E-DEFENSE EXPERIMENT ON THE SEISMIC PERFORMANCE OF A BRIDGE COLUMN BUILT IN 1970S

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ABSTRACT :

This paper presents a large scale shake table experiment on a reinforced concrete bridge column conducted using E-Defense in December 2007. The model was a typical column which was built in 1970s in Japan. Collapse of this type of columns was one of the major causes of the extensive damage during 1995 Kobe, Japan earthquake. Invaluable data on the failure mechanism and response of a RC bridge column was obtained.

KEYWORDS: bridges, seismic response, seismic design, ductility, flexure failure, E-Defense

1. INTRODUCTION

E-Defense was built to advance the scientific knowledge in the 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 were the mechanism of failure?, and what extent do structures fail under near-field ground motions?" are the basic motivations of constriction of E-Defense [Katayama 2005].

The first large-scale shake table experiment using E-defense on bridge structures was conducted in 2007 on a reinforced concrete column which represented a typical column built in 1970s in Japan. Because collapse of this type of columns was one of the major sources of the extensive damage of bridges during 1995 Kobe, Japan earthquake [Kawashima and Unjoh 1997], it was considered important to clarify its failure mechanism based on the original motivation of E-Defense [Nakashima et al 2008]. The seismic design criteria for bridges before 1980 had various deficiencies. The design essentially stood on a static working stress analysis considering 0.2-0.3 seismic coefficient. It was considered at those days that bridges designed based on the traditional static analysis might be safe for responses higher than 0.2-0.3g because various redundancies were included in design. However, such a design concept only prevented and delayed to introduce new research accomplishments on ground motions, nonlinear structural response and the capacity of structural components. An overestimation of the shear capacity of RC columns, insufficient development of ties, insufficient development of main reinforcements when they were terminated at mid-heights, underestimation of seismic force demands for bearings and girders and lack of the capacity design concept were the major problems included in the design of bridges which suffered extensive damage during the 1995 Kobe earthquake.

This paper introduces the first E-Defense experiment on a RC bridge column which represents typical bridge columns in 1970s. This column model is denoted hereinafter as C1-1 column mode.

2. MODEL

C1-1 is a 7.5 m tall 1.8 m diameter circular reinforced concrete column as shown in Fig. 1. It was designed based on a combination of the static lateral force method and the working stress analysis specified in the 1964 Design Specifications of Steel Road Bridges, Japan Road Association [JRA 1964]. Combination of the lateral seismic coefficient of 0.23 and the vertical seismic coefficient of +/-0.11 (upward and downward seismic force) was

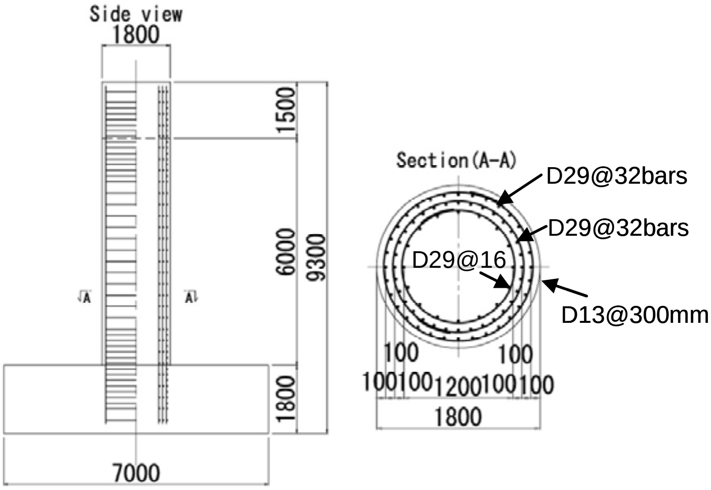


Figure 1 Column built 1960-1970s which fails in Flexure (C1-1 column)

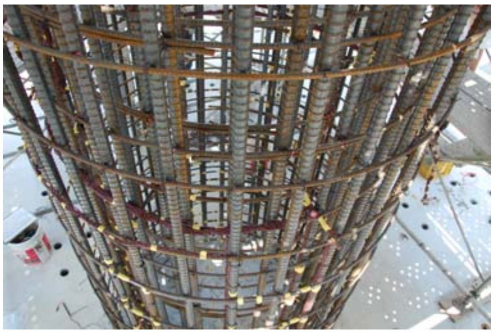


Photo 1 C1-1 under Construction

Photo 2 Reinforcements in Three Layers

assumed in design. It is also assumed that the column is built on the Type II soil (moderate) site. It was nearly a half scale of prototype columns, therefore it was designed assuming that it was a small prototype column without considering the scaling rule. This is because the scaling rule cannot be explicitly implemented to structures in which plastic deformation is predominant.

The column had reinforcements in three layers, i.e., 32, 32 and 16 longitudinal reinforcements were provided at the outer, middle and inner layers as shown in Photos 1 and 2. Longitudinal and tie reinforcements were deformed 29 mm and 13 mm diameter bars, respectively, with the nominal yield strength of 345MPa. Ties were provided at every 300 mm interval, except the outer ties at the top 1.15 m zone and the bottom 0.95 m zone where they were provided at 150 mm interval. Ties were lap spliced. It was the common practice by the mid 1980s because the importance of the lateral confinement was not considered. The longitudinal reinforcement ratio is 2.02% and the tie volumetric reinforcement ratio ρ_s is 0.32% except the top 1.15m and bottom 0.95 m zones where ρ_s is 0.42%. The design strength of concrete was 27 MPa.

Table 1 shows an evaluation of the seismic performance of C1-1 in the longitudinal direction based on the current design code [JRA 2002]. The seismic performance of another column which is designed based on the 2002 design code (denoted here as C1-5 column) is also presented here for comparison. C1-5 is a 2m diameter circular column

Table 1 Evaluation of C1-1 Column in Comparison with C1-5 Column in the Longitudinal Direction

Demand & Capacity	Properties	C1-1	C1-5
Design Force	Design response spectrum S_A	1.75×9.8=17.15m/s^2	
	Force reduction factor	1.58	2.56
	Response acceleration demand	10.83m/s^2	6.70m/s^2
Demand	Displacement demand of the column	0.328m	0.168m
Capacity	Yield displacement capacity u_y	0.046m	0.045m
	Ultimate displacement capacity u_u	0.099m	0.231m
	Design displacement capacity u_d	0.081m	0.169m

with the longitudinal reinforcement ratio of 2.19% and tie volumetric reinforcement ratio of 0.911%. The E-Defense excitation for C1-5 is scheduled in 2008. Because the moderate soil condition (Type II) is considered, the design response acceleration S_A is 17.15 m/s^2 for both C1-1 and C1-5. The yield displacements u_y and ultimate displacement u_u are 0.046m and 0.099m in C1-1 and 0.045m and 0.231m in C1-5. The design displacement u_d is evaluated from u_u and u_y as

$$u_d = u_y + \frac{u_u - u_y}{\alpha u_y} \tag{1}$$

in which α depends on the type of ground motions (near-field or middle-field) and the importance of the bridge. Because α is 1.5 for a combination of the near-field ground motion category and the important bridges category, the design displacement u_d is 0.081m in C1-1 and 0.169m in C1-5.

On the other hand, because the force reduction factor is 1.58 in C1-1 and 2.56 in C1-5, the displacement demand of the column u is 0.328m in C1-1 and 0.168m in C1-5. Consequently, C1-1 is evaluated to be unsafe while C1-5 is safe. From the evaluation of the shear capacity, both fails in flexure.

3. EXPERIMENTAL SETUP

The C1-1 column was set on E-Defense as shown in Fig. 2 and Photo 3. Two simply supported decks were set on the column and two steel end supports. The decks are a device to fix four mass blocks on the column and they are not designed to idealize 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 Photos 4 and 5. Two side sliders (friction bearings) were provided at the both sides of the fixed and movable bearings for preventing overturning of the decks around its axis.

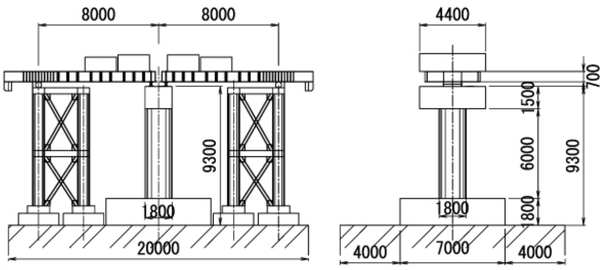


Figure 2 Setup of Model

A 78 t mass block and a 44.6 mass block per deck are fixed to the decks. The mass blocks are of laminated steel plates. Total mass due to 4 mass blocks, 2 decks, 2 fixed bearings, 2 movable bearings, 8 side sliders and 32 load


(a) NNW view


(b) NE view

Photo 3 C1-1 column on E-Defense


Photo 4 Fixed bearing and 2 Side Sliders on the Column


Photo 5 Movable Bearing and 2 Side Sliders on an End Support

cells is 302 t. Including masses of the column with a 1.8 m thick footing, 2 steel end supports and table protections, the total mass of the model is slightly over 1,000 t.

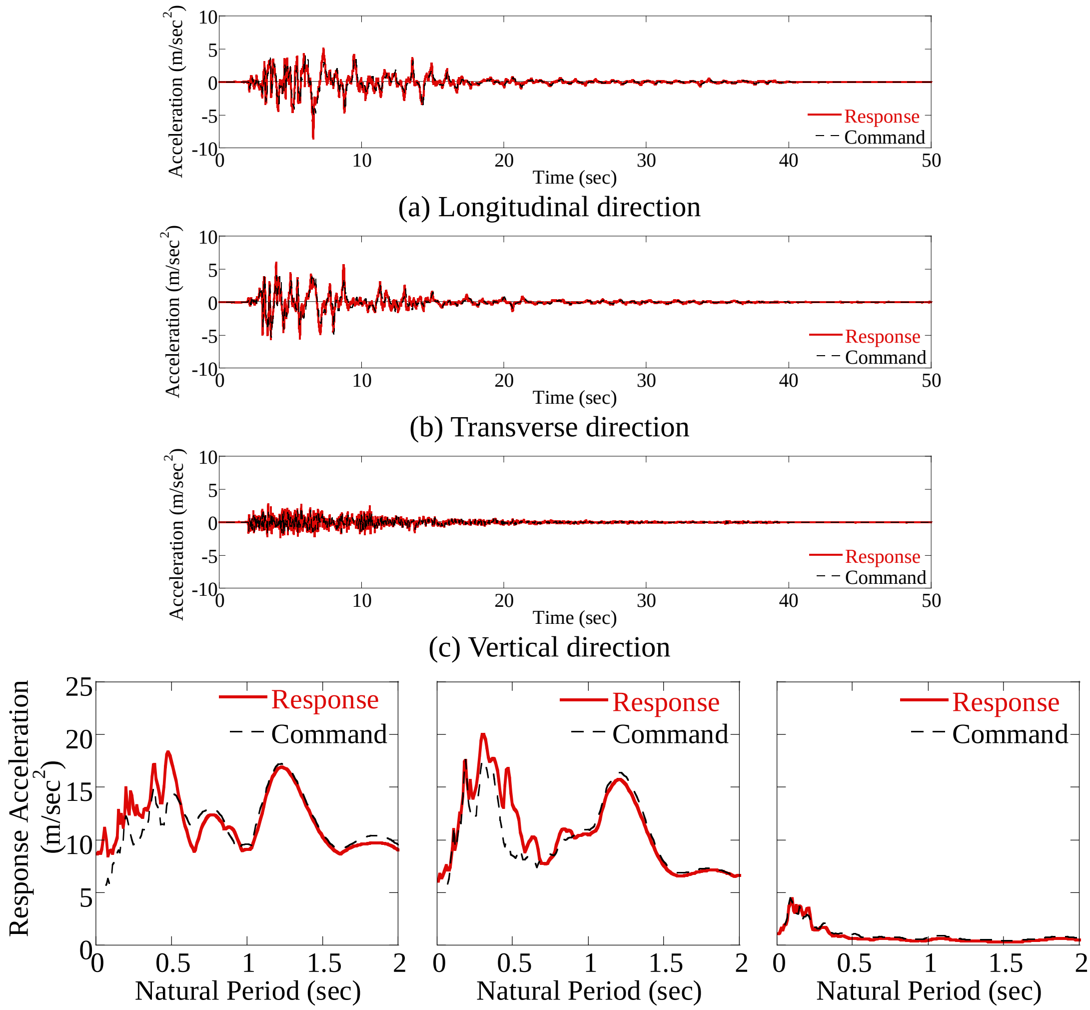
A ground acceleration recorded at JR Takatori Station during the 1995 Kobe, Japan earthquake (refer to Fig. 3) was used for the table motion. It is well known that the radiational energy dissipation of a model on a shake table is extremely smaller that the real energy dissipation. Taking account of the soil structure interaction effect, a ground motion with 80% the original intensity of JR Takatori record was used as a command to the table in the experiment. This ground motion is called hereinafter as E-Takatori ground motion.

The model was subjected to a 10%, two 20% and six 30% E-Takatori ground motion excitations to check the response and measurement. No visible cracks occurred during those excitations. Main excitation using 100% E-Takatori ground motion was conducted twice. Only the response and failure mode for the first 100% E-Takatori excitation are shown in this paper.

4. SEISMIC PERFORMANCE OF THE C1-1 COLUMN

Fig. 4 shows the response displacements at the top of the column. The combined displacement of two lateral components had a peak of 0.195 m (2.56 % drift) at 6.9 s. Because the ultimate displacement at the top of the column is 0.091 m, the peak response exceeded the ultimate displacement by a factor of 2.1.

Photo 6 shows progress of failure of the column at the plastic hinge on NE and SW surfaces. It should be noted that N-S and E-W correspond to the longitudinal and transverse directions of the model, respectively. At an



(a) Longitudinal direction
(b) Transverse direction
(c) Vertical direction
Longitudinal direction Transverse direction Vertical direction
(d) Response accelerations

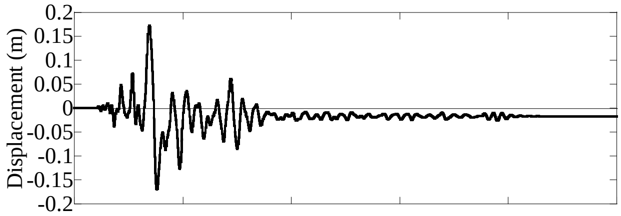
Figure 3 E-JR Takatori Ground Accelerations

instance of 6.9 s, the covering concrete started to spall at SW surface due to compression. At SW, covering concrete spalled between the bottom and 0.6 m from the bottom of the column and several outer longitudinal reinforcements locally buckled between the bottom and 0.2 m from the bottom of the column during the excitation.

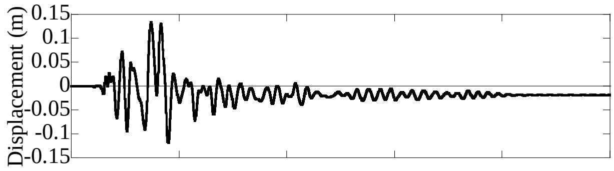
Figs. 5 and 6 show strains of the outer longitudinal and tie reinforcements, respectively, at 6.9 s [JSCE 2008]. Strains in the longitudinal reinforcements were over 4000 μ in tension at SE, E, NE, N and NW while they were over 2000 μ in compression at W and SW. Strains in the longitudinal reinforcements are extremely large between 0.25 m below and 1.5 m above the surface of the footing. It is interesting to note that extensive yielding of longitudinal reinforcements occurs at the zone higher than the anticipated plastic hinge region. Deformation of the longitudinal reinforcements inside the footing contributes to the lateral response of the model.

Although it is not presented here, interaction of three layered longitudinal reinforcements is complex. Similarly, the lateral confinement among the three layered ties is very complex. The lateral confinement is not uniform around the ties, and it is not the same among the three ties. Mechanism of the lateral confinement by multi-layered ties should be critically investigated.

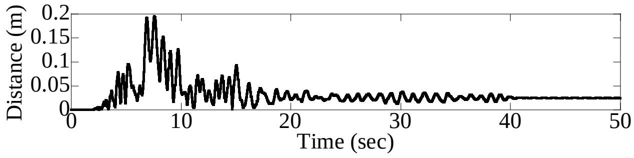
The strains of the tie reinforcements at an instance of 6.9 s are larger at 0.35 m and 0.65 m above the surface of the



(a) Longitudinal



(b) Transverse



(c) Combined bilateral components

Figure 4 Response Displacement of C1-1 at the Top during the First 100% E-Takatori Ground Acceleration Excitation

(a) 6.9 sec

(b) 8.35 sec

Photo 6 Progress of Failure of C1-1 during the First 100% E-Takatori Ground Acceleration Excitation

footing. The maximum strains at the two locations are nearly 2000 μ, slightly larger than the yield strain. It is important to note that strains of tie reinforcements are larger at SW and W where the section is subjected to compression. Obviously this is resulted from the local buckling of longitudinal reinforcements at SW.

5. CONCLUSIONS

C1-1 column which is a typical flexural failure type column in 1970s was excited twice under 3D 100% E-Takatori ground accelerations 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 JRA 1964 design codes. Preliminary findings from the E-Defense experiment are summarized as follows:

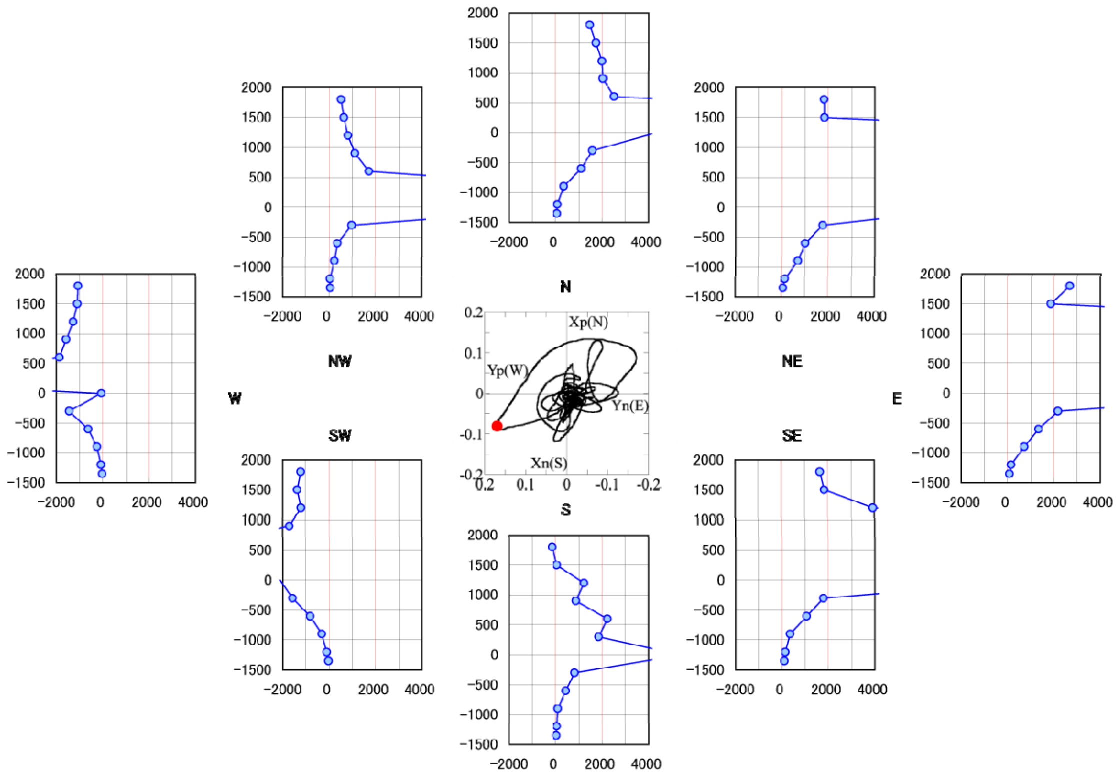


Figure 5 Strains of Outer Longitudinal Bars at 6.9 sec during the First 100% E-Takatori Ground Acceleration Excitation

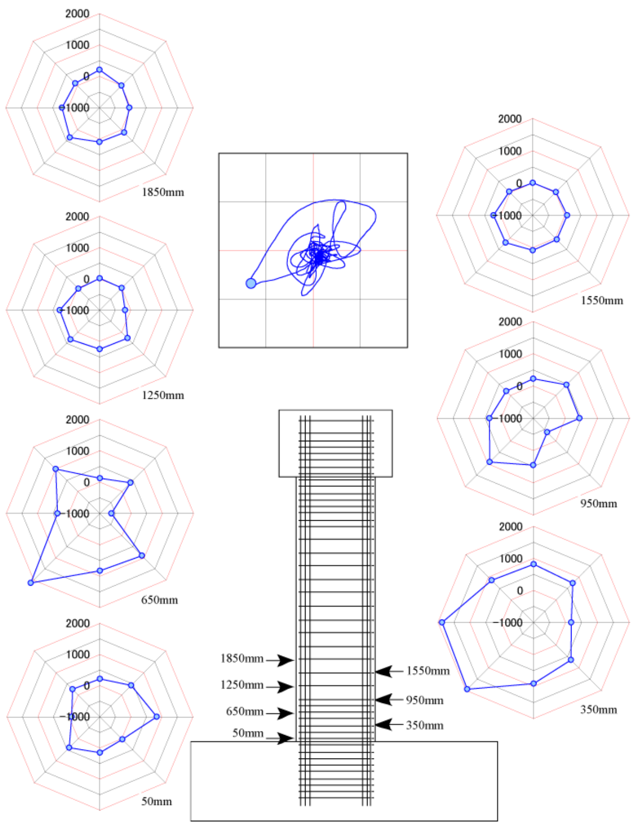


Figure 6 Strains of Ties at 6.9 sec during the First 100% E-Takatori Ground Acceleration Excitation

1. C1-1 column suffered extensive damage under a near field ground motion recorded at JR Takatori Station during the 1995 Kobe earthquake.
2. The lateral confinement of three layered ties is very complex. The lateral confinement is not uniform around the ties, and it is not the same among the three ties. Mechanism of the lateral confinement by multi-layered ties should be critically investigated.
3. Although it was anticipated that yielding of the longitudinal and tie reinforcements was less significant at the zone higher than the plastic hinge, extensive yielding occurred up to 83% and 69% the column diameter in the longitudinal and tie reinforcements, respectively. The deformation of both longitudinal and tie reinforcements should be investigated.
4. Deformation of the longitudinal reinforcement inside the footing contributes to the response of the column. The mechanism and its effect should be investigated.
5. Effect of the bilateral excitation should be included in design. The current design still stands on the concept of unilateral excitation.

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