Reduction of Seismic Inertia Force by the Application of Filling Materials to Girder Ends

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ABSTRACT

As a seismic strengthening method of existing multi-span girder bridges, application of rubber-made filling materials to the expansion gaps of girders was proposed. Then, the inertia force reduction effect provided by the filling materials when they resist the response of the superstructure from the very beginning of earthquake loading was evaluated. For the evaluation, sensitivity analysis was conducted by changing the level of axial direction rigidity of the filling materials.

It was found that, as the axial direction rigidity of filling materials increases, the maximum response values of the superstructure and the pier become smaller, but the force acting on the abutments due to thermal expansion of girders becomes larger. Hence, it can be said that a rational seismic strengthening is possible if an adequate number of filling materials that can satisfy both the seismic response of the pier and the stability calculation of the abutments is placed at the expansion gaps.

1. INTRODUCTION

After the Hyogo-ken Nanbu Earthquake in 1995, the seismic design of bridge structures against large-scale earthquakes has been predominantly employing the ductility design method and the dynamic analysis method. In the case of ordinary bridges, as the expansion spacing is small, girders tend to collide against the abutment if supports and piers are damaged during an earthquake, at which time the horizontal resistance provided by the abutment can be expected. However, as the damage to the abutment and the resistance provided by the back soil at the time of girder collision are difficult to quantify, the horizontal resistance of the abutment is not taken into account in the ordinary seismic design of bridge structures. As a result, horizontal displacement of the superstructure and piers becomes large when subjected to earthquake loading, as shown in Fig. 1, which is the reason that the seismic strengthening of piers and foundations is needed. However, jacketing of a pier, particularly those situated in a river or a lake, is difficult in terms of costs because construction of a large-scale cofferdam is required. To overcome such a constraint, development of a novel seismic strengthening method which is more rational and economical compared with conventional strengthening methods is needed.

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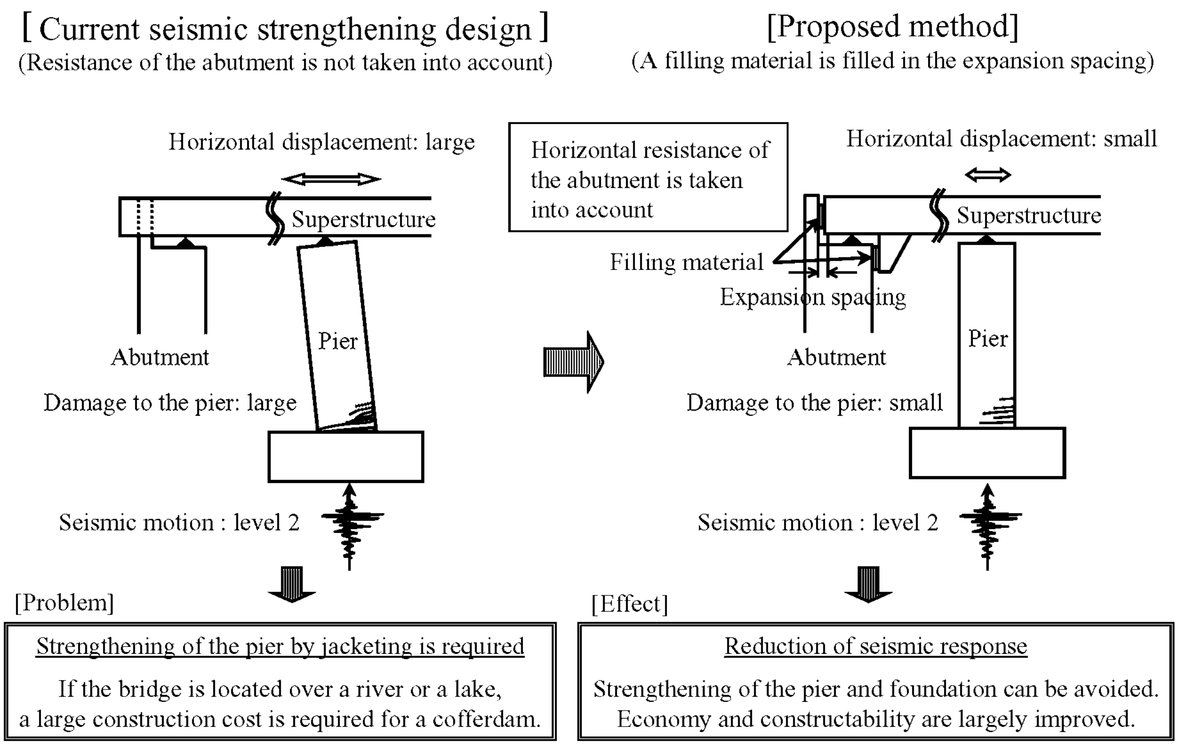

Fig. 1 Outline of the proposed method

As one of such methods, the authors propose a method to apply filling materials, such as those made of rubber, to the expansion spacings between the girder and the abutment as well as between girders, as shown in Fig. 1. These filling materials are expected to provide resistance to the response of the bridge from the very beginning of earthquake loading. Then, the horizontal resistance provided by the abutment section via filling materials can be taken into account in the seismic response analysis of the bridge structure. To adequately evaluate the reduction effect of earthquake inertia force provided by this method, the following three points must be treated properly.

① Adequate modeling of the abutment section
② Evaluation of the performance of filling materials and their application method to the expansion spacings of PC girders
③ Temperature changes of a girder at normal times and safety of an abutment under earthquake loading

In this study, as the basis for the formulation of an analysis model, characteristics of the resistance provided by the abutment section, including the failure mode of the abutment and the resistance of the soil behind the abutment, are presented first using the analysis results of past earthquake damage. Then, by performing a girder collision analysis by taking into account the horizontal resistance from the abutment section, the response characteristics of the target bridge under major earthquake loading were grasped. Concurrently, using the expansion spacing as a parameter, the relationship between the amount of expansion spacing and seismic response was also evaluated.

Next, sensitivity analysis was conducted on the reduction effect of inertia force and the effect on the stability calculation of an existing abutment by applying filling materials to expansion spacings and by changing the axial direction rigidity of those materials. The filling materials to be applied must be a device that has stable compressive strength against repeated loadings and variable loading speeds even under high compression caused by girder collision, and that has little effect on the stability calculation of the vertical wall of the abutment and the foundation not only under earthquake loading but also under normal displacements such as thermal expansion.

From this study, it was found that rational seismic strengthening is possible if an adequate number and adequate shape of filling materials that can satisfy the seismic response of piers and the stability calculation of an abutment are installed at the expansion spacings of an existing bridge.

2. BRIDGE TO BE STUDIED

The bridge to be studied is an existing PC (precast concrete) two-span post-tensioned simple T-girder bridge having a span length of 40 m, which is shown in Fig. 2. Pier 1 of this bridge is situated in the river and the abutments are located at both ends. The bridge structure is right and left symmetrical, with Pier 1 as its center. The bearing support structure at Pier 1 is fixed and those at both abutments are movable.

The detailed structure of the bridge is shown in Fig. 3. The superstructure is constructed of post-tensioned prestressed concrete T girders (four main girders) and the bearing support structure is made of a rubber pad. The pier is constructed of reinforced concrete and oval shaped. The foundation below the pier is a caisson foundation (pile length L = 30 m). The foundation at each abutment is a cast-in-place pile foundation (pile length L = 30 m). The abutment is the reverse T type having a wing and a counterfort. The bridge had once been seismically strengthened using the modified seismic coefficient method in accordance with the Guidelines for the Seismic Design of Highway Bridges, 1971.

The ground at the bridge site is Type II and liquefaction is predicted not to occur even though an earthquake occurs.

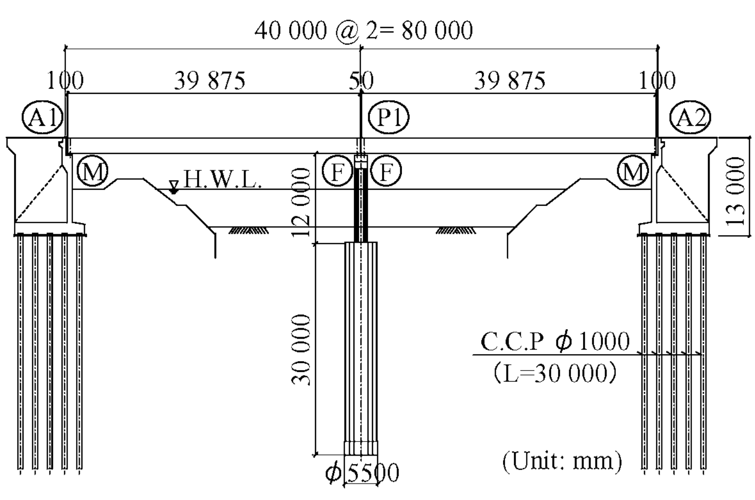

Fig. 2 Bridge to be studied

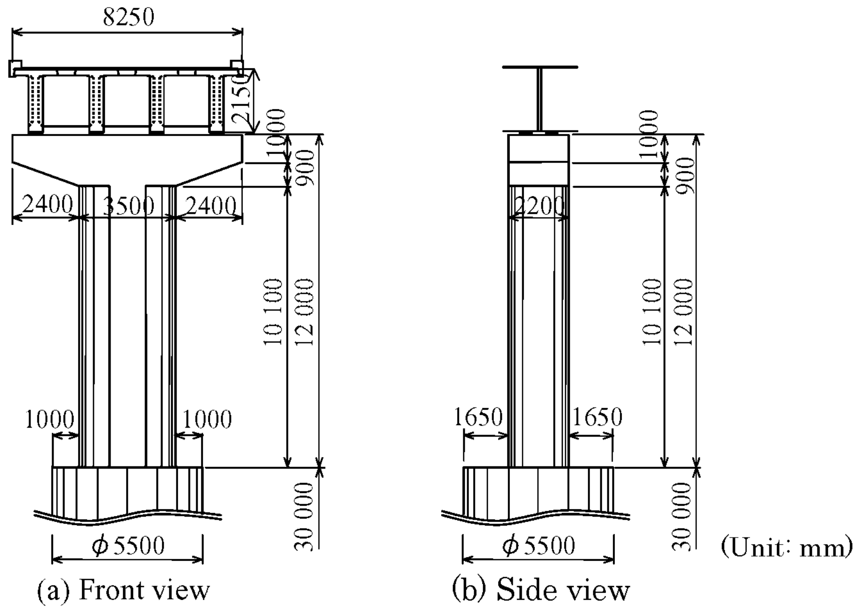

Fig. 3 General structure of Pier 1

3. CHECK OF SEISMIC RESISTANCE OF THE PRESENT STRUCTURE AND AFTER STRENGTHENING BY RC JACKETING

3.1 Check of Seismic Resistance of the Present Structure

The seismic resistance of the present structure of the bridge was checked against Level 2 earthquake. The check was conducted by the dynamic method using the nonlinear time history response analysis. As the check results in the bridge axis direction, Fig. 4 shows the response results of the bending moment-rotational angle relationship at the bottom of Pier 1. The results indicate that Pier 1, which is the fixed support type and will fail by bending, does not satisfy the allowable ductility factor in the bridge axis direction required by the Specifications for Highway Bridges and Commentary, Part V Seismic Design[1)] (Hereafter referred to as Highway Specifications). As to the direction perpendicular to the bridge axis, the allowable ductility factor is satisfied. As to the foundation at the pier, both the bridge axis direction and the direction perpendicular to it are found not to reach yielding.

3.2 Check after Seismic Strengthening by RC Jacketing

As found from the above check, the bending strength of Pier 1 in the bridge axis direction is insufficient. If the pier is to be strengthened with an ordinary RC jacketing, the jacketing thickness shall be 250 mm, D38 shall be arranged in one layer as the axial direction reinforcement, and D22 shall be placed as the hoop ties at a pitch of 150 mm. By this jacketing, the bending strength of the pier will be increased, but a burden to the foundation will also increase. As a result, the strength of the foundation becomes smaller than that of the pier and the required seismic resistance of the foundation is not satisfied.

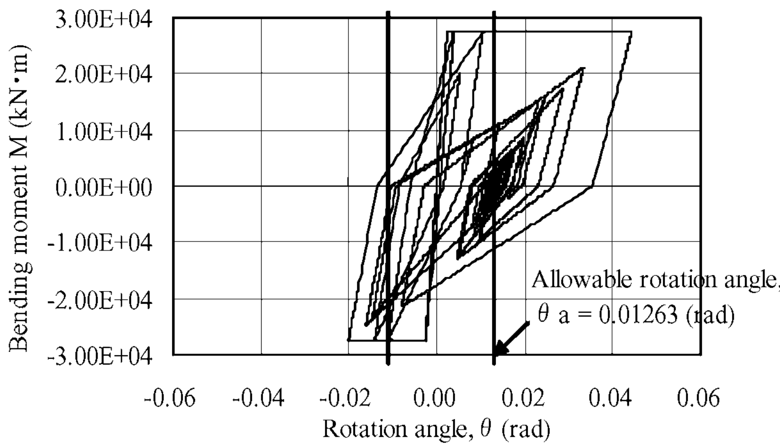

Fig. 4 Response hysteresis of Pier 1 (without strengthening)

4. REDUCTION OF SEISMIC INERTIA FORCE BY THE APPLYCATION OF FILLING MATERIALS TO GIRDER ENDS

4.1 Outline of Investigation and Design Conditions

Based on the check results of the present structure and the structure seismically strengthened by ordinary RC jacketing, we now propose a different seismic strengthening method, as shown in Fig.5, which is to place filling materials, made of rubber, to the expansion gaps at the ends of girders. This method is intended to take into account the horizontal resistance of the abutments in the seismic response analysis of the bridge. It is designed to reduce the seismic inertia force by allowing the filling materials to resist the response of the superstructure from the very beginning of earthquake loading. To evaluate the resulting effect, sensitivity analysis was conducted focusing on the effect on the stability calculation of the existing abutments and the effect to reduce a seismic inertial force, by changing the rigidity level of rubber filling materials in the bridge axis direction. The following assumptions were presumed in this analysis.

(1) The objective of analysis is to find the effect on the bending strength of the pier in the bridge axis direction.
(2) The bearing supports remain intact even though girders collided to the parapet.
(3) The abutment is modeled as failing at the bottom of the parapet. And, the stability calculation must be satisfied as the whole abutment.

4.2 The Entire Analysis Model

As the entire analysis model, a frame model of the entire bridge structure is created, which is shown in Fig. 6. The setting method of individual sub-models will be explained later. At the parapet section, a combination of a collision spring and a resistance spring that takes into account the presence of the parapet and the soil behind the abutment is installed. At the bottom of Pier 1, a nonlinear spring element having a bending moment-rotation angle relationship of the perfectly elastoplastic type is installed in accordance with the Highway Specifications, Part V Seismic Design. Its hysteresis characteristics are modeled using the Takeda model. For the beam section of the pier and the footing, rigid beam elements are used.

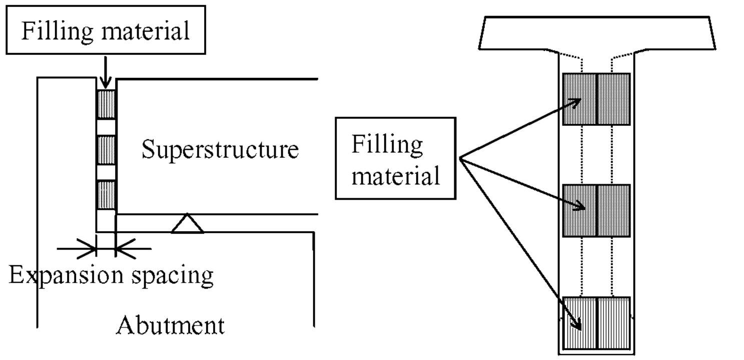

Fig. 5 Outline of the proposed method

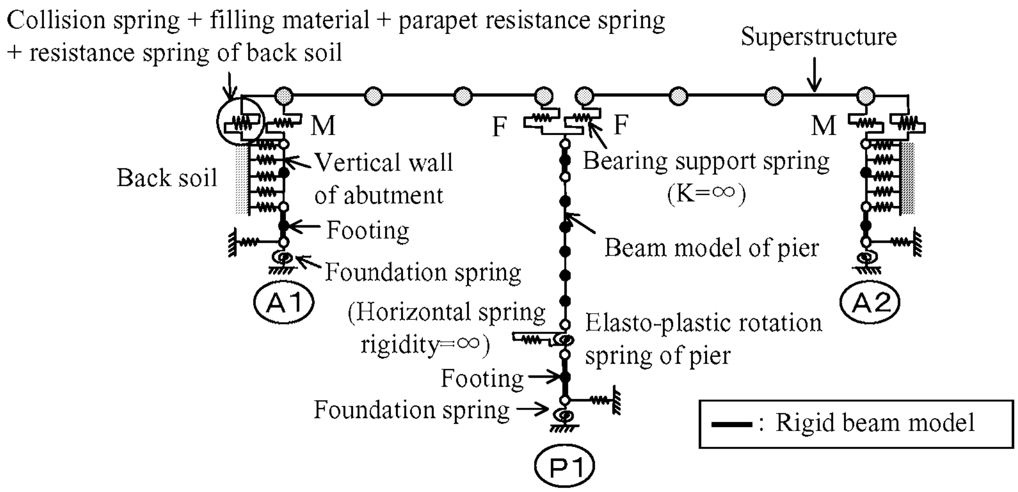

Fig. 6 Frame model for analysis

4.3 Modeling of the Parapet

The parapet at the abutment is modeled by taking into account the failure mode of the parapet at the time of girder collision. As no damage case of a parapet has been reported in Japan, the parapet model was established by referring to the parapet damage at the Chang-Geng Bridge in Taiwan which was damaged during the 1999 Chi-Chi Earthquake[2)]. At this bridge, the parapet was damaged by shear and pushed into the soil behind the parapet for about 1 m as a result of collision of girders against the parapet, as shown as Fig.7. This is considered to have happened because the shear strength of the parapet calculated from Equation (1)～(3) was smaller than the bending strength of the parapet, which is shown in Table 1. For this calculation, ordinary shear strength evaluation equations were used considering that the impact force is part of the external force. Accordingly, in the current analysis, too, a model that is damaged or failed by shear is used, and it is considered that emergency vehicles can go through if steel plates or others are placed even though the parapets are damaged and the expansion gaps are enlarged by earthquake loading. In the calculation of shear strength, the strengths of the parapet and the wing are both taken into account because it was confirmed by a separate FEM analysis that a collision load is resisted by both the parapet and the wing[3)].

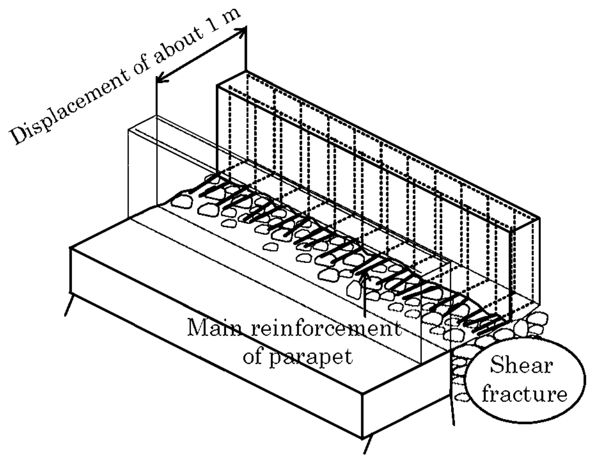

Fig. 7 Damage to the abutment of the Chang-Geng Bridge in the Chi-Chi Earthquake, Taiwan

Table 1 Failure mode of parapets

	Chang-geng bridge	Bridge under study
Tensile main reinforcement ratio (%)	0.200	0.197
Hoop tie ratio by volume (%)	0.000	0.405
Bending strength Py (kN)	4805.3	12443.1
Shear strength Ps (kN)	1908.0	9129.2
Failure mode	Shear failure	Shear failure

The soil behind the abutment is presumed as the sufficiently compacted sandy soil. The N value of the soil by the standard penetration test is assumed as 15, the shear friction angle $\phi = 30^\circ$, and the constant of viscosity C = 0. The initial rigidity $\mathbf{K_1}$ becomes 8.14×10^5 kN/m from the coefficient of subgrade reaction in the horizontal direction of the caisson foundation which is specified in the Highway Specifications, Part IV Substructures[4)] and by taking into account the area behind the parapet. The maximum strength of the earth pressure resistance is set as 4.478×10^3 kN from the calculation using the upper limit value of the horizontal ground resistance at the front face of the caisson foundation and by taking into account only the parapet height, h = 2.638 m.

$$Ps = Sc + Ss \qquad (1)$$
$$Sc = 0.82 \times Pt^{1/3} \times (1/d)^{1/3} \times (\sigma ck)^{1/3} \times b \times d \qquad (2)$$
$$Ss = Aw \times \sigma sy \times d \times (\sin\theta + \cos\theta) \; / \; 10 \times 1.15 \times a \qquad (3)$$

Where Sc : shear strength carried by concrete
Ss : shear strength carried by shear reinforcement
Pt : tensile main reinforcement ratio = 0.197% (D16@150mm)
d : effective height of the cross section of parapet = 0.50 m
b : width of the cross section of parapet = 8.30 m
Aw : sectional area of hoop tie = 90.26 cm^2
σ ck : design strength of concrete = 21N/mm^2
σ sy : yield point of hoop tie = 300N/mm^2
θ : angle between hoop tie and vertical axis = 90°
a : spacing of hoop ties = 250 mm

4.4 Modeling of Filling Materials

Filling materials are placed at the expansion gaps at three locations: between the two main girders and between the main girder and the parapet at two abutments. Here, a rubber is selected as the filling material because its compressive capacity is stable against repeated loadings and a high loading speed even under high strains and high plane pressures. A general type natural rubber is chosen and its compressive stress-strain relationship is established by referring to the results of a past material test shown in Fig. 8 and a past formulation investigation (rubber size: 250 $\times$150 $\times$100 mm thick)[5).

4.5 Modeling of the Abutment

The modeling method of the abutment section varies by the cases filling materials are placed and not placed, as shown in Fig. 9. Three kinds of springs, namely (a) a collision spring or a resistance spring of filling materials; (b) a shear resistance spring of the parapet, and (c) a resistance spring of the back soil, are synthesized into one spring, which is used to model the abutment section as just one combined spring. In the case of the bridge under study, as the strength of the parapet is larger than that of the back soil, the hysteresis characteristics of the combined spring differ before and after the failure of the parapet. Hence, after the parapet failed by shear and after being displaced up to the maximum displacement of the former hysteresis, the model becomes the type to resist seismic loading only by the back soil.

4.6 Analysis Cases

To grasp the effect on the stability calculation of abutments and the seismic inertia force reduction effect, sensitivity analysis was conducted on a total of five cases. One is the case in which filling materials are not placed and a focus is placed on only the expansion gap amount. The other four cases are as shown in Table 2, namely, the number of rubber filling materials is changed from 12, 24, 36, to 48.

4.7 Analysis Method

As the input seismic waveform, No. 1 standard seismic waveform (maximum acceleration 686.831 gal) for Type II ground was used from among seismic waveforms for Level 2, Type II ground motion shown in Reference for the Seismic Design of Highway Bridges[6). As the numerical integration method for the time history response analysis, the Newmark β method was used (β = 1/4). The integration time interval was made to $\triangle$t = 1/20000 seconds to improve the convergence of a solution and the accuracy of response acceleration of girders. In addition to the hysteresis damping, the viscosity damping constant was assumed for each nonlinear material: 2% for the pier, 20 % for the foundation, 5 % for the vertical wall at the abutment, and 0% for the parapet and rigid members. As the viscosity damping of the entire bridge system, the Rayleigh damping was used.

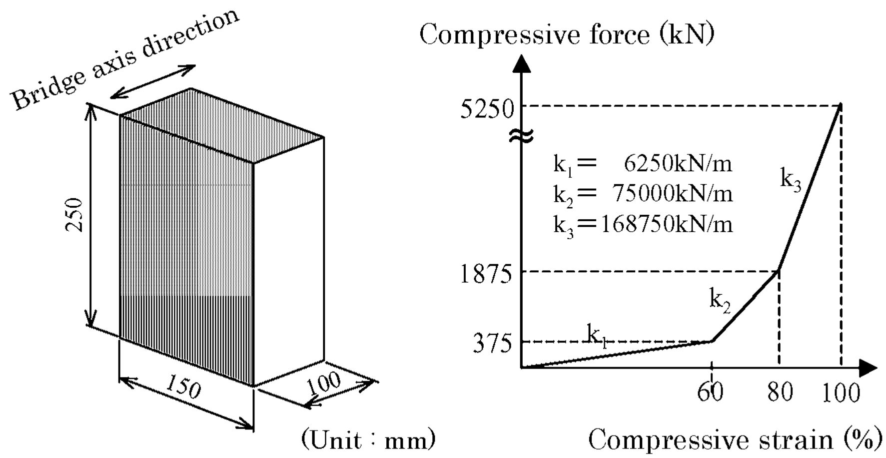

Fig. 8 Types of filling materials and relationship between compressive force and compressive strain

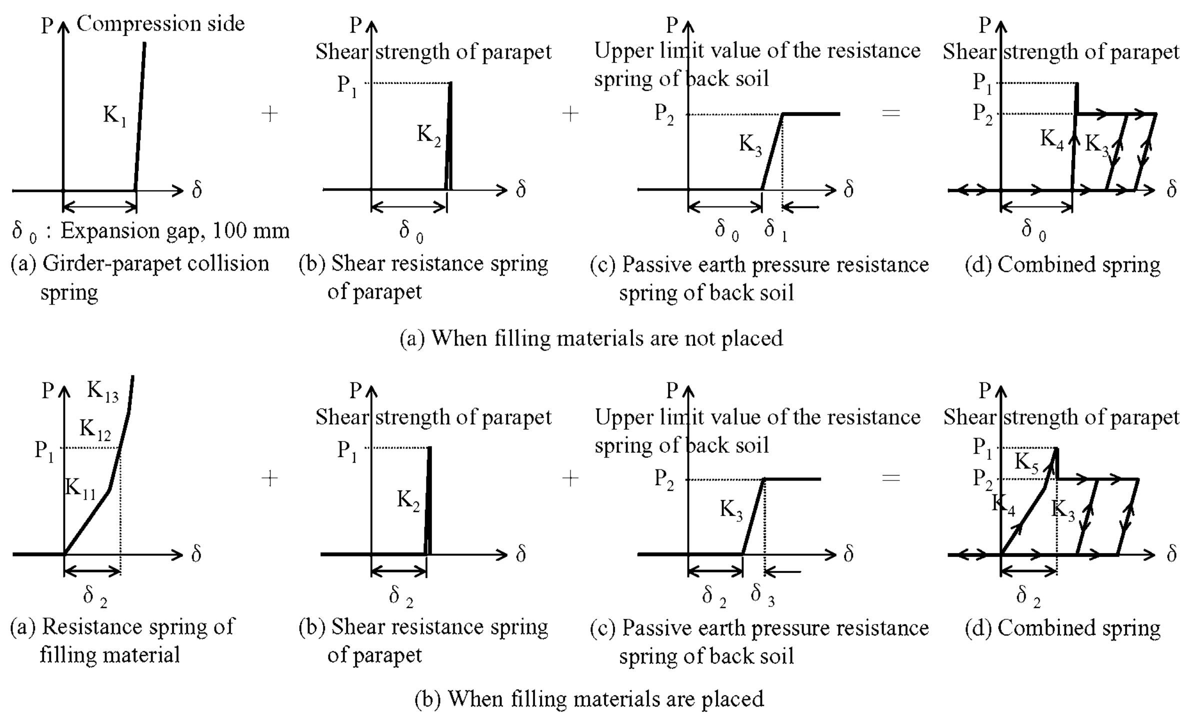

Fig. 9 Combined spring model of the abutment

Table 2 Analysis cases (No. of filling materials varies)

Analysis case	Case1	Case2	Case3	Case4
No. of filling materials	12	24	36	48
Area (m^2)	0.45	0.90	1.35	1.80
Compressive strain	60%	60%	60%	60%
Spring constant (MN/m)	75.0	150.0	225.0	300.0
Horizontal reaction (MN)	4.5	9.0	13.5	18.0
Horizontal displacement (m)	0.060	0.060	0.060	0.060
Primary natural period (sec)	0.516	0.431	0.378	0.340

* The primary natural period when filling materials are not placed: 0.686 (sec)

5. RESULTS OF ANALYSIS

5.1 Response Results Focusing on the Expansion Gap Amount

Firstly, to grasp the response characteristics of the existing bridge structure, analysis was conducted on the bridge behavior without placing filling materials and just focusing on the expansion gap amount. Figure 10 shows the relationship between the maximum response ductility factor of Pier 1 and the amount of expansion gap which is changed by the increment of 20 mm, taking 100 mm as the standard gap amount. It is known from the figure that, as the amount of expansion gap, or the amount a girder can move, becomes smaller, the maximum response ductility factor decreases. When the expansion gap is reduced to as small as 20 mm, it falls below the allowable ductility factor of the pier which is μ a = 4.299.

Figures 11 and 12 respectively show the relationship between response displacement and response velocity and the relationship between response displacement and response acceleration of the superstructure when the expansion gap amount is 100 mm. In Fig. 11, when the response displacement is immediately before 0.10 m which is equivalent to the expansion gap amount of 100 mm, the response velocity reaches a maximum velocity of 1.63 m/s. Likewise, in Fig. 12, when the response displacement is around 0.10 m, the response acceleration reaches a maximum value of 14.91 m/s^2. From these results, it is known that the response velocity reaches its maximum immediately before colliding against the parapet, and that the response acceleration reaches its maximum when collided against the parapet.

Figure 13 describes the relationship between the amount of expansion gap and the maximum energy and the maximum response acceleration of the superstructure. Here, the maximum energy of the superstructure means its maximum kinetic energy and it is calculated from Equation (4).

$$W = 1/2 \cdot m \cdot v^2 \tag{4}$$

where m : mass of the superstructure = 62400 kg
v : maximum response velocity of the superstructure

It is known from the figure that both the maximum energy and the maximum response acceleration of the superstructure become smaller as the amount of expansion gap reduces.

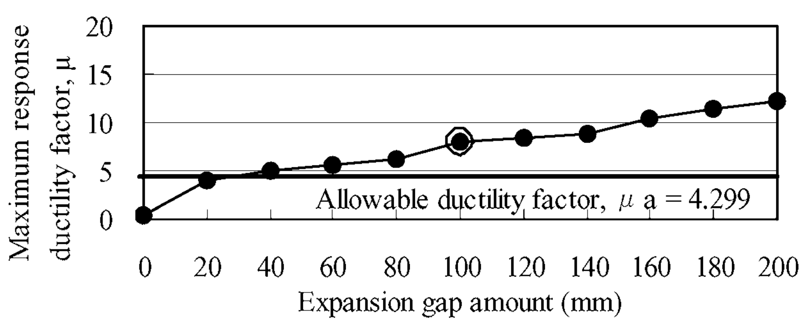

Fig. 10 Expansion gap amount and the maximum response ductility factor of Pier 1

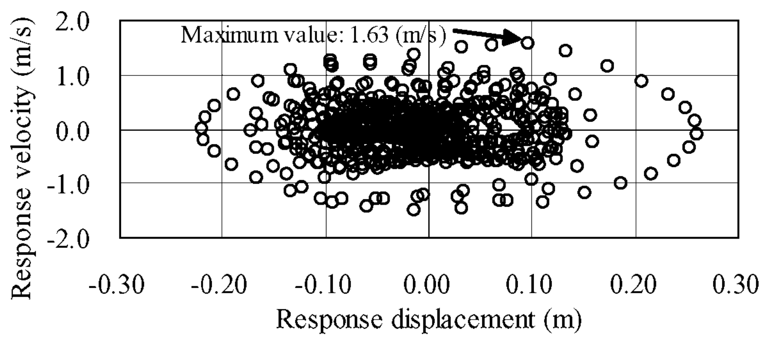

Fig. 11 Relationship between response displacement and response velocity

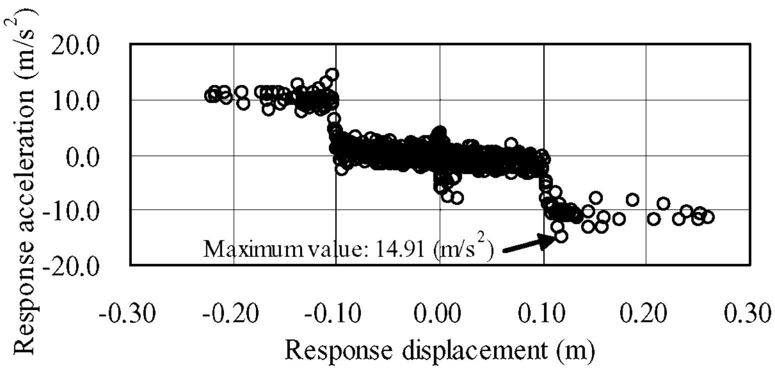

Fig. 12 Relationship between response displacement and response acceleration

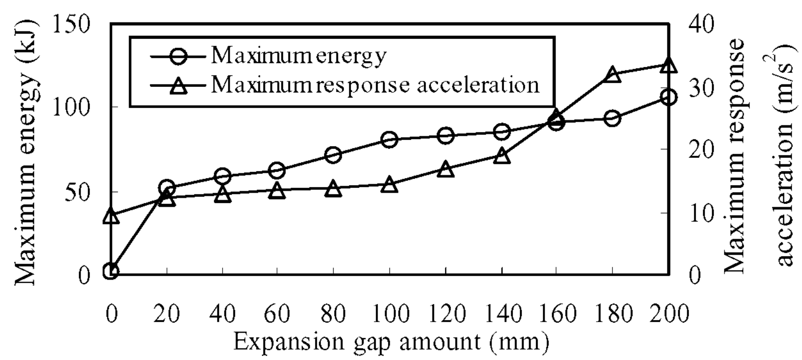

Fig. 13 Relationship between expansion gap amount and the maximum energy and maximum response acceleration of the superstructure

5.2 Response Results When Filling Materials are Taken as the Parameter

Here, the response results are presented which were obtained by taking the number of rubber filling materials placed at the 100 mm-wide standard expansion gap as the parameter.

Figure 14 shows the relationship between the axial direction rigidity of filling materials and the maximum response displacement of the superstructure. It is seen that as the axial direction rigidity of filling materials becomes larger, the maximum response displacement becomes smaller. Figure 15 shows the relationship between the number of filling materials placed and the maximum kinetic energy and the maximum response acceleration of the superstructure. It is found in the figure that, like the tendency of the maximum response displacement, as the axial direction rigidity of filling materials

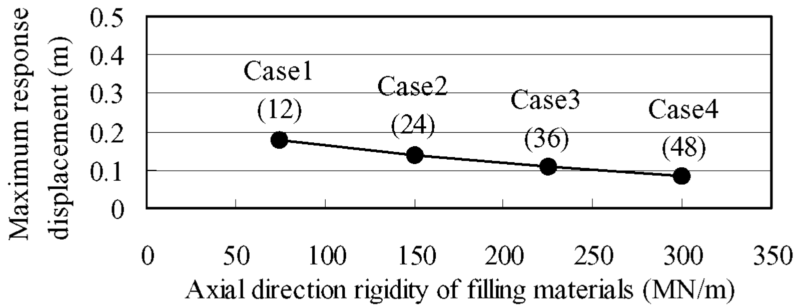

Fig. 14 Filling materials and the maximum response displacement of the superstructure

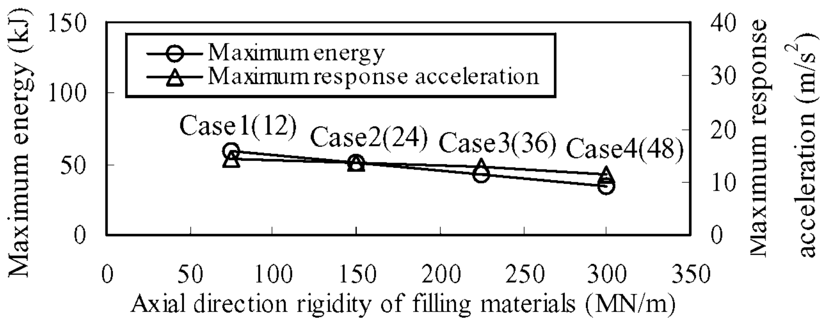

Fig. 15 Filling materials and the maximum energy and maximum response acceleration of the superstructure

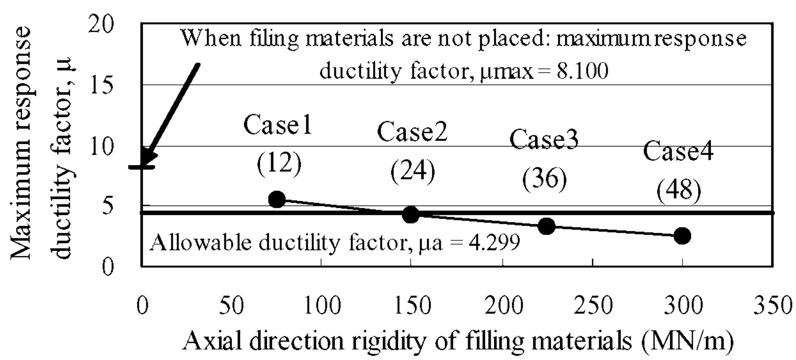

Fig. 16 Filling materials and the maximum response ductility factor of Pier 1

becomes larger, both the maximum kinetic energy and the maximum response acceleration of the superstructure become smaller. This is because the natural period of the entire bridge system becomes smaller due to the addition of the axial direction rigidity of filling materials, as shown in Table 2, and then the seismic response is reduced resultingly.

Figure 16 shows the relationship between the axial direction rigidity of filling materials and the maximum response ductility factor of Pier 1. The observed tendency is similar to that of the response displacement of the superstructure. When 24 filling materials are placed at the expansion gaps, the maximum response ductility factor falls below an allowable ductility factor of μ a = 4.299.

In the meantime, as shown in Fig. 17, it was confirmed that both the horizontal force acting on the bottom of the abutment under earthquake loading and the resulting bending moment fall below the respective allowable values which are back calculated from the stability calculation at the time of an earthquake, except for Case 1 in which 12 filling materials are placed.

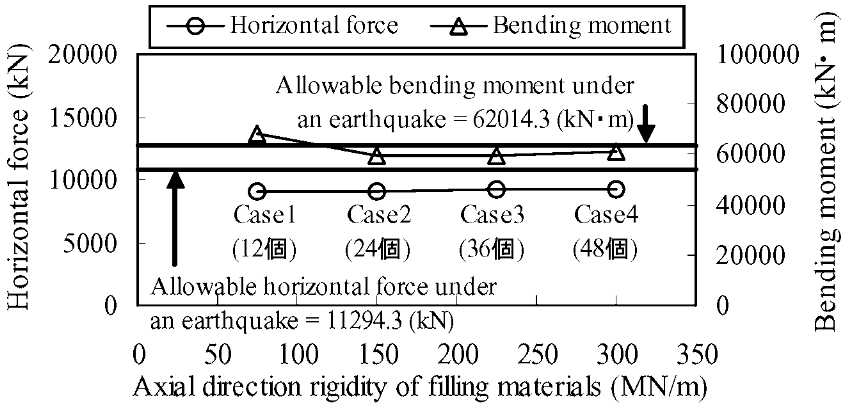

Fig. 17 Maximum force acting on the abutment under an earthquake

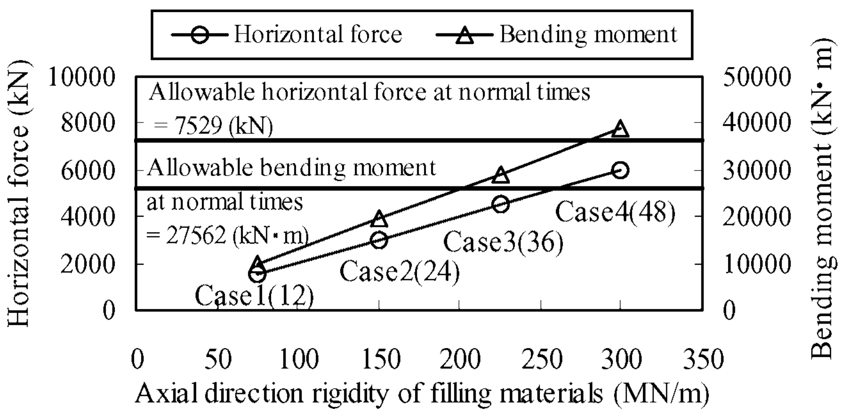

Fig. 18 Force acting on the abutment by the thermal expansion of the superstructure

5.3 Response against Displacement at Normal Times

Figure 18 shows the relationship between the axial direction rigidity of filling materials and the acting force at the bottom of Abutment 1 whose support structure is the movable type. To be specific, the relationship between the horizontal force acting at the bottom of the abutment and the resulting bending moment when the thermal expansion at normal times is about 20 mm, is described in this figure. A tendency is seen that both the horizontal force and bending moment become larger as the axial direction rigidity of filling materials increases. But, when compared with allowable values, all four cases are below the allowable horizontal force value which is back calculated from the allowable value in the stability calculation at normal times, but Case 3 and Case 4 exceed the allowable bending moment value.

From these results, it is can be said that a rational seismic strengthening is possible if filling materials that can satisfy both the seismic response of the pier and the stability calculation of the abutments are to be installed at the expansion gaps of the bridge.

However, as these results were obtained from just one kind of seismic waveform, the adequate number of filling materials may differ depending on the type of waveforms.

6. CONCLUSIONS

The following conclusions were drawn from the current study.

(1) When no filling materials are placed at the expansion gaps of girders, the maximum kinetic energy and the maximum response acceleration of the superstructure become smaller as the amount of expansion gap reduces, thus resulting in the smaller maximum response ductility factor of the pier.

(2) When filling materials are placed at the expansion gaps of girders, it was confirmed that, as the number of filling materials increases, which means as the axial direction rigidity of filling materials increases, the maximum response displacement of the superstructure and the maximum response ductility factor of the pier become smaller, hence enabling to reduce the seismic response.

(3) As the number of filling materials placed increases, the maximum kinetic energy and the maximum response acceleration of the superstructure become smaller, but the force acting on the abutments accompanying the displacement of girders at normal times will increase.

(4) A rational seismic strengthening is possible if filling materials that can satisfy both the seismic response of the pier and the stability calculation of the abutments are to be installed at the expansion gaps of the bridge.

What is needed hereafter before the practical application of this seismic strengthening method is to accurately evaluate the effect of deterioration of filling materials with time and the resistance characteristics of the soil behind the abutment through an experiment or by other means.

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