SEISMIC PERFORMANCE OF SLIDING ISOLATION BEARINGS DURING VERTICAL MOVEMENT OF BRIDGE GIRDERS

Kazuyuki IZUNO¹, Hiroshi KAWARABAYASHI², Toshihiko NAGANUMA³ and Tsutomu NISHIOKA⁴

¹Member of JSCE, Professor, Dept. of Civil Eng., Ritsumeikan University
   (1-1-1 Noji-Higashi, Kusatsu, Shiga 525-8577, Japan)
   E-mail: izuno@se.ritsumei.ac.jp

²Member of JSCE, Nagoya City Municipal Office (Former student of Ritsumeikan Univ.)
   (Shiomigaoka 254, Midori-ku, Nagoya 458-0037, Japan)
   E-mail: h.kawarabayashi@rd.city.nagoya.lg.jp

³Member of JSCE, Eng. Dept., Hanshin Expressway Public Corporation
   (4-1-3 Kyutaro-cho, Chuo-ku, Osaka 541-0056, Japan)
   E-mail: toshihiko-naganuma@hepc.go.jp

⁴Member of JSCE, Eng. Dept., Hanshin Expressway Public Corporation
   (4-1-3 Kyutaro-cho, Chuo-ku, Osaka 541-0056, Japan)
   E-mail: tsutomu-nishioka@hepc.go.jp

The behavior of sliding isolation bearings with respect to vertical movement due to earthquake excitation is investigated through experiments and simulations. Bridge girders may undergo vertical motion during severe earthquake events, and since the friction damping provided by sliding isolation bearings is dependent on maintaining contact between the bearing and the girder, there remains an issue as to whether sliding isolation bearings are appropriate for this application. It was shown in this study that multi-layer rubber horizontal load distribution (HLD) devices with steel plate reinforcement are essential for bridge applications, as non-reinforced HLD devices are likely to fail under severe vertical tensile loads. The bearing surface, however, is found to be resilient to impact with an uplifted girder, and simulations demonstrate that contact is maintained with at least one bearing during severe ground motion due to lateral rocking of the girder. The results therefore suggest that appropriately designed sliding isolation bearings are suitable for bridge applications.

Key Words: sliding bearings, friction coefficient, impact tests, uplift, PTFE

1. INTRODUCTION

Sliding isolation bearings have been adopted recently for seismic-resistant construction of bridges, particularly in cases where the installation space is restricted. However, it remains necessary to establish a rational design method for sliding isolation bearings. At present, the design methodology for laminated rubber bearing isolators is currently used for sliding isolation bearings, even though the mechanisms of isolation differ considerably.

Fig. 1 shows a schematic diagram of a sliding isolation bearing. The sliding plate under the main girder supports vertical dead and live loads, and dissipates earthquake energy by friction damping, which is achieved by the sliding mechanism of the upper surface of the plate. The horizontal load distribution (HLD) device acts as a horizontal spring that provides a restoring force for the girder, assisting the girder to return to its original position during earthquake motion. A conventional laminated rubber device is often used as the HLD device.

Practical use of sliding isolation bearings first began with the implementation of isolators for buildings¹². Sliding isolation bearings began to be applied for bridges a few years ago, as the designers...
recognized the advantages of these bearings: compactness, and broader flexibility in setting the equivalent natural period compared to conventional laminated rubber isolation bearings. However, the mass above the sliding isolation bearing is much lighter for bridges than for buildings, with the result that bridges are sensitive to lifting during earthquake excitation. This type of floating may break the contact between bearing surfaces, resulting in a loss of friction damping. The lifting of the supported mass therefore represents a worst-case scenario for sliding isolation bearings.

Although many experiments have been conducted on sliding isolation bearings in an attempt to clarify the dependency of the friction coefficient on the loading speed and axial load, most have focused on horizontal movement\(^3\)\(^-\)\(^9\). Furthermore, while many tension tests have been conducted for laminated rubber bearings\(^10\)\(^-\)\(^12\), no test considering the uplift and falling back of structures has been performed for sliding isolation bearings in Japan. Therefore, the performance of sliding isolation bearings with respect to vertical movement is still not clearly understood.

Even though the HLD device does not support the dead load of the girder, the conventional laminated rubber devices used for this function are reinforced with steel plates, which is usually performed to strengthen the device with respect to compression force. The need for such reinforcement and the appropriate vertical stiffness required for these devices is clarified in this study through experiments and analyses. As girders may be uplifted by strong seismic motion\(^13\), the behavior of sliding isolation bearings in response to such vertical motion is also clarified.

### 2. PERFORMANCE OF HORIZONTAL LOAD DISTRIBUTION (HLD) DEVICE

#### (1) Test procedures

The HLD device acts as a horizontal spring during earthquakes to provide a restoring force to the girder. Therefore, the performance of the device is usually evaluated only for horizontal movement. However, this device also experiences vertical force associated with the vertical movement of the girder. A study on the vertical performance of the device is therefore important, particularly with respect to the tension force related to girder uplift.

Loading tests were conducted using a conventional laminated rubber device with reinforcing steel plates (Type A) and a single-layer rubber device without reinforcing steel plates (Type B). Table 1 and Fig. 2 show the specifications of the two devices. Type A consists of three layers of 10-mm-thick rubber (shape factor \(S = 6.0\)), and Type B has a single layer of 30-mm-thick rubber (shape factor \(S = 2.0\)).

**a) Shear tests**

Shear tests were performed to measure the shear stiffness as a basic characteristic of each HLD device. Shear deformation of 52.5 mm, corresponding

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**Table 1** Specifications of test devices.

<table>
<thead>
<tr>
<th></th>
<th>Type-A</th>
<th>Type-B</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dimensions</strong></td>
<td>(a = 250\times250 \text{ mm})</td>
<td>(a' = 250\times250 \text{ mm})</td>
</tr>
<tr>
<td><strong>Dimension of steel plates</strong></td>
<td>(a' = 240\times240 \text{ mm})</td>
<td>(240\times240 \text{ mm})</td>
</tr>
<tr>
<td><strong>Height</strong></td>
<td>(T = 66.6 \text{ mm})</td>
<td>(62 \text{ mm})</td>
</tr>
<tr>
<td><strong>Shear modulus</strong></td>
<td>(G = 1.0 \text{ MPa})</td>
<td>(1.0 \text{ MPa})</td>
</tr>
<tr>
<td><strong>Thickness of rubber layer</strong></td>
<td>(t_r = 10 \text{ mm})</td>
<td>(30 \text{ mm})</td>
</tr>
<tr>
<td><strong>Number of layers</strong></td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td><strong>Total thickness of rubber</strong></td>
<td>(H = 30 \text{ mm})</td>
<td>(30 \text{ mm})</td>
</tr>
<tr>
<td><strong>Shape factor</strong></td>
<td>(S = 6.0)</td>
<td>(2.0)</td>
</tr>
<tr>
<td><strong>Thickness of reinforcing steel plate</strong></td>
<td>(t_r = 2.3 \text{ mm})</td>
<td>-</td>
</tr>
<tr>
<td><strong>Thickness of upper and lower steel plate</strong></td>
<td>(t' = 16 \text{ mm})</td>
<td>(16 \text{ mm})</td>
</tr>
<tr>
<td><strong>Thickness of covering rubber</strong></td>
<td>(c = 5 \text{ mm})</td>
<td>(5 \text{ mm})</td>
</tr>
</tbody>
</table>
Table 2 Shear stiffness of HLD device.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specimen B</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Shear stiffness 1.82</td>
<td>1.92</td>
</tr>
<tr>
<td>1.96</td>
<td>1.94</td>
</tr>
</tbody>
</table>

An ordinal loading procedure is typically employed for conventional laminated rubber bearings, assuming that the 175% shear deformation is an allowable value taken as 70% of the capable shear deformation of 250%10),15). Although the HLD device does not support the vertical dead load like a laminated rubber bearing, the HLD device is physically identical, and the same ordinal loading procedure was adopted. The loading was controlled by displacement of a horizontal actuator using a sinusoidal input of 0.01 Hz.

b) Static loading tensile tests
After the shear test, tension of up to 6 MPa stress was applied without shear to obtain the relationship between tensile force and vertical deformation. The stress of 6 MPa is three times higher than the allowable stress of 2 MPa, which corresponds to a tensile force of 346 kN. After three cycles of sinusoidal loading (0.01 Hz), the shear tests were repeated to evaluate the change in shear stiffness.

c) Destructive tensile tests
The ultimate phenomena and breaking strength were determined through destructive tensile tests. Tensile force was applied without shear until the device was destroyed.

b) Static loading tensile tests
After the shear test, tension of up to 6 MPa stress was applied without shear to obtain the relationship between tensile force and vertical deformation. The stress of 6 MPa is three times higher than the allowable stress of 2 MPa, which corresponds to a tensile force of 346 kN. After three cycles of sinusoidal loading (0.01 Hz), the shear tests were repeated to evaluate the change in shear stiffness.

(2) Test results
a) Shear tests
The shear stiffness of the device, assuming that the steel plates are sufficiently stiff compared to rubber, is calculated by

\[ K_H = \frac{G}{H} \times \frac{A}{H} \]  

where \( A \) is the cross-sectional area of the laminated rubber, \( H \) is the total thickness of the rubber, and \( G \) is the shear modulus of the rubber. Thus, the steel plate reinforcement is expected to have a negligible effect on shear stiffness in this equation. Eq. (1) gives 1.92 MN/m from Table 1 for both devices examined.

Fig. 3 shows the load-displacement hysteresis curves for the shear tests, and Table 2 presents the shear stiffnesses calculated from the hysteresis curves. Low shear stiffness of specimen A-1 was affected by accidentally loosening of the fastening bolts during the loading. The shear stiffnesses of the other specimens are very close to the predicted values, confirming that the reinforcing steel plates have little effect on the shear stiffness of these devices.

b) Tensile tests
Three cycles of tension loading were performed, applying a maximum tension three times higher than the allowable tensile stress in each cycle. Fig. 4 shows the test results. While the reinforced device survived the three cycles of loading, the non-reinforced device was destroyed in the first cycle. The failure started at the boundary between the upper
steel and the rubber layer. After failure, the rubber exhibited many small voids on the broken surfaces. The allowable tension stress is 2 MPa for this device. At 3 MPa (200 kN in Fig. 4) and above, the tensile stiffness decreases sharply in both devices. Thus, tension of more than 1.5 times the allowable tensile stress may cause degradation of the tensile stiffness. Beyond the range of linear behavior, the rubber develops many small voids, which grow until destruction\(^{10}\). Although the reinforced specimen had not changed visibly after the tensile tests, it is likely to have suffered some void-type damage inside.

The initial tensile stiffness calculated from Fig. 4 is 57.0 MN/m for the reinforced specimen, and 18.5 MN/m for the non-reinforced specimen. The hysteresis curve for the reinforced specimen follows a reversed S shape in the second and third cycles of loading. As seen in the destructive tests (described below), the 346 kN load applied in this test is very close to the destructive limit for the reinforced specimen.

Fig. 5 compares the shear test results for the reinforced specimen before and after the tensile test. Although the hysteresis curve is still stable after the tensile test, the shear stiffness is considerably lower (1.53 MN/m). Thus, the shear stiffness dropped by approximately 20-25% following the tensile test, attributable to the void-type damage that is likely to have occurred in the tensile test. However, the stability of the hysteresis curve indicates that the HLD device would have continued to function as a horizontal spring during a severe earthquake with strong vertical motion.

c) Destructive tensile tests

Destructive tensile tests were conducted by extending an actuator until the device failed. The displacement and force were monitored throughout the test. Failure was observed to start at the boundary between the upper steel plate and the rubber layer. Fig. 6 shows the force-displacement relationship for the two devices. The maximum restoring force for the non-reinforced device was 310 kN; 15% lower than that for the reinforced device (370 kN). The displacement at the maximum force point was 78 mm for the non-reinforced device; 12% smaller than that for the reinforced device (89 mm).

Although the minimum necessary amount of steel plate reinforcement cannot be determined from these tests, the reinforced device clearly performs better than the non-reinforced device. This may also be due to the shape factor and the thickness of the rubber layer. As the heat conductivity of rubber is low, it is difficult to control the sulfuration condition of thick rubber such as Type B specimen, possibly resulting in variations in the ultimate strength of the rubber.

3. PERFORMANCE OF SLIDING PLATE

(1) Test procedures

The performance of the sliding plate under impact force, representing the situation in which an uplifted girder falls back onto the plate, was examined through a series of experiments. Fig. 7 and Table 3 show the specifications of the test device. The effective diameter \(d_0\) in Table 3 is equal to the diameter of the reinforcing steel plate in Fig. 7, which provides the design stiffness in compression. The sliding surface of the device was coated with a polytetrafluoroethylene (PTFE) layer strengthened with glass fiber and graphite.
Table 3 Specifications of sliding plate.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of device</td>
<td>$D \phi 200$ mm</td>
</tr>
<tr>
<td>Thickness</td>
<td>$T$ 37 mm</td>
</tr>
<tr>
<td>Diameter of PTFE</td>
<td>$d \phi 180$ mm</td>
</tr>
<tr>
<td>Effective diameter</td>
<td>$d_0 \phi 45$ mm</td>
</tr>
<tr>
<td>Spring constant in compression</td>
<td>$K_c$ 230 MN/m</td>
</tr>
<tr>
<td>Allowable stress in compression</td>
<td>$\sigma_a$ 25 MPa</td>
</tr>
<tr>
<td>Allowable restoring force</td>
<td>$R_a$ 413 kN</td>
</tr>
</tbody>
</table>

Fig. 7 Sliding plate device.

Fig. 8 Test setup for friction coefficient measurement.

a) Axial stiffness and friction coefficient

The axial compressive stiffness and friction coefficient were measured as basic characteristics of the device. The axial stiffness was measured by a vertical loading test in which 3-25 MPa of axial stress was applied three times to the specimen. The axial stiffness was calculated from the third cycle of the obtained hysteresis curve. The maximum stress of 25 MPa was determined based on the allowable stress of the device given the diameter of the reinforcing steel plate. The friction coefficient was measured using the test setup shown in Fig. 8.

Assuming that the device will support the dead load of the girder, a constant force of 207 kN (corresponding to 12.5 MPa, half the allowable stress) was applied vertically. Sinusoidal motion of 0.01, 0.5 and 1.0 Hz was applied horizontally with a displacement of 50 mm. As the specimen slides horizontally, friction occurs between the PTFE surface of the specimen and the stainless steel (SUS316) plate of the loading device. The friction coefficient was calculated as the average of 10 cycles of loading from the measured horizontal restoring force at the end of each loading cycle (displacement: 0 mm). Therefore, the static friction coefficient was not included in this calculation.
b) Impact tests

Impact tests were conducted to verify the performance of the sliding plate in the case of the uplifted girder falling back onto the plate during an earthquake. A weight of 400 kg was dropped on the sliding plate from heights of 1.0, 1.5 and 2.0 m (one impact per specimen). These heights were determined so as to achieve the same potential energy as a real girder (20 t for each bearing) falling from a height of 2-4 cm. Numerical simulations shown later indicate that a girder may lift by up to about 1 cm during a severe earthquake.

Photo 1 shows the test setup for the impact test. The raised weight is released from the specified height onto the specimen. The bottom of the weight was planar. The impact force was measured by a load cell mounted beneath the specimen, and the deformation of the specimen in the vertical direction was measured using a laser displacement meter.

The axial stiffness and friction coefficients were measured 5 days after the impact test. As there is no data suggesting that the friction coefficient changes over this period, it is assumed in this paper that the friction coefficient does not exhibit a time dependence.

(2) Test results

a) Impact tests

The specimens did not exhibit any change in appearance after the impact tests. The specimen subjected to the 1.5 m test was lifted by the shock of the impact (the specimens were not fixed), and was damaged by inadvertent plastic deformation. The other two specimens did not suffer such plastic deformation. As this test was designed to simulate a girder falling from a height of only 2-4 cm, the sliding plate will not be able to bounce high enough to become dislodged as in the present experiment for 1.5 m. It is assumed that the potential energy of the first impact in these tests is equivalent to that which may occur in a real structure. However, it should be noted that effect of further impacts may be more severe than a real case.

Fig. 9 shows the hysteresis curves for these three specimens in the impact tests. The dissipated energy calculated from this figure is 7.1 kJ for 1.0 m, 10.0 kJ for 1.5 m, and 15.3 kJ for 2.0 m. The energy was dissipated in these tests as sound, heat, plastic deformation and damping by the specimen. These values are proportional to the release height.

b) Friction coefficients

Figs. 10-13 show the horizontal restoring force-displacement hysteresis curves measured before and after the impact test. As all specimens exhibited similar behavior, the results for specimen 3 are discussed here as a representative example (2.0 m impact test).

The restoring force is highest in the initial loading cycle and gradually decreases with further loading cycles. In general, the higher the loading speed, the greater the variation in the restoring force. However, no clear trends in the degree of decrease or initial restoring force could be identified in this study.

As the restoring force in Fig. 10 (loading rate: 0.01 Hz) is smaller than that in Figs. 11-13, the friction coefficient under low-frequency loading is considered to be smaller than that under high-frequency loading. The tendency that higher loading speed results in a larger friction coefficient is common for the friction between PTFE and stainless steel. Due to this dependence of the friction coefficient on the loading speed, the rectangular shape of the hysteresis curve rotates clockwise as the loading speed increases. As a sinusoidal input was used for horizontal loading, the loading speed changes during each loading cycle. Thus, the zero displacement point corresponds to the maximum loading speed, and a larger friction coefficient is obtained.

Table 4 and Fig. 15 show the friction coefficients for all loading cases and specimens. For low-frequency loading of 0.01 Hz, such as that which may occur for a girder due to changes in temperature, the friction coefficient is 30% lower than that before the impact test. However, for high-frequency loadings of 0.1-1.0 Hz such as that which may occur during an earthquake, the friction coefficients were not affected by impact. These results clearly show that the friction-damping function of the bearing is not degraded by vertical impact with a dropping girder under seismic loading.

c) Axial stiffness

Fig. 16 compares the axial stiffness of the sliding plate before and after the impact test for specimen 3 (2.0 m impact test). Fig. 17 shows the same comparison for all cases. The specimens all exhibited a
Fig. 10  Force-displacement relationship before and after impact test under 0.01 Hz loading.

Fig. 11  Force-displacement relationship before and after impact test under 0.1 Hz loading.

Fig. 12  Force-displacement relationship before and after impact test under 0.5 Hz loading.

Fig. 13  Force-displacement relationship before and after impact test under 1.0 Hz loading.

Fig. 14  Vertical axial force histories.
Table 4 Change in friction coefficients.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Loading Frequency (Hz)</th>
<th>Before Tensile Test</th>
<th>After Tensile Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen 1</td>
<td>0.01</td>
<td>0.11</td>
<td>0.10</td>
</tr>
<tr>
<td>Specimen 2</td>
<td>0.1</td>
<td>0.14</td>
<td>0.15</td>
</tr>
<tr>
<td>Specimen 3</td>
<td>0.5</td>
<td>0.15</td>
<td>0.16</td>
</tr>
<tr>
<td>Specimen 4</td>
<td>1.0</td>
<td>0.15</td>
<td>0.16</td>
</tr>
</tbody>
</table>

maximum decrease in axial stiffness of 15% (except for specimen 2, which was inadvertently damaged in the test). Although the effect of this degradation of axial stiffness on the seismic response of the entire bridge system should be considered in more detail in future studies, the present results indicate that sliding plate supports the vertical load well, even after the impact due to falling back of the girder onto the plate.

4. NUMERICAL SIMULATIONS

(1) Simulation model

The seismic response of a bridge pier model was analyzed numerically to study the influence of the axial stiffness of the HLD device on the bridge response. The two-dimensional model employed is shown in Fig. 18. This has a T-shaped RC pier and 5 I-shaped steel main girders equipped with 5 sliding plates and 4 HLD devices. The pier is 10.5 m in height, 16.8 m in width, and has a supported mass of 466 t. The mass of the pier itself is 516 t. The pier and girder are modeled using beam elements, and the HLD devices and sliding plates are modeled by two springs (horizontal and vertical) at each location.

As the pier may exhibit an inelastic response during a large earthquake, the modified Takeda rule was used as an inelastic hysteresis model for the pier. Fig. 19 shows the assumed moment-curvature relationship for the pier.

The damping factors of all elements were set to 0.02. The bottom of the pier was fixed in all directions for more reliable evaluation of the response considering uplift of the girder. Fig. 20 shows the fundamental vibration mode (0.30 s, 3.1 Hz).
Spring element for HLD device
Spring element for sliding plate
Beam element for pier and girder
Fixed at bottom

Fig. 18 Model of bridge pier with sliding isolation bearings.

Fig. 19 Moment-curvature relationship for bridge pier model.

Fig. 20 Fundamental vibration mode.

Fig. 21 Vertical load-displacement relationship for sliding plate model.

Fig. 22 Vertical load-displacement relationship for HLD device model.

Table 5 Vertical stiffness assumed for spring elements.

<table>
<thead>
<tr>
<th></th>
<th>Stiffness in compression</th>
<th>Stiffness in tension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding plate</td>
<td>400</td>
<td>0.4</td>
</tr>
<tr>
<td>Multi-layer HLD</td>
<td>400</td>
<td>60</td>
</tr>
<tr>
<td>Single-layer HLD</td>
<td>50</td>
<td>20</td>
</tr>
</tbody>
</table>
The sliding plate was modeled as an asymmetric piecewise linear spring (Fig. 21) in the vertical direction, and a bilinear spring with perfectly elasto-plastic skeleton curve in the horizontal direction. As the sliding plate supports the dead load of the girder, the vertical spring is set initially as an offset from the origin. The friction coefficient used for the horizontal spring is 0.1, as determined in the previous chapter.

The HLD device was also modeled as an asymmetric piecewise linear spring (Fig. 22) in the vertical direction, but as a linear spring in the horizontal direction. An asymmetric model was used for the vertical direction because rubber exhibits lower axial stiffness in tension than in compression. As this device does not support the dead load of the girder, the vertical spring model was set initially at the origin. The spring constant of the horizontal spring was set at 2 MN/m based on the present experimental results.

The sliding plate was modeled as an asymmetric piecewise linear spring (Fig. 21) in the vertical direction, and a bilinear spring with perfectly elasto-plastic skeleton curve in the horizontal direction. As the sliding plate supports the dead load of the girder, the vertical spring is set initially as an offset from the origin. The friction coefficient used for the horizontal spring is 0.1, as determined in the previous chapter.

Although the sliding plate does not resist tensile force due to uplift of the girder, a small value (1/1000 of its stiffness in compression) is assumed to improve the stability of the numerical simulation. The compressive stiffness of the sliding plate was set to the value shown in Table 5 according to the present experimental results.

The axial tensile stiffness of the HLD device was set at 57.0 MN/m to represent a multi-layer reinforced device and 18.5 MN/m to represent a single-layer non-reinforced device based on the present experiments. The axial compressive stiffness for the multi-layer device was set at 400 MN/m based on calculations using the Japanese bridge bearing design handbook (403 MN/m). As the shape factor of the single-layer device is 2.0 and out of the range shown in the handbook, a value of 50 MN/m was used based on the value calculated from the conventional Hattori-Takei equation (56 MN/m).

Fig. 23 shows the input accelerations used in the analysis. The inputs are JR Takatori records of the 1995 Hyogo-ken Nanbu earthquake in Japan (17). The NS component was used for the transverse direction, and the UD component was used for the vertical direction. Newmark's $\beta$ method with $\beta = 1/4$ was used with a time increment of 0.002 s.

(2) Simulation results

a) Effect of HLD device on bridge response

Fig. 24 shows the seismic response of the system with the multi-layer HLD device, and Fig. 25 shows the response for the single-layer HLD device. Due to the input of vertical accelerogram, the responses in Figs. 24-25 exhibit significant high-frequency components. Table 6 lists the maximum responses in these simulations.

A vertical displacement greater than zero indicates uplift of the girder. One or two uplifts of several millimeters occur in both simulations. The maximum girder uplift is 2.5 mm for the multi-layer HLD device and 1.9 mm for the single-layer HLD device. Considering the precision of the numerical simulations, these values are considered to be identical. Therefore, the vertical stiffness of the HLD device appears to have little effect on the uplift of the girder.

In the restoring force histories, positive values indicate tension and negative values denote compression. The restoring force of the sliding plate of the multi-layer device is smaller than that for the single-layer device. Although the maximum compressive displacements are similar (3.5 and 4.1 mm), the compressive force differs appreciably (1200 kN)
vs. 1500 kN) due to high stiffness of the sliding plates. This difference can be attributed to the difference in force supported by the HLD device: the multi-layer HLD device supports 150 kN while the single-layer device supports only 50 kN. In other words, the sliding plate of the system with single-layer HLD device assumes more of the vertical load than in the system with multi-layer HLD device. Nevertheless, the maximum vertical compressive stress experienced by the multi-layer HLD device is 3 MPa, which although being three times larger than that for the single-layer device, is still sufficiently small compared to the allowable stress of 25 MPa.

The necessary tensile strength required for girder uplift in this bridge system based on the specifications for Japanese highway bridges is 300 kN, corresponding to 0.3$R_d$ ($R_d$: dead load). However, the maximum tensile response as shown in Table 6 is smaller than this value. For the single-layer HLD system in particular, the maximum tensile restoring force is only 20 kN, which is less than 1/10 of the required value. This is due largely to the low vertical stiffness of the HLD device. Although it is indeed necessary for the system to resist uplift of the girder, the empirical requirement of 0.3$R_d$ is substantially overestimated based on the present investigation. Thus, determining the minimum tensile strength requirement based on the dynamic response of the bridge will allow for more rational design of the bearing system.

b) Bending moment response of pier

The effect of the vertical stiffness of the HLD device on the response of the bridge pier was studied by calculating the bending moment at the joint of the overhanging beam section (A-A’ in Fig. 26) and at the pier bottom (B-B’).
Fig. 26 Cross-sections for bending moment analysis.

Table 7 Maximum responses of bending moments.

<table>
<thead>
<tr>
<th></th>
<th>A-A’ section</th>
<th>B-B’ section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Multi-layer HLD</td>
<td>9.3</td>
<td>17.7</td>
</tr>
<tr>
<td>Single layer HLD</td>
<td>12.3</td>
<td>17.9</td>
</tr>
</tbody>
</table>

Fig. 27 shows the bending moment histories for the multi-layer HLD device system, and Fig. 28 shows the results for the single-layer HLD device system. Each figure contains (a) bending moment at A-A’ section, and (b) bending moment at B-B’ section.

Table 7 shows the maximum response for each case. The single-layer HLD system exhibits a 30% larger bending moment than the multi-layer HLD system. This is attributed to the larger axial force at the sliding plate for the single-layer HLD system due to less support at the HLD device. Calculation of the dynamic response in consideration of the vertical stiffness of the sliding plate and HLD device is therefore necessary for proper design of the overhanging beam.

At the pier bottom, however, there is only 1% difference between the two systems. Although the bridge pier responds inelastically, the stiffness after yielding is not zero, as shown in Fig. 19. Therefore, the similarity of the bending moment response indicates that there is little difference in the seismic response at the pier bottom. Thus, while the difference in the bending moment on the overhanging beam section (A-A’) may affect rotation at the pier top, there is no difference in response at the pier bottom. This suggests that it is not necessary to consider the HLD device in designing the bridge pier.

c) Seismic response of bridge system including girder

A seismic response analysis of a two-span continuous girder viaduct system (Fig. 29) was also conducted to examine the effect of girder vibration on the bearing response. An eigenvalue analysis was performed using a three-dimensional frame model. Fig. 30 shows the first and second vibration modes.

The first vibration mode is a transverse mode with a natural period of 0.35 s (2.8 Hz), and the second vibration mode is a longitudinal mode with a natural period of 0.22 s (4.58 Hz).
Using the same earthquake accelerations (NS and UD components of JR Takatori record), the vertical displacement response of the sliding plate on the right-most main girder on the pier end is shown in Fig. 31. The maximum uplift of the girder is 12 mm, almost 6 times larger than that observed in the two-dimensional model above. The girder also lifted 20 times for a total duration of 1.5 s during the 10 s response. Furthermore, although the girder lifts only at the right-most and left-most bearings in the two-dimensional model, the girder lifts on all bearings in this three-dimensional model.

The results for this model coincide with those of the two-dimensional model when the bending stiffness of the girder is set at infinity. Thus, girder flexural vibration appears to have a substantial effect on the seismic response of the sliding bearing.

Fig. 32 shows the vertical displacement histories at the right-most and left-most main girders at the pier end. If the girder lifts up on all bearings at the same time, no friction damping will be acting. However, the two outer girders vibrate in the reverse phase, adopting a rocking motion. Therefore, the girder does not lift entirely of all bearings. Furthermore, the friction damping on the bearings in contact with the girder will vary due to the pressure dependence of the friction coefficient: high damping for low pressure and low damping for high pressure. This demonstrates that the variation of friction on a transverse line of bearings on the pier will be relatively small.

5. CONCLUDING REMARKS

This paper discussed the performance of sliding isolation bearings with respect to vertical movement of the load through experiments and numerical simulations.

It was found through experiments on the horizontal load distribution (HLD) device that multi-layer rubber devices with reinforcing steel plates can withstand three times the prescribed allowable tensile force. Although the shear stiffness drops by 20% after sever tensile loading, the reinforced device continues to function as a horizontal spring to return the girder to its origin. The non-reinforced single-layer HLD device exhibited much lower tensile capacity and tensile deformability than the reinforced device. Therefore, although the device does not experience compressive force due to the dead load, the reinforcing steel plates are required if the device is to survive a seismic event with strong tensile response.

The sliding surface was found to suffer no damage in tests simulating vertical impact with an uplifted girder. The vertical stiffness was degraded to some
extent by the impact with the falling girder, but the friction coefficient remained unchanged. The sliding plate therefore maintains its function even if the uplifted girder falling back onto the plate during earthquake excitation.

Finally, in simulations it was found that the girder rises by less than 1 cm during severe earthquake events. The girder undergoes a rocking motion such that at least one bearing remained in contact with the girder at all times. Thus, friction damping remains effective during earthquake excitation, even though the girder may lift. And also the bending stiffness for deflection of the girder was shown to have a marked influence on the behavior of girder uplift. HLD devices with low vertical stiffness appear to be a better choice to avoid excessive tensile force, and it is suggested that the minimum requirement with respect to resistance to uplift may be more rationally determined by dynamic response analysis.

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