DYNAMIC CHARACTERISTICS OF AN 8-STOREY BUILDING ESTIMATED FROM STRONG MOTION RECORDS

Toshihide KASHIMA¹ and Yoshikazu KITAGAWA²

SUMMARY

The Building Research Institute (BRI) of Japan is a national institute that is carrying out research and development on building engineering, architecture and urban planning. The BRI is operating a strong motion observation network for buildings throughout Japan as one of its research activities. The BRI annex building is one of the stations of the BRI strong motion network and is densely instrumented with twenty-two accelerometers. In this paper, the variation of the dynamic characteristics of the annex building is discussed through the analysis using strong motion records. The natural frequencies and damping ratios of the annex building were analysed based on a single-degree-of-freedom system for 158 strong motion records. The fall of the identified first natural frequencies with elapsed time was clearly recognised. Sixteen strong motion records with relatively large response were selected in order to examine the cause of the fall in detail. The Evolution Strategies (ES) algorithm was applied to the analysis using a swaying-rocking multi-degree-of-freedom system. The ES algorithm is a powerful problem-solving tool based on natural evolution. The building stiffness, rocking stiffness, swaying stiffness and first natural modal damping ratio were identified for sixteen strong motion records by using the ES algorithm. The rocking stiffness, swaying stiffness and first natural modal damping ratio of the building showed stable values independent of the elapsed time. Consequently, it was inferred that the fall of the natural frequencies was caused by the softening of the building stiffness.

1. INTRODUCTION

It is important to understand the dynamic characteristics of buildings when evaluating the seismic performance of a building structure. Strong motion observation is an effective way to gain information about the actual dynamic behaviour of buildings. In 1957, the Building Research Institute (BRI) of Japan started constructing a network to monitor the dynamic behaviour of buildings during earthquakes [Kashima, 2004a]. A large number of strong motion records have since been accumulated [Kashima et al., 2006].

The BRI annex building is located Tsukuba, Japan and is one of the stations of the BRI strong motion network [Kashima, 2004b]. The building is densely instrumented with twenty-two accelerometers. A number of strong motion records have been accumulated since the building was completed in 1998. Firstly, this paper analyses the fundamental dynamic characteristics of the building using strong motion records. Secondly, a detailed investigation is performed in consideration of the dynamic soil-structure interaction.

¹ Building Research Institute, 1 Tachihara, Tsukuba, Ibaraki 305-0802, Japan
Email: kashima@kenken.go.jp

² Keio University, 3-14-1 Hiyoshi, Kohoku-ku, Yokohama, Kanagawa 223-8522, Japan
Email: kitagawa@sd.keio.ac.jp
2. OUTLINE OF STRONG MOTION OBSERVATION

2.1 Outline of annex building

The Building Research Institute is located in Tsukuba, 60 km north-northeast of Tokyo. The annex building is a steel reinforced concrete (SRC) framed building with eight storeys above ground and one storey below. The building is supported by a mat foundation 8.2 m in depth and is connected to the main building with a passageway which has expansion joints separating two buildings structurally. The external appearance of the annex building is shown in Fig. 1.

![Figure 1: External appearance of BRI annex building](image)

2.2 Strong motion instruments

The strong motion observation system has a total of twenty-two accelerometers in the annex building, in the surrounding ground and in the main building. The configuration of accelerometers is shown in Fig. 2 and Fig. 3.

Seven borehole accelerometers are installed in the surrounding ground in order to investigate the site effect. The vertical array in the ground consists of four accelerometers (A01, A14, A43 and A89) which are laid in soil layers 1 m, 14 m, 43 m and 89 m in depth. Two other accelerometers (B01 and C01) are installed on the ground surface at different distances from the annex building. The last (N14) is buried beneath the building.

Eleven accelerometers are set up in the annex building in total. Three accelerometers are placed in each of the north-west corner (BFN, 8FN), in the south-west corner (BFS, 8FS) and on the east side (BFE, 8FE) of the basement and 8th floors of the building. At the 2nd and 5th floors, two accelerometers are set up on the east and west sides. Consequently, the three-dimensional behaviour of the building can be grasped.

In addition, four accelerometers are placed in the main building which is a seven-storey SRC building adjoining the annex building.
2.3 Strong motion records

One hundred and fifty-eight strong motion records having JMA (Japan Meteorological Agency) instrumental seismic intensities of 1.0 or higher are used in the analysis. Moreover, sixteen records which have relatively large response amplitudes are selected for detailed analysis. JMA instrumental seismic intensities ($I_s$) and peak ground accelerations ($PGA$) of the sixteen records are listed in Table 1 with earthquake parameters. Epicentres of the sixteen earthquakes are plotted in Fig. 4. Most epicentres are located in the eastern Kanto area.

The maximum $PGA$ among the records was 0.743 m/s$^2$ in case of record No. 6, and the maximum $I_s$ was 3.8 in case of record No. 11. The building has never experienced severe earthquake motions which caused structural damage.
Table 1: Strong motion records for detailed analysis

<table>
<thead>
<tr>
<th>No.</th>
<th>Date and time</th>
<th>Epicenter</th>
<th>(h) (km)</th>
<th>(M)</th>
<th>(\Delta) (km)</th>
<th>(I_s)</th>
<th>PGA (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1998/06/24 23:52</td>
<td>S Ibaraki Pref.</td>
<td>68</td>
<td>4.7</td>
<td>3</td>
<td>2.6</td>
<td>0.193</td>
</tr>
<tr>
<td>2</td>
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<td>N Ibaraki Pref.</td>
<td>59</td>
<td>5.0</td>
<td>60</td>
<td>3.3</td>
<td>0.193</td>
</tr>
<tr>
<td>3</td>
<td>1999/04/25 21:27</td>
<td>N Ibaraki Pref.</td>
<td>59</td>
<td>5.2</td>
<td>61</td>
<td>3.1</td>
<td>0.193</td>
</tr>
<tr>
<td>4</td>
<td>2000/04/10 06:30</td>
<td>S Ibaraki Pref.</td>
<td>55</td>
<td>4.7</td>
<td>7</td>
<td>3.3</td>
<td>0.193</td>
</tr>
<tr>
<td>5</td>
<td>2000/07/21 03:39</td>
<td>Off Ibaraki Pref.</td>
<td>49</td>
<td>6.4</td>
<td>104</td>
<td>3.6</td>
<td>0.193</td>
</tr>
<tr>
<td>6</td>
<td>2002/06/14 11:42</td>
<td>S Ibaraki Pref.</td>
<td>57</td>
<td>5.1</td>
<td>13</td>
<td>3.4</td>
<td>0.193</td>
</tr>
<tr>
<td>7</td>
<td>2003/05/26 18:24</td>
<td>Off Miyagi Pref.</td>
<td>72</td>
<td>7.1</td>
<td>330</td>
<td>3.3</td>
<td>0.193</td>
</tr>
<tr>
<td>8</td>
<td>2003/09/20 12:54</td>
<td>S Chiba Pref.</td>
<td>70</td>
<td>5.8</td>
<td>104</td>
<td>2.8</td>
<td>0.193</td>
</tr>
<tr>
<td>9</td>
<td>2003/11/15 03:43</td>
<td>Off Ibaraki Pref.</td>
<td>48</td>
<td>5.8</td>
<td>103</td>
<td>3.0</td>
<td>0.193</td>
</tr>
<tr>
<td>10</td>
<td>2004/04/04 08:02</td>
<td>Off Ibaraki Pref.</td>
<td>49</td>
<td>5.8</td>
<td>101</td>
<td>2.9</td>
<td>0.193</td>
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<td>11</td>
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<td>S Ibaraki Pref.</td>
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<td>16</td>
<td>3.8</td>
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<tr>
<td>12</td>
<td>2004/10/23 17:56</td>
<td>Chuetsu, Niigata Pref.</td>
<td>13</td>
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<td>166</td>
<td>3.2</td>
<td>0.193</td>
</tr>
<tr>
<td>14</td>
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<td>NE Chiba Pref.</td>
<td>52</td>
<td>6.1</td>
<td>66</td>
<td>3.4</td>
<td>0.193</td>
</tr>
<tr>
<td>15</td>
<td>2005/07/23 16:34</td>
<td>NE Chiba Pref.</td>
<td>73</td>
<td>6.0</td>
<td>61</td>
<td>3.1</td>
<td>0.193</td>
</tr>
<tr>
<td>16</td>
<td>2005/08/16 11:46</td>
<td>Off Miyagi Pref.</td>
<td>42</td>
<td>7.2</td>
<td>301</td>
<td>3.5</td>
<td>0.193</td>
</tr>
</tbody>
</table>

\(h\): Focal depth (km), \(M\): JMA magnitude, \(\Delta\): Epicentral distance (km), \(I_s\): JMA seismic instrumental intensity, PGA: Peak ground acceleration (m/s²)

Figure 4: Epicentres of earthquakes of analytical records

3. ANALYTICAL METHOD

3.1 Parameter identification using SDOF model

In order to discuss the fundamental dynamic characteristics of the building, a single-degree-of-freedom (SDOF) system is assumed as shown in Fig. 5. The SDOF system has three parameters, i.e. a mass \(m\), a stiffness \(k\) and a damping coefficient \(c\). Its dynamic characteristics are represented by two parameters, a natural frequency \(f_0\) and a damping ratio \(\xi_0\).
The relative displacement of the system $d$ is given by subtracting the displacements at the top $d_{RF}$ and at the base $d_{BF}$ of the model.

$$d = d_{RF} - d_{BF}$$

(1)

In case of the BRI annex building, $d_{RF}$ and $d_{BF}$ in the X-direction can be calculated by taking the average of the displacements at two sensor locations.

$$d_{BF} = \frac{\delta_{BFN} + \delta_{BFS}}{2}$$

(2)

$$d_{RF} = \frac{\delta_{RFN} + \delta_{RFS}}{2}$$

(3)

where $\delta$ is the displacement calculated from the observed acceleration record. The left subscript indicates the direction X or Y. The right subscript indicates the sensor location as shown in Fig. 3.

In the Y-direction, $d_{RF}$ and $d_{BF}$ are given by the following equations:

$$d_{BF} = \frac{\delta_{BFN} + \delta_{BFE}}{2}$$

(4)

$$d_{RF} = \frac{\delta_{RFN} + \delta_{RFE}}{2}$$

(5)

Since only two parameters, the natural frequency $f_0$ and the damping ratio $h_0$, are used to express the dynamic characteristics of the SDOF system, the steepest descent method is adopted as an identification algorithm. The steepest descent method is a search algorithm for finding an optimal solution by moving a single searching point, and the following searching point is decided by difference calculus. The steepest descent method requires a unique solution in the search area, therefore it is desirable to start the search from a point near the solution. For this reason, a natural frequency is roughly searched from 0.4 Hz to 4 Hz at intervals of 0.1 Hz before applying the steepest descent method. The damping ratio is fixed to 5% in this pre-search. The natural frequency sought by the pre-search and the damping ratio of 5% are taken as the start points of the steepest descent method. The fitness of a search point is evaluated by the integral square difference between observed and simulated response displacements. A function $f(f_0, h_0)$ representing the fitness is defined by the following equation:

$$f(f_0, h_0) = \frac{\int (d' - d^s)^2 dt}{\int (d^s)^2 dt}$$

(6)

where $d$ is a response displacement (refer to Fig. 5), and superscripts $s$ and $o$ indicate simulated and observed values, respectively. $T_f$ is the time section in which the fitness is calculated, and is taken as 20 seconds (for 5 seconds before, and for 15 seconds after the peak time of $d^o$).

3.2 Parameter identification using swaying-rocking MDOF model

In order to discuss the dynamic behaviour of the building in detail, a multi-degree-of-freedom (MDOF) system is assumed. The BRI annex building is modelled as a seven-mass system. Furthermore, two sets of a spring and a
dashpot are added to the building foundation in order to consider the effect of soil-structure interaction. The additional two degrees of freedom represent horizontal movement (swaying) and rotational movement (rocking) of the foundation. Figure 6 illustrates the analytical model.

The swaying displacement $d_s$, the displacement at the building top due to rocking $d_R$ and the displacement at the building top due to building deformation $d_B$ can be calculated from observed records by the following equations.

\[
SB F G L = -d_s \quad (7)
\]

\[
RB L B R() = -d_R \quad (8)
\]

\[
BR F G L S R = -d_B \quad (9)
\]

where $BF_d$, $GL_d$, $BL_d$ and $BR_d$ are displacements at the locations indicated in Fig. 6 (left). H and W are the height and width of the building, respectively.

In identifying parameters of the MDOF system, the Evolution Strategies (ES) are adopted as an optimisation algorithm. The ES algorithm, which simulates natural evolution, can be powerful algorithm in nonlinear optimisation [e.g. Bäck et al., 2000]. The features of ES are that it can treat real numbers as optimising parameters and does not require formulation of the evaluation function. Therefore ES can be applied to various cases in the field of engineering. Four parameters, which are storey stiffness ratio $B_r$, rocking stiffness ratio $R_r$, swaying stiffness ratio $S_r$ and first natural modal damping ratio of the building $1_h$, are the targets of the optimisation. The ratios $B_r$, $R_r$ and $S_r$ are defined by Eqs. (10) to (12).

\[
r_B = k_i / k_i^0, \quad (i = 1, \ldots, 7) \quad (10)
\]

\[
r_R = k_R / k_R^0 \quad (11)
\]

\[
r_S = k_S / k_S^0 \quad (12)
\]

where $k_i$, $k_R$ and $k_S$ are the stiffnesses of the $i$-th storey, the rocking and the swaying, respectively. $k_i^0$, $k_R^0$ and $k_S^0$ are the initial values of $k_i$, $k_R$ and $k_S$, respectively. $k_i^0$ is taken from the results of the frame analysis in the design. $k_R$ and $k_S$ are calculated by the simplified method proposed in the Japanese seismic code [Iiba et al., 2000]. $k_i$, $k_R$ and $k_S$ are listed in Table 2 with masses $m_i$ and inertia moments $I$ taken from the structural
design document. The natural frequencies and natural modes of the original models in both horizontal directions are shown in Fig. 7.

### Table 2: Parameters of swaying-rocking MDOF model

<table>
<thead>
<tr>
<th>(i)</th>
<th>Mass (m_i) (10^6 kg)</th>
<th>Stiffness (k_i) (10^9 N/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.12</td>
<td>0.78</td>
</tr>
<tr>
<td>6</td>
<td>0.74</td>
<td>1.28</td>
</tr>
<tr>
<td>5</td>
<td>0.77</td>
<td>1.43</td>
</tr>
<tr>
<td>4</td>
<td>0.93</td>
<td>1.51</td>
</tr>
<tr>
<td>3</td>
<td>0.78</td>
<td>1.65</td>
</tr>
<tr>
<td>2</td>
<td>0.78</td>
<td>1.83</td>
</tr>
<tr>
<td>1</td>
<td>0.84</td>
<td>2.90</td>
</tr>
<tr>
<td>0/S</td>
<td>1.26</td>
<td>5.43</td>
</tr>
</tbody>
</table>

\(\chi J = 266 (10^6 \text{ kg} \cdot \text{m}^2)\)
\(\chi k_R = 875 (10^9 \text{ N} \cdot \text{m})\)
\(\chi J = 407 (10^6 \text{ kg} \cdot \text{m}^2)\)
\(\chi k_R = 1232 (10^9 \text{ N} \cdot \text{m})\)

Left subscripts \(X\) and \(Y\) indicate the direction.

The damping coefficients of the rocking and swaying (\(c_R\) and \(c_S\)) are defined by the following equations:

\[
c_R = \frac{2h_R k_R}{\omega_1}, \quad c_S = \frac{2h_S k_S}{\omega_1}
\]

(13)

where \(h_R\) and \(h_S\) are the damping ratios of the rocking and swaying, respectively. \(\omega_1\) is the first natural circular frequency of the system. \(h_R\) and \(h_S\) are fixed at 5%, since the amplitudes of ground vibration of the analytical records are small [Iiba et al., 2001].

The fitness of the MDOF simulation is evaluated by the sum of the integral square difference of the displacements at the building top as shown by the following equation:

\[
f(r_b, r_k, r_s, h_i) = \frac{w_b}{T_i} \int (d^{s}_b - d^{o}_b)^2 dt + w_k \int (d^{s}_k - d^{o}_k)^2 dt + w_s \int (d^{s}_s - d^{o}_s)^2 dt
\]

(14)

where \(d^{s}_{b}, d^{s}_{k}\) and \(d^{s}_{s}\) are displacements of the building, rocking and swaying at the building top, respectively (refer to Fig. 6). Superscripts \(s\) and \(o\) indicate simulated and observed values, respectively. \(w_b\), \(w_k\) and \(w_s\) are
weight functions to adjust differences of amplitudes among \( d_B, d_R \) and \( d_S \). \( T_c \) is the time section in which the fitness is calculated, and is taken as 20 seconds (for 5 seconds before, and for 15 seconds after the peak time of \((d_B^u + d_R^u + d_S^u)\)).

4. ANALYTICAL RESULTS

4.1 Dynamic characteristics of SDOF system

The identified natural frequency \( f_0 \) and damping ratio \( h_0 \) of the SDOF system for every earthquake record are plotted in Fig. 8 with the elapse of time. Diamonds (■, ◆ and ◇) and squares (■, ■ and □) correspond to values in the X-direction and in the Y-direction, respectively. The colour depth and size of the symbols are changed according to the maximum displacement \( d_{\text{max}} \). Figure 8 (c) is a close-up of portion A in Fig. 8 (a) in the Y-direction.

The fall of natural frequencies can be clearly recognised in Fig. 8 (a). The natural frequencies which were 1.8 Hz to 1.9 Hz at the time of completion in 1998, had fallen to 1.3 Hz to 1.4 Hz in 2005. Figure 8 (c) shows that the natural frequencies fall after experiencing earthquakes with large amplitudes. If natural frequencies in the short period of time are compared, it can be observed that the natural frequency becomes lower as the maximum displacement increases.

On the other hand, the damping ratios that are plotted in Fig. 8 (b) widely vary. There is no clear correlation between the damping ratios and the elapse of time, and the values are around 2% in both horizontal directions on average.

![Figure 8: Comparison between observed and simulated displacements](image-url)
4.2 Dynamic characteristics of MDOF system

In order to investigate the cause of the fall of the natural frequencies mentioned above, the physical parameters of the sway-rocking MDOF system shown in Fig. 6 were identified by using the ES algorithm. The identified storey stiffness ratio \( r_b \), rocking stiffness ratio \( r_r \), swaying stiffness ratio \( r_s \) and first natural modal damping ratio \( h_1 \) for the earthquake records listed in Table 1 are plotted in Fig. 9. In the figure, the diamonds (○) and squares (□) indicate values in the X- and Y-directions, respectively.

Although \( r_b \) was about 1.7 just after completion, it has now fallen below 1.0 to about 0.8. The rocking ratio \( r_r \) is stable at around 3. On the other hand, the swaying stiffness ratio \( r_s \) varies compared with \( r_b \), and is about 2 to 10. The clear inclination of the change over the years is recognised in neither the rocking stiffness ratio \( r_r \), nor the swaying stiffness ratio \( r_s \). Although there is some dispersion, the first modal damping ratio \( h_1 \) of the building is 3% to 5% in general, and is 3.6% in the X-direction and 4.1% in the Y-direction on average. Moreover, a large difference appears in the damping ratios between those the X-direction and in the Y-direction in case of earthquake records No. 3, 6, 9 and 10. If one of the two is identified smaller, another becomes larger. Thus, the three-dimensional behaviour of the building, such as torsional movement, was influenced.

![Figure 9: Analytical models](image)

Figure 10 compares the first natural modes between the original (left) and optimised (right) models in the Y-direction for earthquake record No. 11. The rocking ratio \( \frac{d_b}{d_d + d_a + d_s} \) and the swaying ratio \( \frac{d_s}{d_d + d_a + d_s} \) of the optimised model are 5.3% and 1.9%, respectively. These are smaller than the values (13.4% and 4.3%) of the original model, because the rocking and swaying stiffnesses were identified as being higher.

![Figure 10](image)
5. CONCLUSIONS

The annex building of the Building Research Institute (BRI) is one of the stations of the BRI strong motion network and is densely instrumented with many accelerometers. The dynamic behaviour of the building considering the soil-structure interaction was investigated by analysing strong motion records in detail.

By analysing the fundamental dynamic characteristics, the natural frequencies were clearly found to have fallen. The first natural frequencies in the X- and Y-directions fell to 70% in the past seven years.

In order to investigate the cause of the change in detail, the building stiffness, rocking stiffness, swaying stiffness and first natural modal damping ratio of the building were identified using the Evolution Strategies supposing the swaying-rocking MDOF system. From the results, neither the increase nor the decrease tendency was recognised in the rocking stiffness, the swaying stiffness and the modal damping ratio of the building. Therefore it was inferred that the fall of the natural frequencies was caused by the change of the characteristics of the superstructure.

6. REFERENCES

Kashima, T. (2004b), Dynamic Behavior of an Eight-storey SRC Building Examined from Strong Motion Records, 13th World Conference on Earthquake Engineering (13WCEE), Canadian Association for Earthquake Engineering, Paper No.196