第10回地震時保有耐力法に基づく橋梁等構造の
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Seismic Response Simulation of E-Defense C1 Model designed based on 2002 JRA Design Codes

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1. INTRODUCTION

E-Defense is the world largest shake table facility in Miki, Japan constructed by the National Research Institute for Earth Science and Disaster Prevention (NEID)[1]. Since it was built to clarify the extensive damage of structures in the 1995 Kobe earthquake, a series of breakthrough tests using full/large-scale models are expected as a benchmark test. The bridge program has been formulated by planning meeting among Japanese and US researchers[2]. Based on these discussions, component model (C1 model) and system model (C2 model) are proposed. C1 model is proposed for clarifying the failure mechanism of reinforced concrete bridge piers using as large models as possible to eliminate barriers of scaling and loading rate effects. On the other hand, C2 model is to study the complex system behavior of bridges to failure.

Currently C1 models as well as its setup are being designed. However, because it is still preliminary stage, further discussions among researchers will be needed to refine the details of the model, the setup, and the test cases. Thus to have a primary simulation on the C1 model project which is proposed in current stage, a dynamic response analysis is conducted on the C1 model which is designed based on the current seismic design codes[3] including its setup.

2. C1 MODEL AND ITS SETUP

(1) C1 Model based on 2002 JRA[3]

Configuration of the column proposed as a C1 model which is designed in accordance with the current seismic design criteria[3] is shown in Fig. 1. It is 7.5 m tall and has a circular section with a diameter of 2 m. It is supported by 2 m thick footing. Shear span ratio is 3.75. Deformed bars with diameters of 35 mm and 22 mm are used for longitudinal and tie reinforcements, respectively. The longitudinal bars are set in 2 lines; 36 longitudinal bars for both outer and inner bars. Longitudinal reinforcement ratio is 2.2 %. Tie bars are provided at 150 mm and 300 mm intervals to the outer and inner longitudinal bars, respectively, for the entire column height. The volumetric tie reinforcement ratio is 0.91 % based on the current design codes[3]. The axial stress at the plastic hinge region of the column model is 1.06 MPa. The grade of the reinforcements for both longitudinal and tie bars is SD345 with 345 MPa nominal yield strength. Design concrete strength is 27 MPa.

The column is designed under Type-I (middle field) and Type-II (near field) ground motions at a site corresponding to the moderate ground condition (Type-II Ground Condition) based on the current design codes[3]. Seismic performance of the C1 model which is estimated based on the current codes

Table 1 Seismic Performance of C1 Model based on 2002 JRA

	Lateral Displacement (m)	Lateral Force (kN)	Ductility
Initial Yield	0.0266	1702	–
Yield	0.0393	2514	–
Allowable	0.1540	2514	3.91
Ultimate	0.2113	2514	5.37

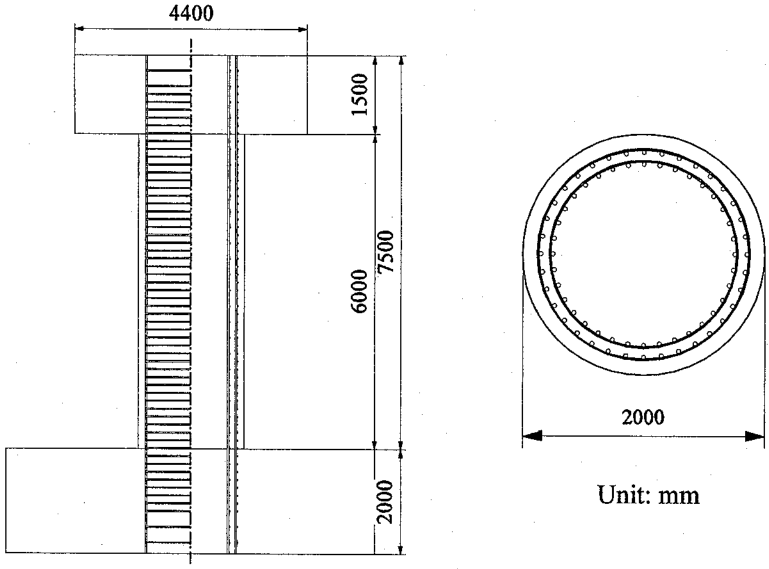
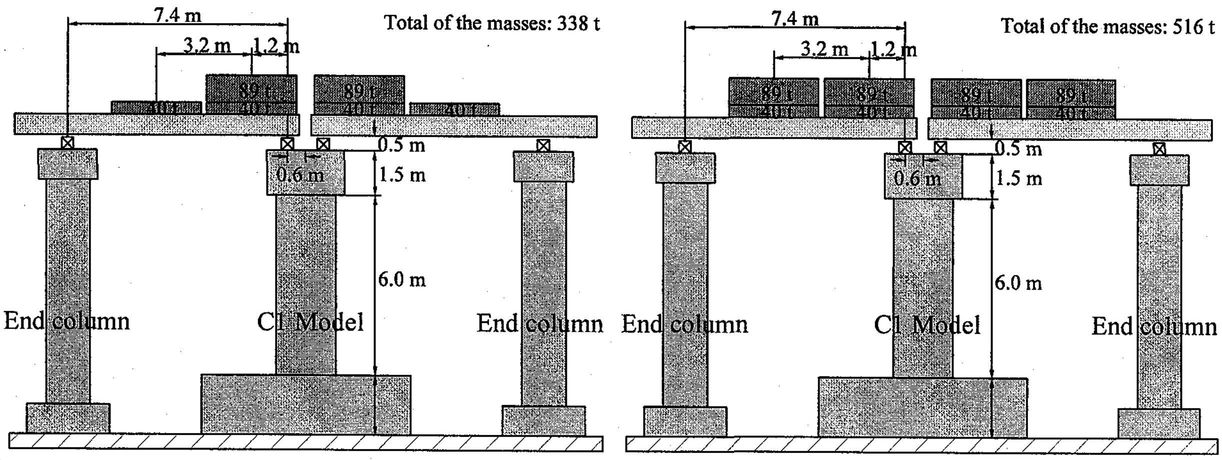


Fig. 1 Configuration of the C1 Model designed based on JRA 2002



(a) Yield and Design Level (b) Maximum Level

Fig. 2 Setups of the C1 Model

is summarized in Table 1. The lateral strength of the C1 model is 2.51 MN. The yield and ultimate displacements are 0.039 m and 0.211m, respectively.

(2) Setup of the C1 Model

Setups of the C1 model which are currently proposed are shown in Fig. 2. For a C1 model which satisfy the current seismic design requirements, it is proposed to conduct a series of tests by varying the seismic force level such as; a) the yield level test, b) the design level test, and c) the maximum level test. Depending on the excitations applied, the deck

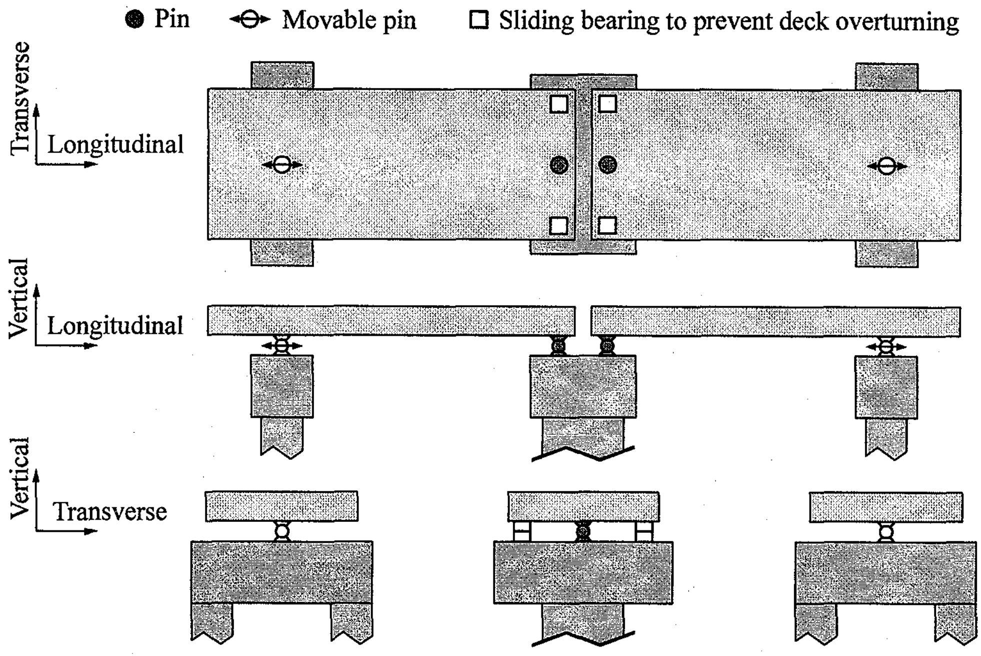
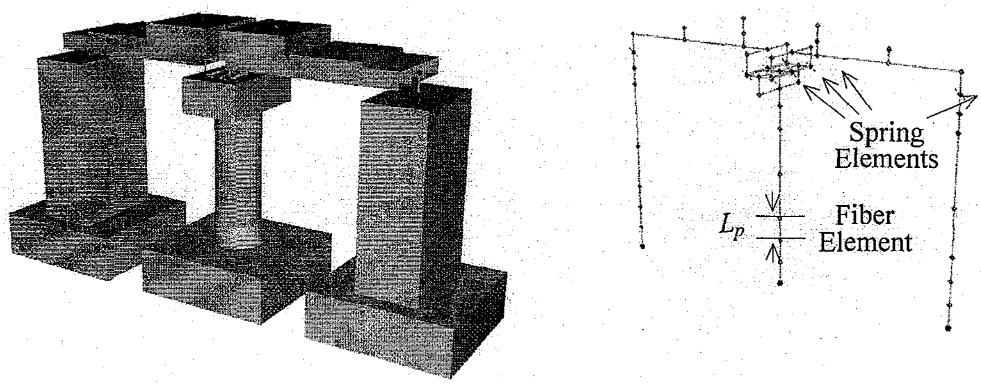


Fig. 3 Bearing Conditions of the C1 Model



(a) Target Structure (b) Analytical Idealization

Fig. 4 Analytical Idealization of the C1 Model and its Setup

weight will be changed as shown in Fig. 2. Total weight of the decks is 338 tf in the yield level and design level tests as assumed in the design procedures on the C1 model. On the other hand, it is assumed to be 516 tf which is corresponding to approximately 150 % of the original weight of the design level setup. The purpose of the maximum level test is to clarify the seismic behavior to collapse of reinforced concrete columns which satisfy the current design requirements under the further extensive ground motion,

Fig. 3 shows the supporting condition of the decks by bearings. Two simple decks each are pin connected by the C1 model. The decks are supported by two steel columns with high stiffness and strength at both ends using movable bearings. When inertia force of the two decks applies to the C1 model in the longitudinal direction, a certain friction force develops at the movable bearings but it may be limited. However, because two end columns contribute to support the inertia force of two decks through the movable bearings (movable in the longitudinal direction, but pin connected in the transverse direction) in the transverse direction, the weights on the two decks are set as close to the C1model as possible. On the top of the C1 model a set of sliding bearings are planned to set to prevent overturning of the decks as shown in Fig. 3. Since

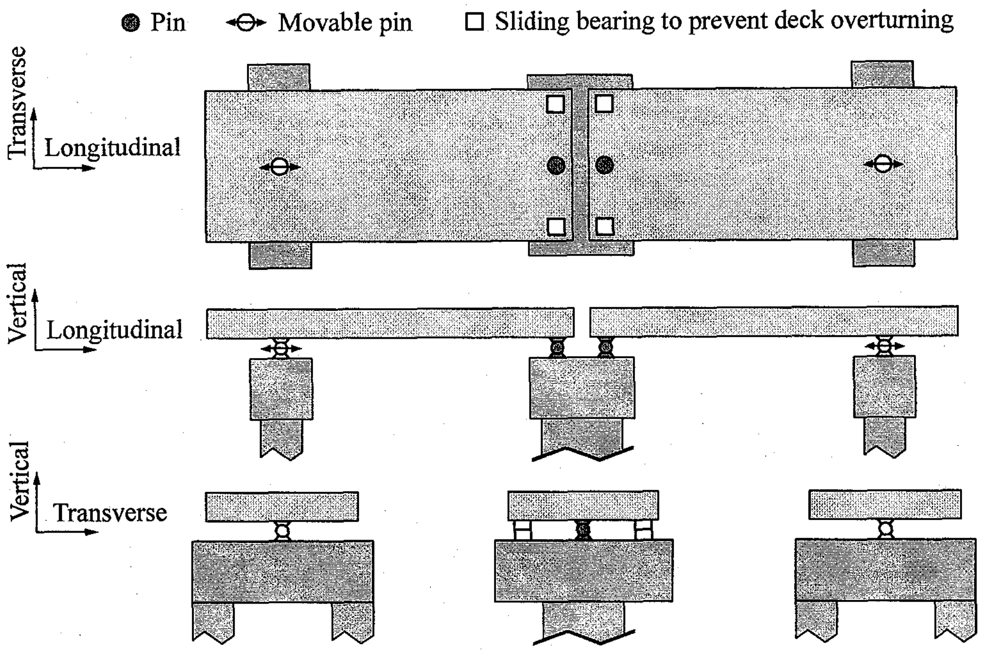
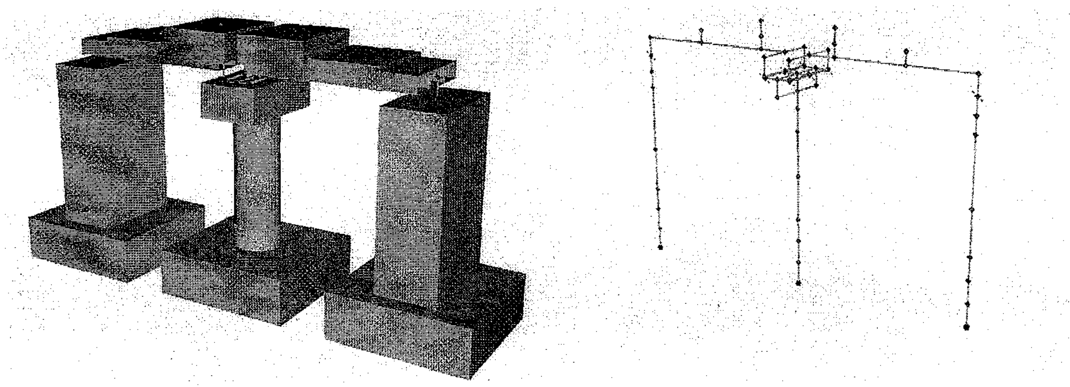


Fig. 3 Bearing Conditions of the C1 Model



(a) Target Structure (b) Analytical Idealization

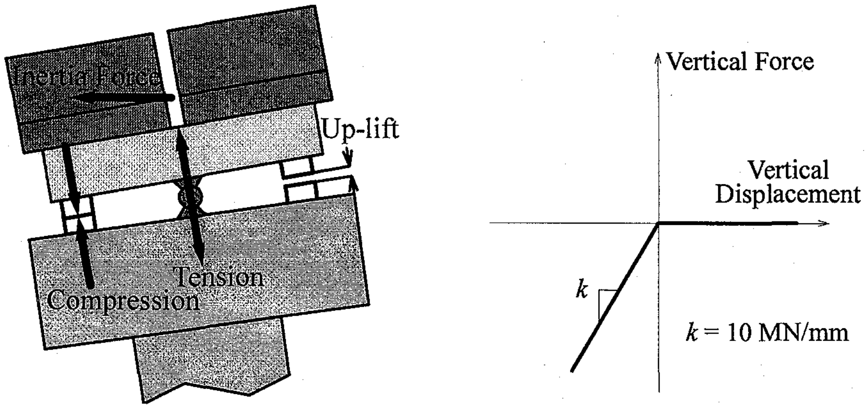
Fig. 4 Analytical Idealization of the C1 Model and its Setup

these sliding bearings do not resist tension in the vertical direction, it is noted that up-lift of the sliding bearings can slightly occurs due to inertia force in the transverse direction as will be descried later.

3. ANALYTICAL IDEALIZATION

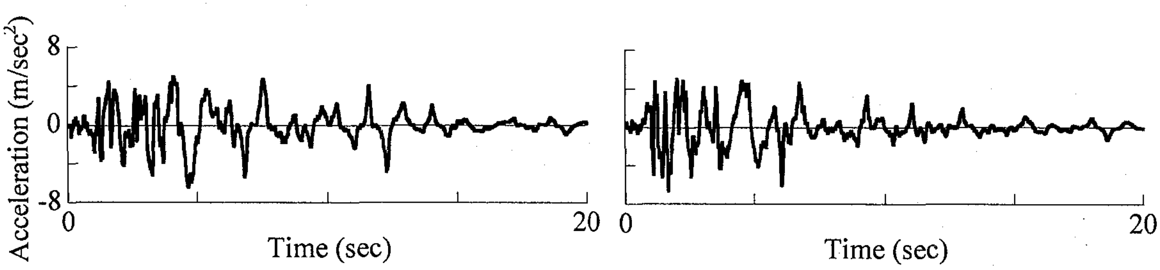
The C1 model as well as its setup including the decks, the end columns, and bearings are idealized by finite elements as shown Fig. 4. Because end columns have not been yet designed in this stage, concrete columns with a 4.4 m by 2.6 m rectangular section are assumed to be used. To represent nonlinear flexural behavior, a fiber element is used in the plastic hinge region of the C1 model, assuming that the plastic hinge length L_p is 1 m which is corresponding to a half of the column width[3]. In the fiber element, to simulate hysteretic behavior of cover and core concrete, Hoshikuma *et al.* model[4] is used. For reinforcements, a bilinear model which takes into account Bauschinger effect is used. The column body other than the plastic hinge region of the C1 model, decks, and end columns are idealized by linear beam elements.

To represent the bearing condition shown in Fig. 3, spring elements are employed. As described

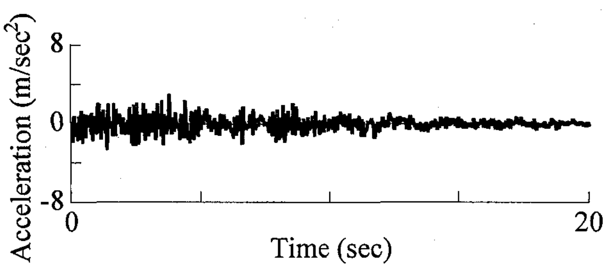


(a) Up-lift of the Sliding Bearings (b) Hysteresis assumed for the Sliding Bearings

Fig. 5 Idealization of Up-lift of the Sliding Bearing



(a) NS Component (b) EW Component



(c) UD Component

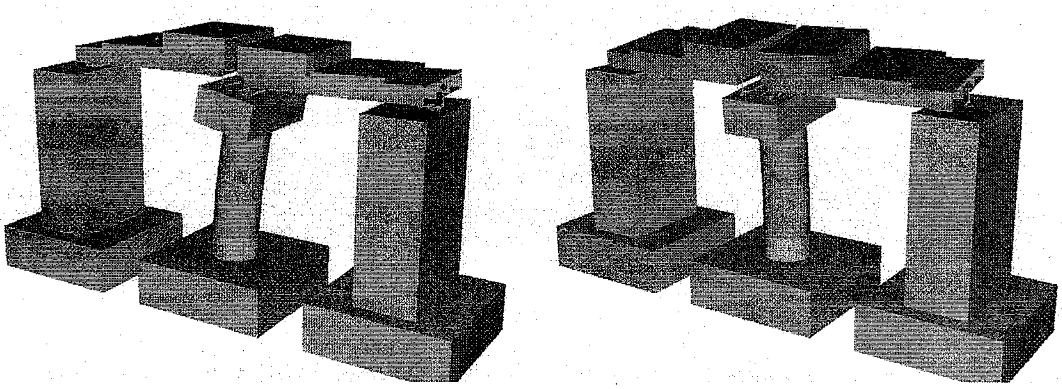
Fig. 6 Input Ground Acceleration (JR Takatori Ground Motion)

earlier, because the sliding bearings do not resist tension in the vertical direction, up-lift of the sliding bearings possibly occurs due to inertia force in the transverse direction as shown in Fig. 5(a). To idealize this effect, a nonlinear spring element which does not resist tension is applied for the sliding bearings in the vertical direction as shown in Fig. 5(b).

Ground acceleration record observed in JR Takatori station during 1995 Kobe earthquake is used as an input ground motion as shown in Fig. 6. NS, EW, and UD components are simultaneously applied to the longitudinal, transverse, and vertical directions, respectively. The intensity of ground motion is scaled down to 25 % of the original record in the yield level simulation, while the original record is used in the design level and maximum level simulations.

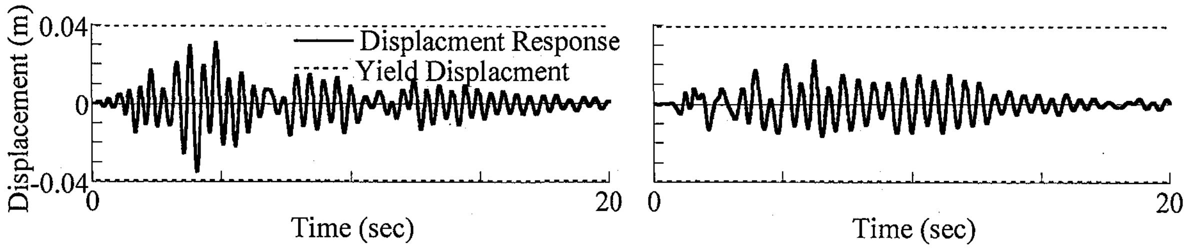
To represent accumulation of damage due to a series of the excitations, all the level excitations should be imposed to an analytical model continuously. However, only the yield level and design level simulations are conducted continuously. This is because in the maximum level simulation the analytical model should be changed by putting additional weights.

The time history analysis is carried out by using the Newmark β method. The constant acceleration is assumed in each step of the numerical integration and the time interval of the



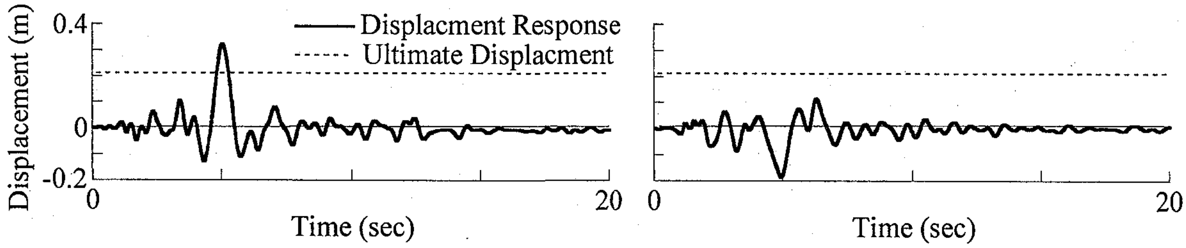
(a) Longitudinal Direction (0.50 sec) (b) Transverse Direction (0.59 sec)

Fig. 7 Fundamental Mode Shapes of the C1 Model under the Design Level Setup



(a) Longitudinal Direction (NS) (b) Transverse Direction (EW)

Fig. 8 Displacement Response at the top of C1 Model under the Yield Level Excitation



(a) Longitudinal Direction (NS) (b) Transverse Direction (EW)

Fig. 9 Displacement Response at the top of C1 Model under the Design Level Excitation

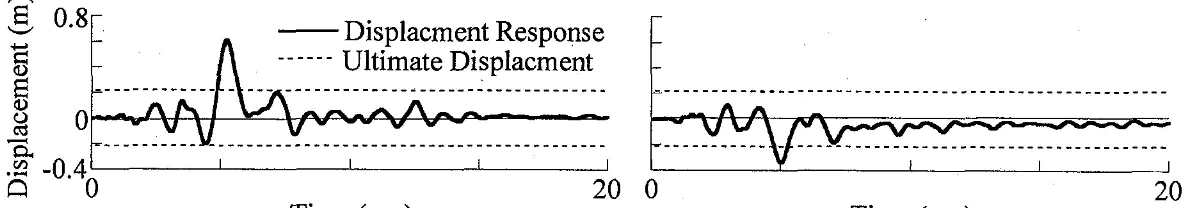
integration is 0.005 sec. Rayleigh damping is applied, assuming that damping ratio is set to 2%.

4. THE YIELD LEVEL AND DESIGN LEVEL SIMULATIONS

Based on the analytical idealization described above, a modal analysis is conducted on the C1 model and its setup for the yield and design level tests. Fig. 7 shows the natural mode shapes of the C1 model including the setup. The fundamental periods are 0.5 sec and 0.59 sec in the longitudinal and transverse directions, respectively.

Displacement response at the top of the C1 model under the yield level excitation is shown in Fig. 8. The maximum displacement response is -0.035 m (-0.46 % drift) and 0.022 m (0.29 % drift) in the longitudinal and transverse directions respectively, while the nominal yield displacement is 0.039 m (0.52 % drift). The 25 % amplitude of the original motion may be good to obtain yield level response of the C1 model.

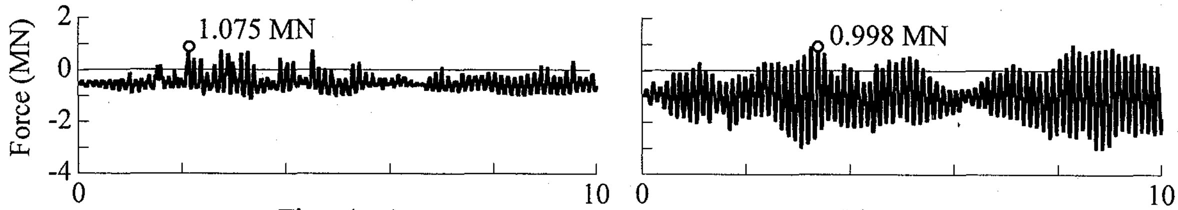
Fig. 9 shows displacement response the C1 model in the design level simulation. The maximum response is 0.32 m (4.2 % drift) and -0.19 m (-2.5 % drift) in the longitudinal and transverse directions, respectively. On the other hand, the ultimate displacement estimated based on the current design codes is 0.21 m (2.8 % drift). The maximum



Time (sec) Time (sec)

(a) Longitudinal Direction (NS) (b) Transverse Direction (EW)

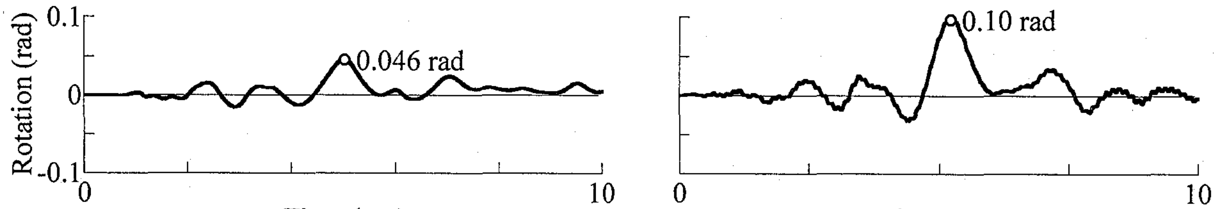
Fig. 10 Displacement Response at the top of C1 Model under the Maximum Level Excitation



Time (sec) Time (sec)

(a) Pin Connection on the C1 Model (b) Pin Connection on the End Column

Fig. 11 Vertical Forces of the Pin Connections under the Maximum Level Excitation



Time (sec) Time (sec)

(a) About the Vertical Direction (b) About the Transverse Direction

Fig. 12 Rotation of the Pin Connection on the C1 Model under the Maximum Level Excitation

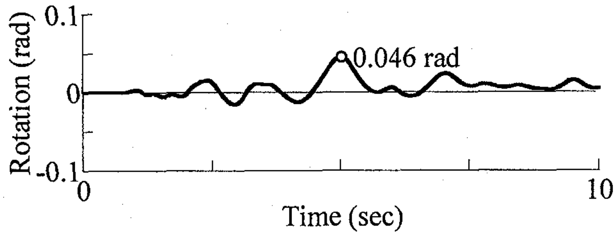
response is 52 % larger than the ultimate displacement.

5. THE MAXIMUM LEVEL SIMULATION

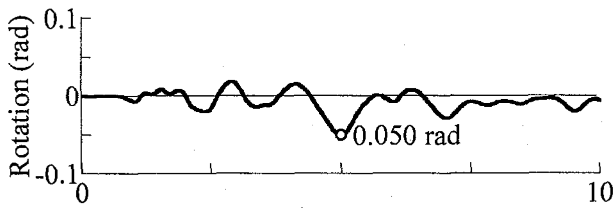
A modal analysis is conducted on the C1 model and its setup for maximum level test (see Fig. 2(b)). Although the fundamental mode shapes under the this setup is similar to those under the design level setup (see Fig. 7), the natural periods are 0.61 sec and 0.67 sec in the longitudinal and transverse directions, respectively, which are longer than those under the design level setup due to the additional weights.

Fig. 10 shows the displacement response of the C1 model under the maximum level excitation. The maximum displacement is 0.61 m (8.1 % drift) and -0.34 m (-4.5 % drift) in the longitudinal and transverse directions respectively, while the ultimate displacement is 0.21 m (2.8 % drift). The maximum displacement is 190 % larger than the ultimate displacement. Because the purpose of the maximum level test is to clarify a failure mode of the C1 model designed based on the current design codes under the further extensive excitation, 150 % of the original weight of the decks may be enough to satisfy this purpose.

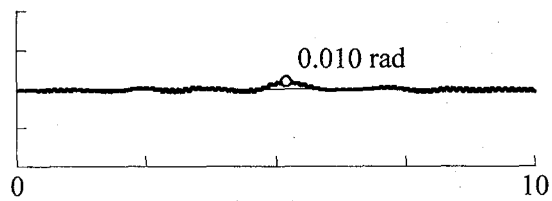
As described above, up-lift of the sliding bearings occurs due to the inertia force in the transverse direction as presented in Fig. 5(a). The maximum separation of the sliding bearings under the maximum level excitation is 0.36 mm. Because the sliding bearings do not resist the tension in the vertical direction, extensive tensile force develops in the pin connection. Fig. 11 shows the vertical force of the pin connections. The maximum tensile force is 1.1 MN and 1.0 MN at the pin connections on the C1 model and the end column, respectively. It is necessary that the pin connections can resist such a tensile force to avoid undesirable failure of the bearings. To determine the rotation capacities of the



(a) About the Vertical Direction



(b) About the Longitudinal Direction



(c) About the Transverse Direction

Fig. 13 Rotation of the Pin Connection on the End Column under the Maximum Level Excitation

pin connections, it is important to clarify the rotation demands by this analysis under the maximum level excitation. The maximum rotation developed in the pin connection on the C1 column is 0.046 rad and 0.10 rad about the vertical and transverse directions, respectively, while that in the pin on the end column is 0.046 rad, -0.05 rad, and 0.010 rad about the vertical, longitudinal, and transverse directions respectively.

6. CONCLUSIONS

1) To simulate seismic response of C1 model designed in accordance with the current codes and its setup, response analysis was conducted under yield, design, and maximum level excitations.
2) 25 % amplitude of the original record of JR Takatori ground motion may be good to obtain yield level response of C1 model.
3) The maximum displacement under design level excitation is 52 % larger than ultimate displacement estimated based on the current codes, while that under maximum level excitation is 190 % larger than ultimate displacement.
4) Because sliding bearings do not resist tension, tensile force develops in pins. To evaluate rotation capacities of pins, rotation demands are summarized.

7. ACKNOWLEDGEMENTS

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REFERENCES

1) Katayama, T.: Construction of the E-Defense a Large-sized 3-Dimensional Shake Table, *Proc. the 1st International Conference on Advances in Experimental Structural Engineering*, Vol. 1, pp. 29-38, Nagoya, Japan, 2005.
2) Kawashima, K., Ukon, H. and Kajiwara, K.: Large-scale Shake Table Test on Bridges using E-Defense based on NEES and E-Defense Collaboration, *The 4th International Workshop on Sesmic Design and Retrofit of Transportation Facilities*, Session 1, pp.1-9 (CD-ROM), San Francisco, California, USA, 2006.
3) Japan Road Association: *Part V seismic design, design specifications of highway bridges*, Maruzen, Tokyo, Japan, 2002.
4) Hoshikuma, J., Kawashima, K., Nagaya, K. and Taylor, A. W.: Stress-Strain Model for Confined Reinforced Concrete in Bridge Piers, *J. Structural Engineering*, ASCE, 123(5), pp. 624-633.1997.