UP-DOWN VIBRATION EFFECTS ON BRIDGE PIERS

KOICHI ONO\textsuperscript{3)}, HIROSHI KASAI\textsuperscript{11)} and MOTOFUMI SASAGAWA\textsuperscript{11)}

ABSTRACT

A relatively large up-down vibration was recorded during the 1995 Hyogoken-Nanbu earthquake. Many people in Kobe area felt a strong up-down shock at the beginning of the earthquake. The effect of this vibration is not normally considered in the seismic design of bridge piers. Many different types of failure and damage of bridge piers were observed after the earthquake. Most of the concrete piers were damaged by bending or shear, or a combination of the two forces.

Some damages in a pier such as a racket type, however, seem to be affected by uniaxial tension. Seismic analysis of bridge piers was performed to check the effect of up-down vibration. Evaluation of up-down impact effect on bridge piers and chimneys was also attempted. The time dependent dynamic analysis represented the shear or bending failure of the piers and the analysis confirmed that the up-down vibration recorded during the earthquake only had a minor effect on the failure.

Analysis of piers due to up-down impact load indicated the possibility that concrete bridge piers might be affected by the uniaxial tension which was imposed at the beginning of the earthquake. A strange failure associated with horizontal cracking which occurred near the top of bridge columns might have been affected by the up-down impact.

Key words: bridge pier, failure, seismic analysis, uniaxial tension, up-down impact load (IGC: H61/E8)

TYPICAL FAILURE MODE OF BRIDGE PIERS

Figure 1 shows typical examples of bridge piers damaged mainly in bending. In Fig. 1(a), only several bending cracks were found and no delamination of the concrete can be observed. In Fig. 1(b), partial delamination of the concrete cover can be seen and the reinforcing bars are exposed. In Fig. 1(c), delamination of cover concrete and cracking of the internal concrete can be seen and some of the reinforcing steel bars are bent. In Fig. 1(d), not only total delamination of the cover concrete but also failure of the internal concrete can be seen. Almost all reinforcing steel bars are bent and some of the stirrups are cut. The pier looks buckled. This damage was mainly seen at the foot of a T-shape single pier. In some cases, damage occurred at the mid section of the pier where the number of reinforcing steel bar was reduced. Figure 2 shows typical examples of a shear failure of a T-shape single pier. Only a few diagonal cracks can be seen in Fig. 2(a) and continued diagonal cracks are developing in Fig. 2(b). In Fig. 2(c), delamination of the cover concrete can be seen and axial reinforcing steel bars are bent. In Fig. 2(d), the upper part of the pier slipped down at the surface of the shear failure and the pier is inclined out of alignment. The internal concrete is completely disintegrated and the reinforcing steel bars are bent with excess elongation. This type of shear failure was found at various sections of the reinforced concrete piers.

SEISMIC ANALYSIS OF BRIDGE PIERS

Seismic analysis of the damaged bridge piers was executed to simulate the damage. T-shape single long and short pier, double deck single pier of racket type and portal frame pier were selected for the analysis.

Analytical Model

Each bridge pier was modelled with plane frame members. The foundation including the piles was modelled as a spring having three components of translation, vertical and rotation. The spring constant was evaluated based on an N-value considering the pile foundation\textsuperscript{11).} The N-value to evaluate the spring constant was used as the average value of the subsoil profile from the pile top to a depth of 5 m. The detail of the spring constant evaluation is shown in the Appendix.

The weight of the superstructure was distributed to

\textsuperscript{3) General Manager, Civil Engineering Department, Konoike Construction Co., Ltd., Chuo-ku, Osaka 541.}
\textsuperscript{11) Chief, ditto.}
\textsuperscript{11) Civil Engineering Department, ditto.}

Manuscript was received for review on August 2, 1995.

Written discussions on this paper should be submitted before August 1, 1996 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101, Japan. Upon request the closing date may be extended one month.
Fig. 1. Damaged bridge pier (mainly in bending)

Fig. 2. Damaged bridge pier (mainly in shear)

Table 1. Piers analyzed

<table>
<thead>
<tr>
<th>Pier Type</th>
<th>Young’s modulus of concrete (MPa)</th>
<th>Mass of the superstructure (t)</th>
<th>$N$-value</th>
<th>Base spring constant</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>T-shape longer pier</td>
<td>$29.4 \times 10^3$</td>
<td>445</td>
<td>10</td>
<td>$2.78 \times 10^6$</td>
<td>$1.37 \times 10^6$</td>
</tr>
<tr>
<td>T-shape shorter pier</td>
<td>$29.4 \times 10^3$</td>
<td>237</td>
<td>20</td>
<td>$1.11 \times 10^6$</td>
<td>$1.08 \times 10^6$</td>
</tr>
<tr>
<td>Racket type shorter pier</td>
<td>$27.9 \times 10^3$</td>
<td>805</td>
<td>10</td>
<td>$7.35 \times 10^6$</td>
<td>$2.44 \times 10^6$</td>
</tr>
<tr>
<td>Portal frame pier</td>
<td>$29.4 \times 10^3$</td>
<td>668</td>
<td>10</td>
<td><strong>Left</strong>: $1.65 \times 10^6$</td>
<td>$1.07 \times 10^6$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>Center</strong>: $2.36 \times 10^6$</td>
<td>$1.52 \times 10^6$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>Right</strong>: $1.41 \times 10^6$</td>
<td>$9.12 \times 10^6$</td>
</tr>
</tbody>
</table>
Weight of Superstructure

<table>
<thead>
<tr>
<th>Weight of Superstructure</th>
</tr>
</thead>
<tbody>
<tr>
<td>445t</td>
</tr>
</tbody>
</table>

![Fig. 3. Framed model of a T-shape pier](image)

Each joint of the analytical beam members. The Young's modulus of concrete and the spring constant of the foundation were assumed to be unchanged. Table 1 shows the selected piers and the weight of the superstructure. Figure 3 shows the framed model of the T-shape single pier. Only the direction perpendicular to the bridge was subjected to the analysis.

**Dynamic Analysis**

Dynamic analysis for the piers modelled was done using modal analysis with NS and UD component of the acceleration waves obtained from the Kobe Marine Meteorological Observatory as shown in Fig. 4. The maximum intensity of the NS component was 818 Gal and that of the UD component was 332 Gal. The calculation of response was performed using a damping coefficient of 5%. The time step of the calculation was set to be 0.02 sec. In order to distinguish the effect of the up-down vibration, the response calculation was performed with both NS wave only and the NS + UD wave. This dynamic analysis was done based on the assumption of linear elastic behavior.

**Results**

The horizontal vibration mode predominated at each bridge pier. The calculated first order natural frequency is shown in Table 2. Table 3 shows the acceleration response for each pier. The amplification of the acceleration response to the maximum base shear input is also shown in this table.

Figures 5 to 8 show the bending strength ratio $R_b$ and shear strength ratio $R_s$ for each section of the piers. $R_b$ and $R_s$ are defined as follows:

$$R_b = \frac{M}{M_u} \quad \text{and} \quad R_s = \frac{V}{V_u}$$

where $M$ and $V$ are the maximum bending moment and shear existing in the pier due to a maximum horizontal acceleration of 100 Gal and the maximum up-down acceleration of 40 Gal. Both $M$ and $V$ were calculated time-dependently and $R_b$ and $R_s$ were calculated for the maximum bending moment and shear force considering a uniaxial force. $M_u$ and $V_u$ are the bending and shear capacity of the pier in the ultimate state. $M_u$ and $V_u$ for each pier were evaluated using a design value for reinforcing steel and the concrete strength. In these Figures, the value of the maximum NS input acceleration at which $R_b$ or $R_s$ becomes unity (1.0) is also shown for the case of NS + UD vibration.

The actual damage, which occurred for many T-shape and racket type piers was severe. The damage in the portal frame pier, however, was very minor although this pier is located near the heavily damaged T-shape pier.

The following points can be stated based on these calculated results:

1. The calculated acceleration response at the top of...

Table 3. Acceleration response of the piers

<table>
<thead>
<tr>
<th>Pier</th>
<th>Position (*)</th>
<th>NS only</th>
<th>NS+UD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Pr, R</td>
<td>R, P</td>
</tr>
<tr>
<td>T-shape long</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T.C.</td>
<td>1144</td>
<td>1.40</td>
<td>1145</td>
</tr>
<tr>
<td>E.B.</td>
<td>1145</td>
<td>1.40</td>
<td>1143</td>
</tr>
<tr>
<td>B.C.</td>
<td>829</td>
<td>1.01</td>
<td>829</td>
</tr>
<tr>
<td>T.C.</td>
<td>2000</td>
<td>2.44</td>
<td>2000</td>
</tr>
<tr>
<td>E.B.</td>
<td>2003</td>
<td>2.45</td>
<td>2003</td>
</tr>
<tr>
<td>B.C.</td>
<td>908</td>
<td>1.11</td>
<td>908</td>
</tr>
<tr>
<td>Racket</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T.C.</td>
<td>1284</td>
<td>1.57</td>
<td>1284</td>
</tr>
<tr>
<td>E.B.</td>
<td>2776</td>
<td>3.39</td>
<td>2777</td>
</tr>
<tr>
<td>B.C.</td>
<td>911</td>
<td>1.11</td>
<td>911</td>
</tr>
<tr>
<td>T.C. (side)</td>
<td>1262</td>
<td>1.54</td>
<td>1960</td>
</tr>
<tr>
<td>B.C. (side)</td>
<td>850</td>
<td>1.04</td>
<td>864</td>
</tr>
<tr>
<td>T.C. (center)</td>
<td>1397</td>
<td>1.71</td>
<td>1966</td>
</tr>
<tr>
<td>B.C. (center)</td>
<td>859</td>
<td>1.05</td>
<td>861</td>
</tr>
</tbody>
</table>

(*) T.C. = top of column, E.B. = end of beam, B.C. = bottom of column
(**) horizontal response by NS only/maximum horizontal input (818 Gal)
(***)) horizontal response by NS+UD/maximum horizontal input (818 Gal)

(2) The up-down vibration effect on the behavior of the piers was minor. The increase of the uniaxial stress due to the UD vibration was 1.3 MPa in the T-shape longer column, 0.7 MPa in the T-shape shorter column, 3.0 MPa in the Racket column and 1.8 MPa in the portal frame, both in tension and compression. These tensile stresses are in the order of their compressive stresses due to their own weight.

(3) The bending failure is more dominant than the shear failure for this particular T-shape longer column pier. The location of failure obtained from the analysis agrees with what happened during the earthquake. Since Rb and Rs in T-shape longer column piers are at the same level, then it is possible that both bending and shear failures occur in these piers as seen in many examples of the damage.

(4) In the T-shape shorter column pier, shear failure is dominant since Rs is larger than Rb. In reality, shear
failure occurred in many shorter column piers.

(5) In the racket type pier, the analysis shows that the bending failure is much more dominant than the shear failure and that the location of the bending damage is at the bottom of the column. In reality, however, extensive damage was observed at the top of column. It can be inferred that the up-down impact vibration might have given some damage to the top of the column at the beginning of the earthquake.

(6) In the portal frame pier, shear failure is dominant. Damage which occurred in the portal frame piers, however, was relatively minor although the calculation indicates that the shear failure might have occurred at the base shear level of 200 Gal for this particular pier.

Figure 9 shows the acceleration response spectrum for the input earthquake wave together with the first order natural frequency for each pier analyzed.

For the earthquake wave used in the analysis, the response level of the portal frame pier is of the same order of magnitude. The input wave was recorded for a relatively stiffed soil profile although both the portal frame and T-shape piers are located in relatively softer soils.

If the response spectrum of the portal frame location is available, the peak would be shifted to the left relative to the peak in Fig. 9. Therefore the response of the portal frame pier would become less sensitive. The reasons that the portal frame type of piers survived well should be investigated more carefully.

EFFECTS OF UP-DOWN IMPACT ON BRIDGE PIERS

Many people experienced a strong up-down impact at the beginning of the earthquake. It is still unclear at this time whether or not the impact was recorded or there was no such up-down impact vibration resulting from the earthquake. In this report, an estimate of the up-down impact effect on a bridge pier and concrete chimney was attempted.

Behavior of a Bridge Pier Due to Up-down Impact

A T-shape bridge pier was modelled to be a spring fixed at the footing and carrying the weight of the superstructure as shown in Fig. 10. Assuming that up-down im-
Impact is transmitted to the top of the column and hits the mass of the superstructure $m$ in the upwards direction due to vibration velocity $V_0$ extending the column spring, the following equation can be derived using the law of conservation of energy:

$$\frac{1}{2}mV_0^2 + \frac{1}{2}k_c\left(\frac{mg}{k_c}\right)^2 = \frac{1}{2}k_c h^2 + mg\left(\frac{mg}{k_c} + h\right)$$  \hspace{1cm} (1)

where $k_c$ is the spring constant of the column of the T-shape pier and $h$ is the maximum elongation of the pier.

The initial contraction of the pier due to its own weight including the super structure is considered in the calculation.

From this relation, the maximum uniaxial tension $\sigma$ can be calculated as:

$$\sigma = \frac{E_c}{L} \sqrt{\frac{m}{k_c} v_0 - \frac{mg}{k_c}}$$  \hspace{1cm} (2)

where $E_c$ is the Young's modulus of the concrete column and $L$ is the length of the column.

Figure 11 shows the relation of $\sigma$ and $v_0$ for the T-shape pier and racket type piers. As shown, it is possible that cracking from uniaxial tension occurs at any section of the column when the input velocity exceeds 15 to 20 kine. Figure 12 shows the damage in a racket type pier.

**Behavior of Concrete Chimney by Up-down Impact**

A concrete chimney of height $L$ was modelled to be a uniform pole fixed at the footing as shown in Fig. 13. Using the wave equation, the impact stress $\sigma$ in the chimney by the Impact vibration velocity $v_0$ given to the chimney at the bottom is given by

$$\sigma = -\rho c v_0 \exp\left\{\frac{\rho A}{M}(x-ct)\right\}$$  \hspace{1cm} (3)

where

$\rho$ = density of the chimney

$E$ = Young's modulus of the chimney

Fig. 11. Stress in column due to up-down impact vibration

Fig. 12. Damage in a Racket type pier

Fig. 13. Modelling of a chimney as a uniform column
The stress at each level is a summation of the tension stresses due to reflection waves, compression stresses from the input waves and compression stresses due to its own weight.

\[
\sigma = \rho cv_0 \exp \left( \frac{\rho A}{M} (2L - x - ct) \right) - \exp \left( \frac{\rho A}{M} (x - ct) \right) - \frac{L - x Mg}{L \ A} \]

(4)

within the range \(2L - ct \leq x \leq L\) and

\[
\sigma = -\rho cv_0 \exp \left( \frac{\rho A}{M} (x - ct) \right) - \frac{L - x Mg}{L \ A} \]

(5)

with the limit \(x < 2L - ct\) where, \(L/c < t < 2L/c\).

Figure 14 shows the results of tensile stress calculations coming down from the top of the chimney for \(v_0 = 50\) kine. It is assumed in this calculation that the compressive stress wave from the bottom to top of the chimney reflects at the top of the chimney with the same magnitude but in tension. In this calculation, the following values were used

\[
L = 50 \text{ m} \\
E = 29.4 \times 10^9 \text{ MPa} \\
\rho = 2.5 \text{ Mg/m}^3 \\
A = 5.0 \text{ m}^2.
\]

The compressive stress in the chimney due to its own weight is taken into considerations.

**CONCLUSIONS**

Time dependent dynamic analysis was done to simulate the damage which occurred in the various types of reinforced concrete bridge piers due to the Hyogoken-Nanbu earthquake. The analysis could represent the type and location of failure which occurred in the piers at a certain level although the analysis was only within a linear elastic range and modelling of the piers was done by plane frame members with spring boundaries at the bottom representing the pile foundation including the subsoils below ground surface.

Portal frame piers, in general, survived well compared to other type piers. Besides the structural characteristics, portal frame piers might have been less sensitive to the Hyogoken-Nanbu earthquake motion due to their higher natural frequency.

The failure mode for racket type piers was rather strange and curious. In many racket piers, a failure associated with horizontal cracking occurred at the top of column although ordinary shear and bending failure occurred on some other racket piers as well.

The cause of such a strange and curious failure should be investigated more carefully including the effect of up-down impact.
REFERENCES

APPENDIX
Pile spring constants were computed from the following expressions:

\[ K_v = \Sigma K_a \]
\[ K_H = \Sigma K_1 \]
\[ K_\theta = \Sigma (K_a X_i^2 + K_4) \]

Where,
\( K_v \): Vertical spring constant of the pile foundation (N/m)
\( K_H \): Lateral spring constant of the pile foundation (N/m)
\( K_\theta \): Rotational spring constant of the pile foundation (N·m/rad)
\( X_i \): Distance of pile i from the footing center (m)
\( K_a \): Pile axial spring constant (N/m). For cast in place piles, the following equation was used.

\[ K_a = \left(0.031(l/D) - 0.15\right) \times \frac{A_p E_p}{l} \]

Where,
\( A_p \): Pile cross-section area (m²)
\( E_p \): Young's modulus of the pile material (N/m²)
\( l \): Pile length (m)
\( D \): Pile diameter (m).

\( K_1, K_4 \): Pile axial-inclined spring constant (N/m, N·m/rad), computed as follows:

\[ K_1 = 4E_p I_p \beta^3 \]
\[ K_4 = 2E_p I_p \beta \]

Where
\( E_p I_p \): Pile bending stiffness (N·m²)
\( \beta \): Pile characteristics constant (m⁻¹)

\[ \beta = 4 \left(\frac{K_H D}{4E_p I_p}\right)^{1/4} \]

\( K_H \): Lateral subgrade reaction modulus (N/m²), computed from the following equation

\[ K_H = K_{H0} \left(\frac{B_H}{0.3}\right)^{-3/4} \]

\[ K_{H0} = \frac{1}{30} \alpha E = \frac{1}{30} \times 2 \times 28N \times 9.80665 \times 10^{-6} \]

\[ B_H = \frac{D}{\beta} \]

Where
\( N \): Standard penetration N-value
\( D \): Pile diameter (m).

The following table summarizes the cases examined in the present analysis.

<table>
<thead>
<tr>
<th>Pile variations</th>
<th>( X_i ) (m)</th>
<th>( n ) (No.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-shape longer pier</td>
<td>Cast in place</td>
<td>-3.5</td>
</tr>
<tr>
<td>Concrete piles</td>
<td></td>
<td>0.0</td>
</tr>
<tr>
<td>( D = 1.0 ) m</td>
<td></td>
<td>3.5</td>
</tr>
<tr>
<td>( l = 10.0 ) m</td>
<td></td>
<td>1.75</td>
</tr>
<tr>
<td>( n = 9 )</td>
<td></td>
<td>1.75</td>
</tr>
<tr>
<td>T-shape shorter pier</td>
<td>Cast in place</td>
<td>-3.75</td>
</tr>
<tr>
<td>Concrete piles</td>
<td></td>
<td>-1.25</td>
</tr>
<tr>
<td>( D = 1.0 ) m</td>
<td></td>
<td>1.25</td>
</tr>
<tr>
<td>( l = 21.0 ) m</td>
<td></td>
<td>3.75</td>
</tr>
<tr>
<td>( n = 16 )</td>
<td></td>
<td>3.75</td>
</tr>
<tr>
<td>Racket type shorter pier</td>
<td>Cast in place</td>
<td>-3.75</td>
</tr>
<tr>
<td>Concrete piles</td>
<td></td>
<td>-1.25</td>
</tr>
<tr>
<td>( D = 1.0 ) m</td>
<td></td>
<td>1.25</td>
</tr>
<tr>
<td>( l = 8.0 ) m</td>
<td></td>
<td>3.75</td>
</tr>
<tr>
<td>( n = 7 + 10 + 6 )</td>
<td></td>
<td>3.75</td>
</tr>
</tbody>
</table>

Table 4. Pier analysis and pile variations