the curvature at the lowest measurement (40 mm from the base) was determined from relative displacement between the surface of footing and 80 mm from the base, and because the relative displacement contributed from the “pulling out” of longitudinal bars from the footing and the relative displacement contributed from the curvature between the base and 80 mm from the base cannot be distinguished, it should be noted here that “pulling out” effect of longitudinal bars is counted here as curvature at the lowest measurement. Large scattering of the measured curvatures depends on where cracks
occurred. Because the computed yield curvature is 0.00164 1/m, curvature over the yield curvature occurred up to 1.58 m from the base, which is higher than the anticipated plastic hinge zone.

Bending moment at the base of the column $M_{CB}$ is evaluated as

$$M_{CB, LG} = P_{LG}(h + \Delta h) - P_{UD}u_{LG} - \int_0^h \lambda \ddot{u}_{LG} x \, dx$$

(3)

$$M_{CB, TR} = P_{TR}(h + \Delta h) - P_{UD}u_{TR} - \int_0^h \lambda \ddot{u}_{TR} x \, dx$$

(4)

where, $P_{LG}$, $P_{TR}$, $P_{UD}$ = inertia force transferred from two decks and four mass blocks to the top of the column in the longitudinal, transverse and vertical directions, $h$ = column height, $\Delta h$ = distance from the column top to the center of two fixed bearings, $u_{LG}$, $u_{TR}$ = displacement response at the column top, $\lambda$ = column weight per unit length, $x$ = height from the base, and $\ddot{u}_{LG}$, $\ddot{u}_{TR}$ = response acceleration of the column at $x$ height from the base.

Fig. 17 shows the hystereses of the moment at the base $M_{CB}$ vs. lateral displacement at the top of column $u$. The computed yield displacement, design displacement and ultimate displacement based on the JRA 2002 are also shown on the moment $M$ vs. lateral displacement $u$ relation (skeleton curve) for comparison. It is noted that the moment capacity of the column is nearly the same with the computed moment capacity based on JRA 2002. It is also noted that the moment capacity of the column does not yet deteriorate over 2.1 times the computed ultimate displacement.

6. SEISMIC PERFORMANCE DURING THE SECOND 100% EXCITATION

Figs. 18 and 19 show the response accelerations and displacements at the top of the column. The peak

combined displacement of two lateral components reached 0.314 m (4.19% drift) at 6.97 s as shown in Table 3. Residual displacement was 0.012 m and 0.017 m in the longitudinal and transverse directions, respectively. Because residual displacements were 0.018 m and 0.004 m in the longitudinal and transverse directions, respectively, after the first 100% E-Takatori ground motion excitation, they decreased during the second excitation. It is important to note that residual displacement not only increases but also decreases during seismic excitations.
**Fig. 18** Response acceleration of column at the top during the second 100% E-Takatori ground motion excitation.

**Photo 3** Progress of failure of the column during the second 100% E-Takatori ground motion excitation at SW surface.

acceleration was $-10.53 \text{ m/s}^2$ and $-9.21 \text{ m/s}^2$ in the longitudinal and transverse directions, respectively.

**Photo 4** Damage of the column after the second 100% E-Takatori ground motion excitation.

**Photo 4** Damage of the column after the second 100% E-Takatori ground motion excitation.
was much less with only tension cracks on NE.

Photo 4 shows the column after damaged covering and core concrete were removed. Both covering and core concrete suffered extensive damage between the base and 0.7 m from the base of the column at the SW surface. Eleven outer and three center longitudinal bars locally buckled between ties at 50 mm and 500 mm from the base. Local buckling was not identified for the inner longitudinal bar because they were still covered by the covering concrete.

Similar to the first 100% E-Takatori ground motion excitation, Fig. 20 shows strains of the longitudinal bars at 6.97 s. Strains of longitudinal bars further increased during the second excitation and became extremely large between 0.25 m below and 1.5 m above the base of the column even at the N surface. As shown in Fig. 21, strains of ties extensively increased during the second 100% E-Takatori ground motion excitation. All ties except at 1,550 mm and 1,850 mm yielded. It is interesting to note that ties at 1,250 mm which were located above the plastic hinge region yielded.

It is noted that the longitudinal and tie bars yielded as high as 83% and 69% the column diameter, respectively, from the base. It is also important to note that progress of damage during the second 100% E-Takatori ground motion excitation was more extensive than that of the first excitation. Because the progress of damage during the second 100% E-Takatori ground motion excitation resulted from the separation of ties at the lap splices, it is highly possible that columns without sufficient lateral confinement such as those built based on the pre-1980 design codes can have a similar progress of damage during a long-duration ground motion or strong after-shocks.

Fig. 22 shows curvature of the column at 6.97 s during the second excitation. The curvature at 6.9 s during the first excitation is also shown here for comparison. The curvature during the second excitation almost doubled the curvature during the first excitation.

Fig. 23 shows the moment vs. lateral displacement hystereses. The moment capacity of the column during the second excitation deteriorated from the first excitation than that of the first excitation.
moment capacity during the first excitation by 25% and 32% in the longitudinal and transverse directions, respectively.

7. ANALYTICAL CORRELATION

(1) Analytical idealization

The column was idealized by a 3D discrete analytical model as shown in Fig. 24. The column was idealized by fiber elements. A section was divided into 400 fibers. To clarify the effect of length of a fiber element $l_c$, it was varied as $D/2 (= 900$ mm), $D/6 (= 300$ mm) and $D/18 (= 100$ mm) where $D$ is the width of column. $P - \Delta$ effect was included in the analysis although its effect was limited.

The stress vs. strain constitutive model of confined concrete is assumed as (refer to Fig. 25[9,11])

$$f_c = \begin{cases} E_c e_c \left\{ 1 - \frac{1}{n} \left( \frac{e_c}{e_{cc}} \right)^{n-1} \right\} & (0 \leq e_c \leq e_{cc}) \\ f_{cc} - E_{des}(e_c - e_{cc}) & (e_{cc} \leq e_c \leq e_{c0}) \\ a_f e_{cc} & (e_{c0} \leq e_c) \end{cases}$$

(5)
in which $f_{cc}$ and $e_{cc}$ = strength of confined concrete and strain corresponding to $f_{cc}$, $E_c$ = elastic modulus of concrete, $E_{des}$ = gradient at descending branch, $a = $ residual strength factor depending on the confinement, and $n = E_c e_{cc}/(E_c e_{cc} - f_{cc})$. In Eq. (5), $f_{cc}$, $e_{cc}$, $E_{des}$, $e_{c0}$ and $a$ are defined as

$$f_{cc} = f_{c0} + 3.8a \rho_s f_{sy}$$

(6)

$$e_{cc} = 0.002 + 0.033\beta \rho_s f_{sy}/f_{c0}$$

(7)

$$E_{des} = 11.2(\frac{f_{c0}^2}{\rho_s f_{sy}})$$

(8)

$$e_{c0} = e_{cc} + 0.8f_{cc}/E_{des}$$

(9)

$$a = 0.2$$

(10)
in which $f_{c0}$ = design strength of concrete, $f_{sy}$ = yield strength of tie bars, $a$ and $\beta$ = shape factors (in $\alpha = 1.0$ and $\beta = 1.0$ for circular piers), and $\rho_s$ = volumetric ratio of tie bars.

Stress vs. strain relation of covering concrete was evaluated by Eq. (5) assuming $\rho_s = 0$ in Eqs. (6) and (7). $E_{des}$, $e_{c0}$ and $a$ are given as
was used to idealize the stress vs. strain relation of curve, and the point where reloading path intersects the envelope and reloading from zero stress are idealized as

\[ F_{26} \]

Fig. 26 Unloading and reloading paths of confined concrete.

\[ E_{des} = \frac{f_0}{\varepsilon_{e0} - \varepsilon_{ec}} \] (11)
\[ \varepsilon_{e0} = 0.005 \] (12)
\[ a = 0 \] (13)

Unloading and reloading hystereses consist of combinations of full unloading, partial unloading, full reloading and partial reloading. For example, as shown in Fig. 26, unloading from an envelope curve and reloading from zero stress are idealized as

\[ f_c = f_{ul1} \left( \frac{\varepsilon_c - \varepsilon_{pl1}}{\varepsilon_{ul} - \varepsilon_{pl1}} \right)^2 \] (14)
\[ f_c = 2.5 f_{uln} \left( \frac{\varepsilon_c - \varepsilon_{pln}}{\varepsilon_{ul} - \varepsilon_{pln}} \right)^2 \] (15)

where

\[ \varepsilon_{pl1} = \begin{cases} 0 & 0 \leq \varepsilon_{ul} \leq 0.001 \\ 0.43(\varepsilon_{ul} - 0.001) & 0.001 < \varepsilon_{ul} < 0.0035 \\ 0.94(\varepsilon_{ul} - 0.00235) & \varepsilon_{ul} \geq 0.0035 \end{cases} \] (16)

in which \( f_{ul1} \) and \( \varepsilon_{ul} \) = unloading stress and strain on the envelope curve, \( f_{uln} \) = stress at the unloading point after \( n \)th unloading/reloading, \( \varepsilon_{pln} \) = plastic strain after \( n \)th unloading/reloading, \( \varepsilon_{re} \) = strain at the point where reloading path intersects the envelope curve, and \( E_{c-r} \) = reloading modulus.

Modified Menegotto-Pinto model (refer to Fig. 27) was used to idealize the stress vs. strain relation of longitudinal bars as

\[ f_s = \begin{cases} f_{s1} & f_{s1} \leq f_{sp} \\ f_{sp} & f_{s1} > f_{sp} \end{cases} \] (for tensile loading) (17)
\[ f_s = \begin{cases} f_{s1} & f_{s1} \geq f_{sp} \\ f_{sp} & f_{s1} < f_{sp} \end{cases} \] (for compressive loading) (18)

\[ \text{Fig. 27 Stress vs. strain relation of a longitudinal bar.} \]

\[ f_{s1} = f_0(f_0 - f_r) + f_r \quad (19) \]
\[ f_{sp} = f_p(f_0 - f_{rp}) + f_{rp} \quad (20) \]

and where

\[ \tilde{f} = \frac{f_0 - f_r}{f_0 - f_r} \quad (21) \]
\[ \tilde{f}_p = R_s\tilde{\varepsilon}_p + \frac{(1 - R_s)\tilde{\varepsilon}_p}{\left(1 + \frac{\tilde{\varepsilon}_p}{R_p}\right)^{1/R_p}} \quad (22) \]
\[ \tilde{\varepsilon}_p = \frac{\varepsilon_s - \varepsilon_{rp}}{\varepsilon_{op} - \varepsilon_{rp}} \quad (23) \]

in which \( R_s \) = strain hardening ratio, \( R_{bp} \) = Bauschinger effect coefficient after partial unloading, \( f_r \) = steel stress at reversal point, \( f_{rp} \) = steel stress at reversal point after partial unloading, \( f_0 \) = steel stress at intersection of two asymptotes, \( f_{0p} \) = steel stress at intersection of asymptotes after partial unloading, \( \varepsilon_{rp} \) = steel strain at reversal point after partial unloading, and \( \varepsilon_{op} \) = steel strain at intersection of asymptotes after partial unloading.

It was assumed in the analysis that strength and Young’s modulus of concrete were 33.0 MPa and 25.1 GPa, respectively, based on a cylinder test at the day of excitation, and the yield strength and Young’s modulus of longitudinal bars were 366.0 MPa and 193.0 GPa, respectively, based on a tensile test. Post yield stiffness of longitudinal bars was assumed as 2% of the elastic stiffness.

Bearings and sliders were idealized as shown in Fig. 28. Because rotation of the decks around the bridge axis due to transverse response of the column can result in separation of the decks from the column, contact and separation at the sliders were idealized by impact springs. The friction coefficient at the movable bearings was assumed to be 0.12 based on a series of loading tests. Dependence of the friction force on the vertical load was disregarded here.

Rayleigh damping was assumed to represent the energy dissipation other than the hysteretic energy dissipation at the plastic hinge of the column. Modal
Response acceleration at the base of the column which was evaluated by averaging the measured accelerations on the footing surface at the four corners was imposed as an input motion to the base of the footing assuming the footing is rigid.

Computer program “TDAP III” was used by including the constitutive models of concrete and longitudinal bars. Time interval of the numerical integration was 0.001 s.

(2) Analytical correlation

Fig. 29 shows the analytical correlation on the response accelerations and displacements at the top of the column during the first E-Takatori 100% excitation. Measured response accelerations and displacements which were presented in Figs. 8 and 9 are shown here for comparison. Computed response displacements are close to the experimental responses although agreement of the computed accelerations with the measured ones is not sufficiently high. It is always the case that accuracy of acceleration responses is rather lower than that of displacement responses. It is noted here that the effect of fiber element length $l_e$ on the correlation of response displacements and accelerations is less significant.

Fig. 30 compares measured and computed curvature responses around longitudinal and transverse axes at 40 mm, 180 mm, 380 mm, 580 mm and 780 mm from the base. Because curvatures above 780 mm are small, they are not shown here. Computed responses using $l_e = 100$ mm are presented because curvatures can be directly compared between analysis and experiment. Overall behavior of the curvature response is well captured by analysis. It should be noted here that the pull-out effect of longitudinal bars is not included in the analysis.

Fig. 31 correlates the peak curvature distribution with the experimental results at an instance of 6.9 s. As anticipated, curvature at the lower height tends to increase with the mid-height curvature being decreased as $l_e$ decreases. As shown earlier, the measured curvature has a large scattering due to cracks. The computed curvatures well represents the general trend of the measured curvature.

Fig. 32 compares the computed and measured hystereses of moment at the base vs. lateral displacement.
Fig. 30 Analytical correlation on curvature response for the first 100% E-Takatori ground motion excitation.

Fig. 31 Comparison of measured and computed distribution of peak curvatures at an instance of 6.9 s during the first 100% E-Takatori ground motion excitation.

at the top of column. The computed hysteresis is fairly close to the experimental results although the initial stiffness at smaller response displacement is higher in the analysis than the experiment in particular for the longitudinal direction. This is because uncracked section stiffness is assumed in the analysis at small displacement level.

Fig. 33 shows how the computed strains of an outer longitudinal bar correlate with the measured strains on the SW and NE surfaces at 300 mm and 600 mm from the base. It is important to note in such a correlation that measurement of bar strains over 1% is very difficult. Computed bar strain captures the characteristics of experimental results in particular at the NE surface where the outer longitudinal bar did not buckle.

Figs. 34 and 35 show the same analytical correlation for the response during the second E-Takatori 100% excitation. This analysis was conducted continuously after experiencing the first 100% E-Takatori ground motion excitation so that the initial damage of the column can be taken into account in the analysis for the second 100% E-Takatori ground motion
Fig. 33 Analytical correlation on longitudinal bar strain at 300 mm and 600 mm from the base during the first 100% E-Takatori ground motion excitation.

Fig. 34 Analytical correlation on the response accelerations and displacements for the second E-Takatori 100% excitation.

Fig. 35 Analytical correlation on the moment at the base vs. lateral displacement of the column at the top for the second E-Takatori 100% excitation.
excitation. Because the analysis does not take into account the deterioration of the column stiffness and strength resulting from the spalling-off of covering concrete and local buckling of longitudinal bars, the overall stiffness and strength of the column is overestimated in the analysis. Thus, an analytical model that can predict the response of columns from the post failure region until collapse should be developed.

8. CONCLUSIONS

C1-1 column which is a typical flexural failure type column built in the 1970s was excited twice under 3D 100% E-Takatori ground motion using E-Defense. It was designed assuming 0.23 lateral seismic coefficient and $+/−0.11$ vertical seismic coefficient based on the seismic coefficient method in accordance with the 1964 Japan Road association design code. Based on the results presented herein, the following conclusions may be deduced:

1. C1-1 column which is a typical column in the 1970s suffered extensive damage under a near field ground motion recorded during the 1995 Kobe earthquake.

2. Progress of the damage during the second 100% E-Takatori ground motion was extensive even though it was anticipated before the experiment that damage would not progress unless the intensity of second excitation was much larger than that of the first excitation. This resulted from the extensive deterioration of the lateral confinement due to separation of ties at the lap splices. It is highly possible that columns without sufficient lateral confinement have a similar progress of damage during a long-duration near-field ground motion or strong aftershocks.

3. The lateral confinement of three layered ties is very complex. The lateral confinement is not uniform around the ties. Strains of ties are not similar among the three layers, and they are related to the degree of constraint exerted for preventing local buckling of longitudinal bars. Strains are generally larger in the outer ties than the center and inner ties.

4. Longitudinal and tie bars yielded as high as 83% and 69% the column diameter, respectively, from the base.

5. As has been pointed out from the early days, pull-out of the longitudinal bar inside the footing contributes to the response displacement of the column.

6. Effect of the bilateral excitation was extensive, and it should be included in design. Damage occurred at the side of the peak combined bilateral displacement.

7. During the first E-Takatori 100% excitation, computed response displacements are close to the experimental responses although agreement of the computed accelerations with the measured ones is not sufficiently high. The computed lateral force vs. lateral displacement hystereses is also fairly close to the experimental results although the initial stiffness at smaller response displacement is higher in analysis than the experiment, in particular for the longitudinal direction. This is because uncracked section stiffness is assumed in analysis at small displacement level.

8. The overall stiffness of the column is overestimated in the analysis for the second 100% E-Takatori excitation because the analysis does not take into account the deterioration of the column stiffness and strength resulting from the local spalling-off of covering concrete and local buckling of longitudinal bars. Thus, an analytical model that can predict the response of columns from the post failure region until collapse should be developed.

9. FUTURE DIRECTION

This study about a three-dimensional shake table experiment on a nearly full-size reinforced concrete column is the first in the world. Two excitation programs were conducted in 2008 following this program: 1) a typical column having termination of main reinforcements at mid-heights (C1-2 column) and 2) a typical column which was designed based on the current design practice (C1-5 column).

Reliable experimental data which were free from the scaling-effect inherent to small scaled models were obtained, and it is expected that they can be used worldwide as a benchmark data for analyzing the failure mechanism and verifying analytical models. Because an interval of occurrence of major earthquakes is very long, the effectiveness of countermeasures for mitigating seismic damage of bridges developed in past earthquakes cannot be fully verified until next major event. E-Defense showed its capability for verifying seismic measures without waiting a next major event.

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