Shake Table Experiment on Circular Reinforced Concrete Bridge Column under Multidirectional Seismic Excitation

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INTRODUCTION

Since the 1971 San Fernando, CA, earthquake, a significant amount of research has been conducted on the ductility capacity of reinforced concrete bridge columns, resulting in significant advances in the seismic design of bridges. Since most tests have been done in static or quasistatic conditions and unidirectional loading conditions, however, no method that properly evaluates the effect of multidirectional dynamic loading has been developed, and design recommendations on this effect are still limited. It is essential to evaluate the effect of multidirectional loading on the dynamic response and seismic performance of reinforced concrete bridge columns in order to develop more advanced and reliable design procedures.

Research on the dynamic behavior of reinforced concrete bridge columns under multidirectional loading is still limited due to the limitation of capacity of research facilities. Mahin and Hachem [1999] conducted a series of shake table tests of circular reinforced concrete bridge columns. The effect of multidirectional loading was investigated for 406 mm-circular columns. Nishida and Unjoh [2001, 2004] conducted a series of shake table tests for three types of cross sections, circular (600 mm in diameter), square (600×600 mm) and rectangular (450×800 mm), under a near field ground motion, and then conducted dynamic analyses to investigate the effect of bilateral loading. Sakai and Mahin [2004] conducted shake table tests of a circular reinforced concrete bridge column as a part of a research project to develop a new method that mitigates post-earthquake residual displacements.

Although these tests provide valuable data for development of advanced analytical procedures and findings to be considered in seismic design on the multidirectional dynamic loading effects, most columns were tested under a near field ground motion, which has a few dominant pulses, and the dynamic behavior under repetition of strong pulses has not well been investigated.

In the study presented here, the dynamic response of a circular reinforced concrete bridge column specimen under a repetitive strong shaking is investigated through shaking table tests. Accuracy of an analytical model using a fiber element is evaluated with the test results, and then the effect of multidirectional seismic excitation is analytically investigated.
SPECIMEN AND INSTRUMENTATION

Figure 1 shows the specimen tested in this study. An analytical model described later is also shown in the figure. The specimen has the same configuration tested by Nishida and Unjoh [2004]. This is a 1/4-scale model. The column diameter is 600 mm, and the height from the bottom of the column to the center of gravity of the top mass is 3 m, resulting in an effective aspect ratio of 5. The sides face the X positive and the X negative directions are defined as the Xp and Xn faces, respectively, as shown in Figure 1. Likewise, the Yp and Yn faces were defined.

The specimen supports steel plates that idealize the inertia mass and dead load from a superstructure as shown in Figures 1 and 2. The inertia mass is 27,000 kg and the axial force induced at the bottom of the column is 280 kN, which results in 0.99 MPa in axial stress at the bottom.

The specimen is reinforced longitudinally with 40 of 10-mm diameter deformed bars (D10), providing a longitudinal reinforcement ratio, $\rho_l$, of 1.01%. Hoop reinforcement of 6-mm diameter deformed bars (D6) is used to confine the concrete core, spaced at a 75-mm pitch, resulting in a volumetric ratio, $\rho_s$, of 0.31%. SD295A bars are used for both the longitudinal and transverse reinforcement. The D10 and D6 bars have yield strengths of 351 MPa and 340 MPa, and ultimate strengths of 496 MPa and 514 MPa. The cylinder strength of concrete was 41.7 MPa on the day of the shake table test.

The yield and ultimate displacements of the specimen based on the JRA design specification [Japan Road Association, 2002] are 0.016 m and 0.055 m, respectively, resulting in an ultimate ductility of 3.51. The computed flexural strength is 100 kN.

Response accelerations and displacements in three principal directions (X, Y and Z) were measured at the center of gravity of the top mass. Response accelerations on the shake table were also measured. CCD laser displacement sensors and accelerometers were used to measure displacements and accelerations, respectively. The range of the...
laser sensor in the X direction was $\pm 0.2$ m while that in the Y direction was $\pm 0.25$ m. Displacement transducers were placed on both ends of bars that penetrated the column around a potential plastic hinge region to measure response curvature of the column. The penetrated bars were provided at 0.03, 0.11, 0.27, 0.43, and 0.61 m from the bottom at the faces in the X direction, and 0.008 m above at all the levels in the Y direction. Strain gauges were placed on the longitudinal rebar at -0.13, 0.02, 0.17, 0.32, 0.47, 0.62, 0.77 and 0.92 m from the bottom, and 8 strain gauges were placed at each 45 degrees at each height. The sampling frequency was 200 Hz for measurements of accelerations, displacements and strains.

Five camcorders were used to capture global behavior from the X positive direction, and local behavior at the four faces. The movies provided valuable information of sequence of local damage.

**GROUND MOTION USED**

**Tests Conducted at PWRI and Calibration of Analytical Model**

To select input ground motions of this study, a series of dynamic analyses was conducted for 10 strong motion records. The analytical model was first calibrated by the results from two tests conducted at the Public Works Research Institute (PWRI). The one was tested quasistatically [Unjoh et al. 2000], and the other was tested on the shake table under the two horizontal components of 80% scaled motions of the JR Takatori station records of the 1995 Hyogo-ken Nanbu, Japan, earthquake [Nishida and Unjoh, 2004]. The specimens had the same configuration, reinforcement details, design concrete strength and yield strength of rebar with the specimen of this study; however, the actual material properties were slightly different.

Figure 3 shows the lateral force versus lateral displacement hystereses obtained from the quasistatic loading test. The hysteresis predicted by the analysis is also shown in the figure. The spalling of cover concrete was first observed at the displacement of 0.08 m, and then, the buckling of longitudinal reinforcement was observed at 0.1 m, resulting in reduction of the lateral force capacity.

Figure 4 shows the orbit of response displacement at the center of gravity of the top mass from the shake table test. The hysteresis predicted by the analysis is also shown in the figure. A few strong pulses dominated the response, and large response occurred only in the 4th region. The maximum displacement in the X and Y directions were 0.109 m and 0.151 m, respectively. The maximum displacement expressed as a distance from the origin, $d_t$, defined in (1), was 0.174 m.

$$d_t = \sqrt{d_{X,t}^2 + d_{Y,t}^2}$$

where $d_{X,t}$ and $d_{Y,t}$ are the response displacements in the X and Y directions at time $t$, respectively. After the excitation, the cover concrete was spalled at the bottom of the column and 15 of 40 longitudinal reinforcing bars were buckled.

The analytical model was calibrated with these results. The specimen was idealized as a three-dimensional discrete model as shown in Figure 1. The nonlinear hysteretic behavior of the plastic hinge region was idealized with a fiber element. The plastic hinge length was assumed to be the radius of the cross section (= 0.3 m). Linear elements with
uncracked stiffness properties were used to model the footing and the element from the column top to the gravity center of the top mass. Linear beam elements with cracked stiffness properties were used for the remainder of the column.

The envelope curve of the confined concrete was idealized with the model proposed by Hoshikuma et al. [1997], and the unloading and reloading paths were idealized by the model proposed by Sakai and Kawashima [2006]. The envelope curve of the reinforcing steel was idealized as a bilinear model with the strain-hardening ratio equal to 2%. To represent the hysteretic behavior of the rebar, the modified Menegotto-Pinto model proposed by Sakai and Kawashima [2003] was used. Buckling or fracture of rebar was not considered in this study.

The quasistatic analysis was conducted up to the rebar buckling for the quasistatically loaded specimen as shown in Figure 3. The analysis shows good agreement with the test result including unloading and reloading paths, which ensures that the model used in this study has capability to simulate with sufficient accuracy the hysteretic behavior of the reinforced concrete bridge column specimen of this study.

A parametric study was conducted for the dynamically loaded specimen to explore appropriate damping assumptions. The results shows that no viscous damping and Rayleigh damping using the 1st and 5th modes with a damping ratio, $\xi$, of 2% results in sufficient accuracy of the predicted response. No damping results in a 10% larger response, while the Rayleigh damping results in a 12% smaller response than the test results as shown in Figure 4. Since no damping sometimes causes early termination of the analysis due to numerical instability, the above model and the Rayleigh damping with $\xi$ of 2% were used for the analyses determining the input signals.

![FIGURE 3](image1)
![FIGURE 4](image2)

**FIGURE 3**
QUASISTATIC HYSTERESIS OF SPECIMEN TESTED BY UNJOH ET AL.

**FIGURE 4**
OBIT OF SPECIMEN TESTED BY NISHIDA NISHIDA AND UNJOH

**Selection of Input Ground Motions**

Table 1 shows ground motions considered. The ground motions were scaled using a time scale factor equal to 2 considering the similitude requirements of the specimen.
A primarily research interest is the dynamic behavior under multidirectional loading condition. Thus, it was determined to use two horizontal and one vertical components in this study. The target response was set at 0.17 m (±10%), which was the maximum displacement of the test by Nisida and Unjoh, and the amplitude of input ground motions was adjusted to provide similar maximum response. Ground motions that have repetition of strong pulses were explored.

The results demonstrated that if the ground motions recorded on the ground surface near the Tsugaru Bridge during the 1983 Nihonkai Chubu, Japan, earthquake are scaled up by 400%, the similar maximum displacement is obtained. Repetition of nonlinear behavior and large response in all the regions were expected to be observed due to several strong pulses. Thus, the 400% Tsugaru Bridge records were selected for the input ground motions for this study. The LG, TR and UD components were inputted in the X, Y and Z directions of the table, respectively. The peak ground accelerations for the X, Y and Z directions are 11.12 m/sec², 9.52 m/sec², and 8.2 m/sec², respectively. The expected maximum response displacements in the X and Y directions were 0.15 and 0.16 m.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Station</th>
<th>PGA comp1 (m/sec²)</th>
<th>PGA comp2 (m/sec²)</th>
<th>PGA comp3 (m/sec²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1983 Nihonkai Chubu, Japan, EQ</td>
<td>Tsugaru Bridge</td>
<td>2.78</td>
<td>2.38</td>
<td>2.05</td>
</tr>
<tr>
<td>1989 Loma Prieta, US, EQ</td>
<td>LGPC 1)</td>
<td>5.94</td>
<td>5.52</td>
<td>8.72</td>
</tr>
<tr>
<td>1994 Northridge, US, EQ</td>
<td>Sylmar Olive View Med FF 1)</td>
<td>8.27</td>
<td>5.93</td>
<td>5.25</td>
</tr>
<tr>
<td></td>
<td>JMA Kobe</td>
<td>6.17</td>
<td>8.18</td>
<td>3.32</td>
</tr>
<tr>
<td>1995 Hyogo-ken Nanbu, Japan, EQ</td>
<td>JR Takatori</td>
<td>6.66</td>
<td>6.42</td>
<td>2.90</td>
</tr>
<tr>
<td></td>
<td>East Kobe Bridge</td>
<td>2.81</td>
<td>3.27</td>
<td>3.95</td>
</tr>
<tr>
<td>1999 ChiChi, Taiwan, EQ</td>
<td>TCU084 1)</td>
<td>11.34</td>
<td>4.09</td>
<td>3.34</td>
</tr>
<tr>
<td>2003 Tokachioki, Japan, EQ</td>
<td>K-Net Chokubetsu (HKD086) 2)</td>
<td>7.90</td>
<td>7.30</td>
<td>2.76</td>
</tr>
<tr>
<td>2004 Niigata Chuetsu, Japan, EQ</td>
<td>K-Net Ojiya (NIG019) 2)</td>
<td>13.08</td>
<td>11.47</td>
<td>8.20</td>
</tr>
<tr>
<td></td>
<td>K-Net Tohkamachi (NIG021) 2)</td>
<td>8.50</td>
<td>17.16</td>
<td>5.64</td>
</tr>
</tbody>
</table>

1) Obtained at PEER Strong Motion Database [Regents of the University of California, 2000]
2) Obtained at K-NET [K-NET/NIED]

TABLE 1
GROUND MOTION CONSIDERED

**DYNAMIC RESPONSE OF CIRCULAR REINFORCED CONCRETE BRIDGE COLUMN**

The shake table test had two phases; one was for dynamic response in elastic range, and the other was for that in nonlinear range. The amplitudes of the ground motions were scaled by 20% and 400% for each test. White noise tests for the X, Y and Z components were conducted to determine predominant natural period of the specimen prior to and after the earthquake simulation tests. The periods prior to the test were 0.3 seconds in both horizontal directions, and 0.08 seconds in the vertical direction.

Figure 5 shows the response displacement at the center of gravity of the top mass when subjected to the 20% Tsugaru Bridge records. Analytical prediction described later is also shown in the figure. The maximum displacements were 0.007 and 0.006 m in the X and Y directions and no longitudinal reinforcing bar had yet yielded. The maximum
rebar strain induced during the excitation was 0.0013 at 0.02 m above from the bottom of the column. A few hair cracks were observed after the shaking.

![Graph showing displacement over time](image)

**FIGURE 5**
RESPONSE DISPLACEMENT AT C. G. OF TOP MASS SUBJECTED TO 20% TSUGARU BRIDGE RECORD

Figures 6 and 7 shows the response displacement and its orbit at the center of gravity of the top mass when subjected to the 400% Tsugaru Bridge records. Analytical prediction described later is also shown in Figure 6. Since a much larger response than expected occurred in the X direction, the response exceeded the range of the laser sensor between 16.37 and 16.65 seconds. Table 2 shows peak displacements of each pulse and events occurred. The events occurred only in the X direction are shown in the table because the response was dominant in this direction until fracture of longitudinal bars occurred at 16.4 seconds. The numbers in the figure corresponds to the peaks shown in the table.

When the specimen was subjected to the 400% Tsugaru Bridge records, the first large response occurred at 7.5 seconds, which resulted in a response displacement nearly reaching the ultimate displacement of the specimen (= 0.055 m). No severe damage but slight cracks were observed. The response displacement increased gradually. Even after 4 times repetition of 40-75% larger displacements than the ultimate displacement, slight spalling of cover concrete was observed. The response increased to twice of the ultimate displacement at 11.99 seconds. Between 13 and 15.5 seconds, the cover concrete was spalled, and then buckling of the rebar occurred along with severe spalling of concrete. The maximum response exceeded 0.15 m when the buckling occurred. At around 16 seconds, the response exceeded 0.2 m and several longitudinal bars were fractured. The specimen did not lose the stability and stood almost straight with minor residual displacements after the excitation even though the specimen with such severe damage experienced several strong pulses especially in the Y direction after 17 seconds as shown in Figure 6 (b).

The damage was concentrated from the bottom to a height of 0.25 m, and the bar bucking and fracture occurred between the hoops at 0.075 and 0.150 m as shown in
Figure 8. 22 of 40 longitudinal reinforcing bars were fractured, which occurred mostly at the X faces, and the core concrete were completely crushed at the bottom of the column.

<table>
<thead>
<tr>
<th>Points</th>
<th>Time (sec)</th>
<th>Distance</th>
<th>Disp. in X</th>
<th>Disp. in Y</th>
<th>Events observed at Xp and Xn faces</th>
</tr>
</thead>
<tbody>
<tr>
<td>1p</td>
<td>7.500</td>
<td>0.050 m</td>
<td>0.046 m</td>
<td>0.019 m</td>
<td>Several cracks</td>
</tr>
<tr>
<td>1n</td>
<td>7.875</td>
<td>0.077 m</td>
<td>-0.065 m</td>
<td>-0.040 m</td>
<td>Several cracks</td>
</tr>
<tr>
<td>2p</td>
<td>8.635</td>
<td>0.034 m</td>
<td>0.034 m</td>
<td>0.004 m</td>
<td>Propagation of cracks</td>
</tr>
<tr>
<td>2n</td>
<td>9.125</td>
<td>0.090 m</td>
<td>-0.088 m</td>
<td>-0.014 m</td>
<td>Propagation of cracks</td>
</tr>
<tr>
<td>3p</td>
<td>9.630</td>
<td>0.091 m</td>
<td>0.084 m</td>
<td>-0.035 m</td>
<td>Propagation of cracks</td>
</tr>
<tr>
<td>3n</td>
<td>10.225</td>
<td>0.096 m</td>
<td>-0.093 m</td>
<td>0.022 m</td>
<td>Slight spalling at Xn face</td>
</tr>
<tr>
<td>4p</td>
<td>10.885</td>
<td>0.085 m</td>
<td>0.056 m</td>
<td>-0.063 m</td>
<td></td>
</tr>
<tr>
<td>4n</td>
<td>11.425</td>
<td>0.090 m</td>
<td>-0.083 m</td>
<td>0.035 m</td>
<td></td>
</tr>
<tr>
<td>5p</td>
<td>11.990</td>
<td>0.104 m</td>
<td>0.103 m</td>
<td>0.014 m</td>
<td>Slight spalling at Xp face</td>
</tr>
<tr>
<td>5n</td>
<td>12.520</td>
<td>0.123 m</td>
<td>-0.113 m</td>
<td>-0.048 m</td>
<td></td>
</tr>
<tr>
<td>6p</td>
<td>13.195</td>
<td>0.108 m</td>
<td>0.108 m</td>
<td>0.012 m</td>
<td>Spalling at Xp face</td>
</tr>
<tr>
<td>6n</td>
<td>13.760</td>
<td>0.143 m</td>
<td>-0.139 m</td>
<td>-0.034 m</td>
<td>Spalling at Xn face</td>
</tr>
<tr>
<td>7p</td>
<td>14.410</td>
<td>0.150 m</td>
<td>0.147 m</td>
<td>-0.027 m</td>
<td>Buckling of rebar at Xp face</td>
</tr>
<tr>
<td>7n</td>
<td>15.120</td>
<td>0.154 m</td>
<td>-0.154 m</td>
<td>-0.008 m</td>
<td>Severe spalling at Xn face</td>
</tr>
<tr>
<td>8p</td>
<td>15.765</td>
<td>0.196 m</td>
<td>0.192 m</td>
<td>0.040 m</td>
<td></td>
</tr>
<tr>
<td>8n</td>
<td>16.375</td>
<td>0.188 m</td>
<td>-0.188 m</td>
<td>0.005 m</td>
<td>Fracture of rebar at Xp face and buckling of rebar at Xn face</td>
</tr>
</tbody>
</table>

**TABLE 2**
PEAK DISPLACEMENTS AND EVENTS IN X DIRECTION

**FIGURE 6**
RESPONSE DISPLACEMENT AT C. G. OF TOP MASS SUBJECTED TO 400% TSUGARU BRIDGE RECORD

Figures 9 and 10 show the lateral force versus lateral displacement hystereses, and the orbit of lateral force. The lateral force is computed by multiplying response acceleration by the inertia mass. Figure 11 shows the axial force versus lateral force hystereses. The axial force is obtained by multiplying vertical response acceleration by the inertia mass plus the initial axial force (= -280 kN). Only the responses in the X direction before the rebar buckling are shown in the figures.
FIGURE 7
ORBIT OF RESPONSE DISPLACEMENT

FIGURE 8
LOCAL DAMAGE AFTER TEST (Xp-Yn FACE)

FIGURE 9
LATERAL FORCE VS. LATERAL DISPLACEMENT HYSTERESIS

FIGURE 10
ORBIT OF RESPONSE LATERAL FORCE

FIGURE 11
AXIAL FORCE VS. LATERAL FORCE HYSTERESIS
The maximum response lateral force was about the computed flexural capacity (= 100 kN) as shown in Figure 9. The flexural capacity decreased when the damage level increased. When the rebar at the Xp face, which contributed the flexural strength of the X negative direction, buckled at around 14 seconds, the flexural capacity of this direction decreased by 20%, and then decreased by 70% when the bar fracture occurred.

As shown in Figure 10, the lateral forces of both the directions increased in the negative direction around Point 1n, and reached the capacity. At this point, the lateral forces in the principal directions were 85 kN, and 55 kN, respectively, which were 15% and 45% smaller than the computed flexural capacity. This demonstrates the flexural capacities in the principal directions decreased due to the bilateral flexural loading effects. Similar phenomena were observed at around Points 4p and 4n.

Axial force varied between -55 kN and -630 kN during the excitation, and this can result in 15% change in the response lateral force according to the axial force versus flexural capacity interaction as can be seen in Figure 11. In fact, the lateral force increased up to 108 kN as the axial force decreased to -445 kN at Point A. However, this phenomenon was observed only at this point. This is because the predominant natural period in the vertical direction is 25% of that in the horizontal directions, and thus the horizontal response and axial force barely reached the maximum simultaneously. This explains why the response lateral force is not significantly affected by the fluctuation of the axial force for cantilever-type structures.

**Analytical Simulation**

The effect of damping assumptions on the accuracy of fiber analyses was investigated using the test results, and then the effect of multidirectional seismic excitation on the dynamic response of reinforced concrete bridge columns was analytically investigated.

The same analytical model shown in Figure 1, and the actual material properties were used. The accelerations recorded on the shake table were used for the input signals. No viscous damping and Rayleigh damping using 1st and 5th modes with damping ratios, $\xi$, of 0.1% and 2% were assumed. Since no viscous damping sometimes causes early termination of analyses due to numerical instability, and the Rayleigh damping with $\xi = 0.1\%$ results in almost same response to that with no viscous damping, results from the cases using Rayleigh damping are shown in Figs. 5 and 6.

When the response remains in the elastic range, the analysis with $\xi = 2\%$ predicts the response with sufficient accuracy as shown in Fig. 5. Only 9% larger response is predicted. On the other hand, when $\xi = 0.1\%$ is assumed, the response increases up to an 82% larger response than the test.

On the other hand, the analysis with $\xi = 0.1\%$ provides better agreement with the test results when the column experiences severe nonlinear response as shown in Figure 6. The response is well-predicted up to 14 seconds (between Points 6n and 7p) before the rebar buckling. After the buckling occurs, the analysis predicts smaller response. The analysis with $\xi = 2\%$ predicts 20-30% smaller response than that with $\xi = 0.1\%$.

Using the analytical model with the Rayleigh damping with $\xi = 0.1\%$, the effect of multidirectional loading was investigated. Figure 12 compares the responses subjected to three dimensional loading (XYZ), one horizontal loading (X), one horizontal and one
vertical loading (XZ), and two horizontal loading (XY), respectively. The results up to 14
seconds are shown. Two horizontal ground motions results in 15% larger response than
one horizontal ground motion due to the bidirectional bending effects while the vertical
ground motion does not have a significant effect on the response displacements in the
lateral directions.

Table 3 summarized the effect of multidirectional loading on the maximum response
displacement of the specimen for the Tsugaru Bridge record, JMA Kobe record and
Sylmar Olieve View Med FF record. The amplitude of the ground motions were arranged
to investigate the effect for various level of the response ductility. The table compares the
maximum response under three dimensional loading (3D), two horizontal loading (2D),
and one horizontal loading (1D), and the increasing ratios of the maximum response
compared to those under one horizontal loading are shown in the parenthesis. Table 3
shows the same trend observed in Figure 12. The table also demonstrates that the
increasing ratio depends on the characteristics of ground motions and the response
ductility. When the 300% Tsugaru Bridge record is used, the maximum response
increases by 84% because of the bilateral loading effect. On the other hand, for 80% and
100% of the JMA Kobe record, the bilateral loading results in even smaller response.

![Graph](image)

**FIGURE 12**
EFFECT OF MULTIDIRECTIONAL LOADING

<table>
<thead>
<tr>
<th>Amplitude</th>
<th>1D (m)</th>
<th>2D (m)</th>
<th>3D (m)</th>
</tr>
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<tbody>
<tr>
<td>350%</td>
<td>0.126</td>
<td>0.187</td>
<td>0.191</td>
</tr>
<tr>
<td>300%</td>
<td>0.089</td>
<td>0.160</td>
<td>0.164</td>
</tr>
<tr>
<td>250%</td>
<td>0.073</td>
<td>0.106</td>
<td>0.103</td>
</tr>
<tr>
<td>200%</td>
<td>0.059</td>
<td>0.064</td>
<td>0.063</td>
</tr>
</tbody>
</table>

(a) Tsugaru Bridge record

<table>
<thead>
<tr>
<th>Amplitude</th>
<th>1D (m)</th>
<th>2D (m)</th>
<th>3D (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200%</td>
<td>0.118</td>
<td>0.123</td>
<td>0.124</td>
</tr>
<tr>
<td>120%</td>
<td>0.070</td>
<td>0.078</td>
<td>0.077</td>
</tr>
<tr>
<td>100%</td>
<td>0.071</td>
<td>0.068</td>
<td>0.067</td>
</tr>
<tr>
<td>80%</td>
<td>0.062</td>
<td>0.054</td>
<td>0.054</td>
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</table>

(b) JMA Kobe record

<table>
<thead>
<tr>
<th>Amplitude</th>
<th>1D (m)</th>
<th>2D (m)</th>
<th>3D (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>140%</td>
<td>0.133</td>
<td>0.159</td>
<td>0.157</td>
</tr>
<tr>
<td>120%</td>
<td>0.115</td>
<td>0.131</td>
<td>0.132</td>
</tr>
<tr>
<td>100%</td>
<td>0.092</td>
<td>0.102</td>
<td>0.104</td>
</tr>
<tr>
<td>80%</td>
<td>0.064</td>
<td>0.069</td>
<td>0.070</td>
</tr>
</tbody>
</table>

(c) Sylmar Olieve View Med FF record

**TABLE 3**
EFFECT OF MULTIDIRECTIONAL LOADING
CONCLUSIONS

To investigate the effects of multidirectional seismic excitation on the dynamic response of reinforced concrete bridge columns, an earthquake simulation test and a series of dynamic analyses are conducted. Below are the conclusions determined from the study:

(1) After 4 times repetition of 40-75% larger displacements than an ultimate displacement that is computed based on a current seismic design code of Japan, slight spalling of cover concrete occurs. When the displacement increases up to twice of the ultimate displacement after 6 times of repetition of strong pulses, cover concrete is spalled, and then longitudinal bars are buckled.

(2) The response lateral forces in the principal directions become smaller than the computed flexural capacity due to the bilateral flexural loading effects. The lateral response is not significantly affected by the fluctuation of the axial force because the horizontal response and axial force barely reached the maximum simultaneously due to difference of the predominant natural periods between the vertical and the horizontal directions.

(3) The analysis assuming Rayleigh damping with a damping ratio of 0.1% provides good agreement with the test results for nonlinear dynamic response until rebar buckling while the analysis with a damping ratio of 2% provides better agreement for the elastic response.

(4) The analysis for the 400% Tsugaru Bridge record demonstrates that two horizontal ground motions results in 15% larger response than one horizontal ground motion due to the bidirectional bending effects while the vertical ground motion does not have a significant effect on the response displacements in the lateral directions. The increasing ratio of the maximum response compared to that under one horizontal loading depends on the characteristics of ground motions and the response ductility, and thus further study is needed for various ground motions, and reinforced concrete columns with various natural period and flexural strength.

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