APPLICATION OF THE POWERFUL TMD AS A MEASURE FOR SEISMIC RETROFIT OF OLD BRIDGES

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SUMMARY

Authors have developed a seismic control device which can counter excessive deformation in bridges due to large-scale earthquakes, called the Powerful TMD. It provides higher damping than a conventional TMD by utilizing the principle of lever. Originally, it was developed to be applied to newly designed bridges. But it is possible to apply the device to an old bridge for increasing apparent damping of the bridge as a measure for seismic retrofit. In this paper, authors conducted non-linear earthquake response analyses of two existing bridges to examine an application of the device to old bridges which require seismic retrofitting. As a result, it was proved that the device could be available as a measure for the seismic retrofit of old bridges.

INTRODUCTION

The Powerful TMD is a seismic control device which was developed in order to improve the damping capability of a conventional TMD considerably. The device is attached to a bridge girder and is compulsorily shaken by the reaction force from a bridge abutment or a bridge pier without any external energy. It was developed to be applied to newly designed bridges. Formulation of equation of motion, optimum tuning and numerical simulations were conducted rigorously for a 2-DOF system [Kaneko et al., 1994]. Shaking table tests using a simple bridge model were also carried out to verify the validity of formulation and the seismic control effect for the Powerful TMD [Kaneko et al., 1995].

After the Hyogoken-Nanbu earthquake of 1995, major measures for seismic retrofit of old bridges include use of high damping rubber bearings and reinforcement of bridge piers with steel jackets. When high damping rubber bearings are applied to an old bridge constructed on soft ground whose natural frequency is more than 0.6 seconds, Type III ground condition [Japan Road Association, 1996], the bridge is in danger of resonance. In this case, the Powerful TMD performs better. On the other hand, when seismic retrofit work of an old bridge pier constructed in the water is carried out, temporary coffering work should be conducted first before strengthening piers. In general, the cost needed for coffering is much more expensive than that for strengthening work itself. The Powerful TMD is preferable in this case as well, because it does not need temporary coffering work. Thus, it has become urgently necessary to examine if the Powerful TMD might replace the seismic retrofit of old bridges.

In response to such necessity, authors examined an application of the device to old bridges which require seismic retrofitting. Numerical analyses were conducted and methods to attach the device to old girders were examined, selecting existing bridges as subject. In this paper, the outline and design method of the device are described first. Then, two examples of seismic retrofit design for two different types of existing old bridges using the Powerful TMD are presented.

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OUTLINE AND DESIGN METHOD OF THE POWERFUL TMD

OUTLINE OF THE POWERFUL TMD

Figure 1 illustrates a schematic diagram to represent the function of the Powerful TMD. The device should be placed at a bridge girder which is close to an abutment or a pier, possesing movable bearings. Although the concept of the device is based on conventional passive TMD, a TMD system is not simply fixed at a bridge deck directly but is fixed at a steel frame which is able to slide freely on a bridge deck in the longitudinal direction. The steel frame is connected to an abutment through a lever.

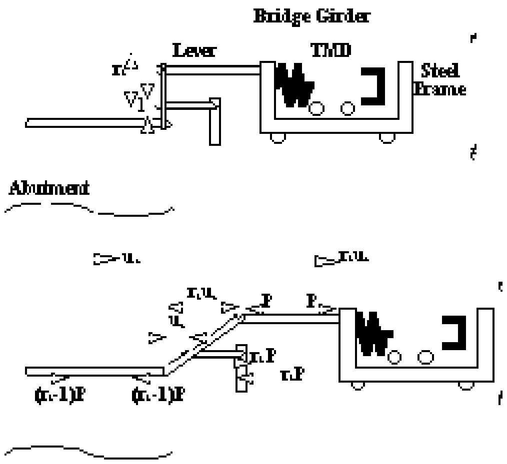

Figure 1: A schematic diagram to represent the function of the Powerful TMD

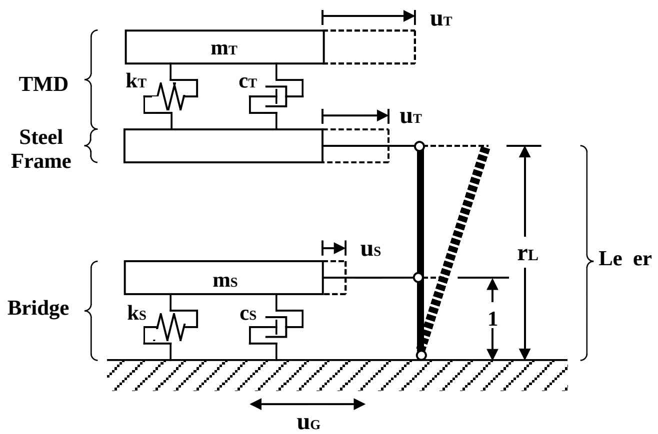

Figure 2: A schematic diagram to represent the dynamic model for the Powerful TMD

If the arm ratio of the lever is r_L and the relative displacement between a bridge deck and a abutment due to earthquake excitation is u_S as shown in the figure, the TMD is compulsorily shaken with an amplitude of $r_L u_S$, which is r_L times larger than that of a bridge deck. When the control force, P is applied by the TMD excitation, it is transmitted to the bridge deck after being amplified by r_L times. Therefore, we can obtain a seismic control force r_L times larger than that obtained by a conventional TMD without any external energy adoption. The dynamic model for the Powerful TMD is illustrated in Figure 2.

DESIGN METHOD OF THE POWERFUL TMD

The design of the Powerful TMD is based on the optimum tuning for harmonic excitation [Den Hartog, 1956]. The optimum frequency ν_{opt} is calculated using Eq.(1). And the optimum damping h_{opt} is calculated using Eq.(2).

$$\nu_{opt} = \frac{1}{1 + r_L^2 \mu} \tag{1}$$

$$h_{opt} = \sqrt{\frac{3r_L^2\mu}{8(1+r_L^2\mu)}} \tag{2}$$

where, μ is the ratio of mass of a TMD to that of a bridge structure. An increment of apparent damping of a bridge structure due to the device is evaluated by the effective damping ratio h_{eff}. This is defined as follows:

$$h_{eff} = c_h \cdot \frac{h_T r_L^2 \mu \nu}{(\nu^2-1)^2 + (2h_T\nu)^2} \tag{3}$$

where, c_h is the parameter which depends on the irregularity of earthquakes (=0.7), ν is the ratio of natural frequency of a TMD to that of a bridge structure, and h_T is the damping ratio of the TMD. Figure 3 shows the variations of effective damping ratio of the optimized device with respect to mass ratio μ for six different values of the arm ratio r_L.

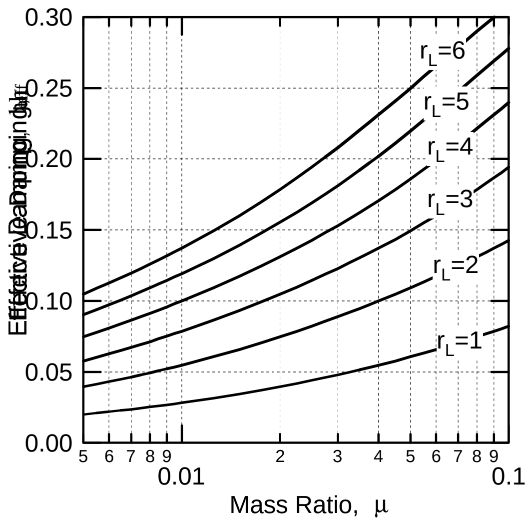

Figure 3: Variations of effective damping ratio of the optimized device with respect to mass ratio μ

EXAMPLES OF THE SEISMIC RETROFIT DESIGN

EXAMPLE-1 (BRIDGE-1)

The outline of Bridge-1 is shown in Table 1, and Figure 1 is its elevation. The seismic retrofit design for this bridge has been conducted based on "Reference for Applying the Guide Specifications for Reconstruction and Repair of Highway Bridges which Suffered Damage in the Hyogoken Nanbu Earthquake, to New Highway Bridges and Seismic Strengthening" [Japan Road Association, 1995]. At first, reinforcing the bridge piers with RC jackets was adopted as a measure for seismic retrofit. But temporary coffering work had to be conducted first before strengthening of piers because piers with fixed bearings had been constructed in the water. The cost needed for coffering work is much more expensive than that for strengthening work itself. And then, applying the seismic control device to the bridge was examined as a substitute to the first measure of seismic retrofit. Parameters for the device are shown in Table 2.

Table 1: Outline of Bridge-1

Type of superstructure	Gerber-type steel plate girder bridge (connected)
Length	40 m + 60 m + 40 m = 140 m
Width	Roadway 8.0 m + Sidewalk 2 @ 1.5 m
Weight of superstructure	12010 kN
Type of piers	RC oval shaped pier, Height : 10.85 m
Type of foundation	Steel pile, ϕ 600 mm X L = 54.0 m
Type of soil	Type III ground condition
Year of construction	1972

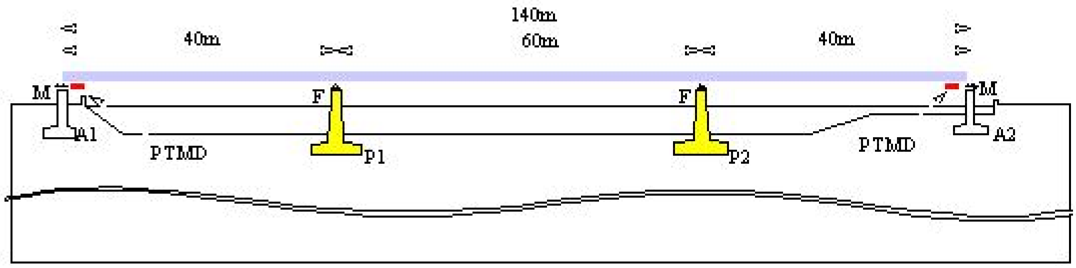

Figure 4: Bridge-1 elevation

Table 2: Parameters for the device for Bridge-1 (Total)

Lever	TMD		
Arm ratio of the lever	Mass ratio μ	Natural frequency ratio ν_{opt}	Damping ratio h_{opt}
5.0	0.01	0.800	0.274

The devices were installed on two abutments with movable bearings (A1, A2) as shown in Figure 4. From the results of the non-linear earthquake response analyses using earthquake inputs for Type III ground condition [Japan Road Association, 1996], it was confirmed that the bridge with the device was safe during a Level 2 Earthquake input [Japan Road Association, 1996].

The load-displacement diagram for pier P1 is shown in Figure 5. The allowable ductility factor μ of this pier is calculated at the time when the concrete cover of this pier comes off, since it has been assumed that fixing of the hoops of this pier was not enough. Figure 6 shows one of the results of the numerical analyses. As shown in this figure, the maximum displacement , when the control device is used, is almost the yield displacement δ_y, although that without using the control device is over the allowable displacement $\mu\delta_y$. And the residual displacement of the structure with the device is close to zero. At last, the structural drawing of the seismic control device designed for Bridge-1 is shown in Figure 7.

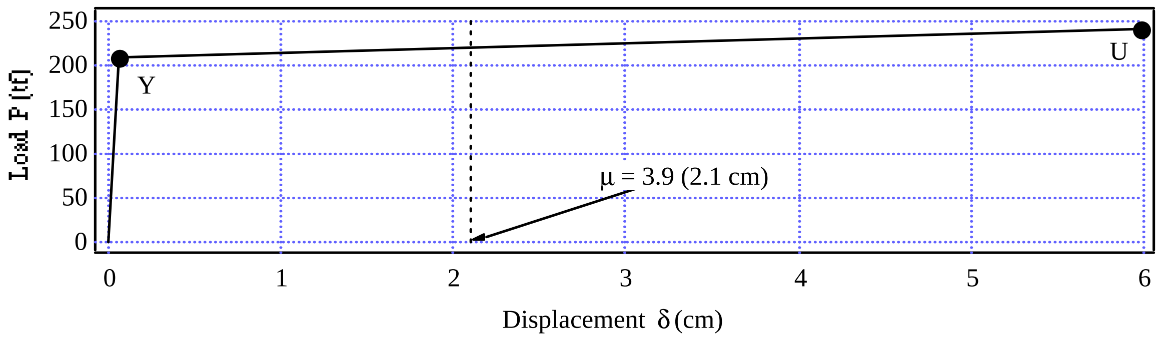

Figure 5: Load-displacement diagram for pier P1 of Bridge-1

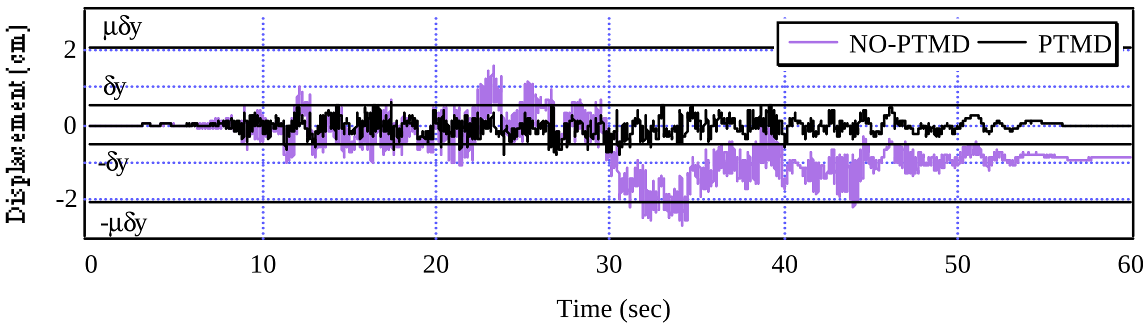

Figure 6: Comparison of the time histories of the displacement of the bride with and without the device (Type-1, Type III ground condition, No.2)

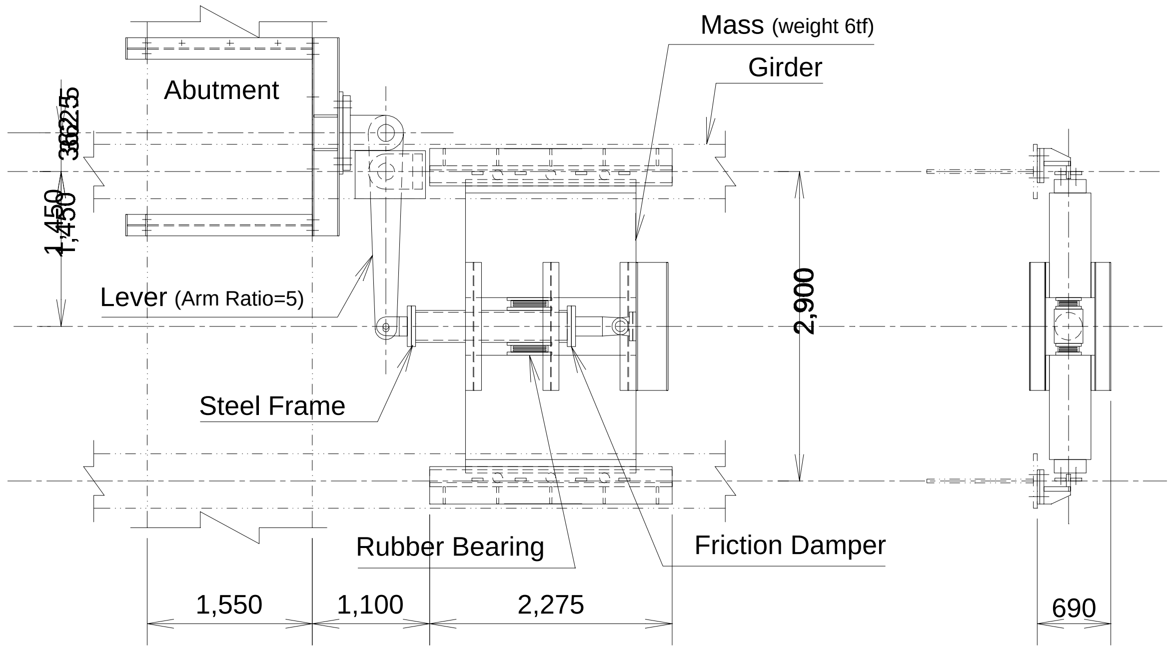

Figure 7: Structural drawing of the seismic control device designed for Bridge-1

EXAMPLE-2 (BRIDGE-2)

Bridge-2 has been built over the canal dividing the plant site into two areas. It is important to resume operations early after a large earthquake. So the seismic retrofit design for this bridge has been conducted based on "Design Specifications of Highway Bridges - Part V Seismic Design" [Japan Road Association, 1996].

The outline of Bridge-2 is shown in Table 3, and Figure 8 shows its elevation. The structure of pier P2 is double-layer RC rigid-frame. At first, the ultimate horizontal strength during an earthquake of this pier was checked by conducting static non-linear analysis. Figure 9 shows the analytical model. From the study, it was confirmed that the pier is safe if shearing strength of two piers of the first layer is reinforced. The load-displacement diagram for pier P2 is shown in Figure 10.

Table 3: Outline of Bridge-2

Type of superstructure	Three span continues steel box girder bridge
Length	60.6 m + 80.0 m + 61.0 m = 201.6 m
Width	Roadway 10.0 m
Weight of superstructure	18110 kN
Type of pier	Double-layer RC rigid-frame pier, Height : 10.85 m
Type of foundation	Footing
Type of soil	Type I ground condition
Year of construction	1965

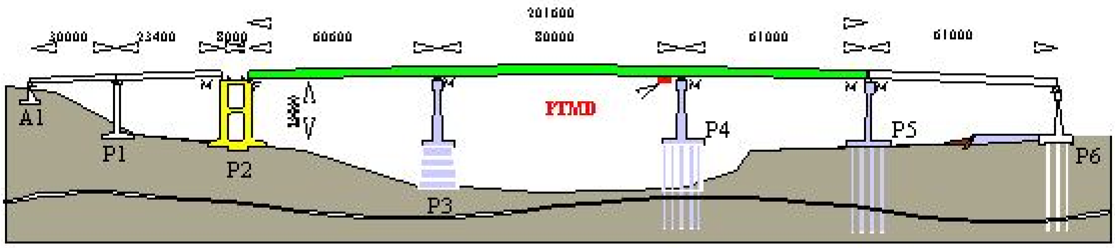

Figure 8: Bridge-2 elevation

Furthermore the non-linear earthquake response analysis using earthquake inputs for Type I ground condition was conducted, since this bridge is important and an unique structure. It was found that the maximum displacements depended on the characteristics of the phase of those inputs, and in one case, that the value was over the allowable displacement. So the seismic control device installed on the bridge was examined. Parameters for the device are shown in Table 4. From the results of the non-linear earthquake response analyses, it was confirmed that the bridge with the device was safe, though the mass ratio (μ=0.003) was smaller than general mass ratio (μ=0.01). For example, the comparison of time histories of displacement of the bridge girder with and without the device is shown in Figure 11. At last, the structural drawing of the seismic control device designed for Bridge-2 is shown in Figure 12.

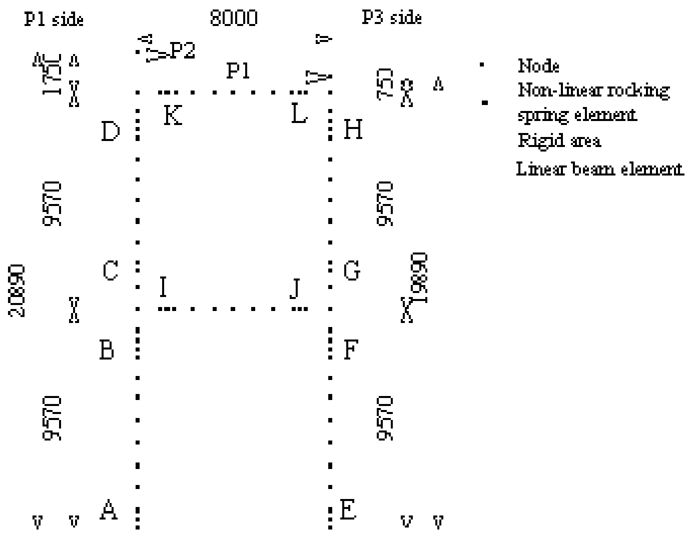

Figure 9: Analytical model for pier P2 of Bridge-2 for static non-linear analysis

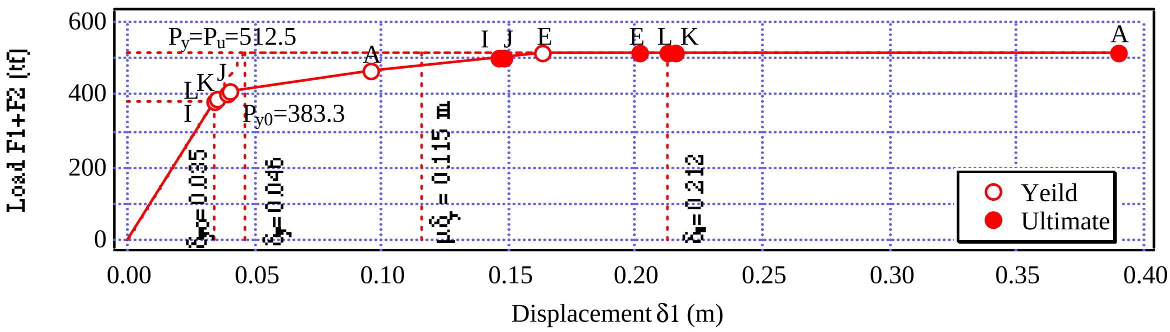

Figure 10: Load-displacement diagram for pier P2 of Bridge-2

Table 4: Parameters for the device for Bridge-2

Lever	TMD		
Arm ratio of the lever	Mass ratio μ	Natural frequency ratio ν_{opt}	Damping ratio h_{opt}
5.0	0.003	0.924	0.169

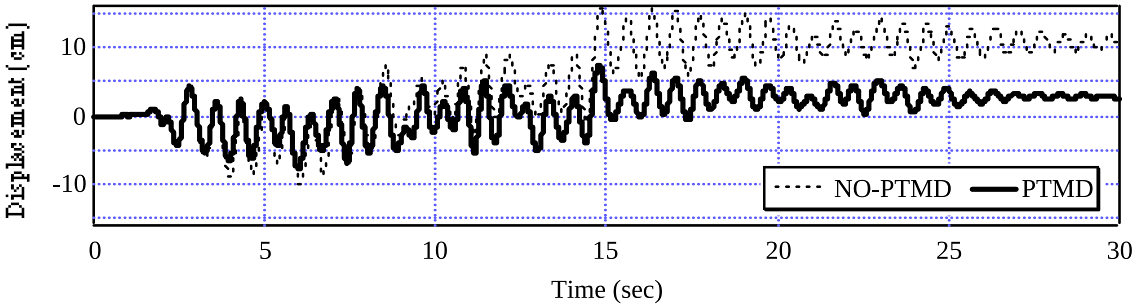

Figure 11: Comparison of the time histories of displacement of the bride girder with and without the device (Type-1, Type I ground condition, No.1)

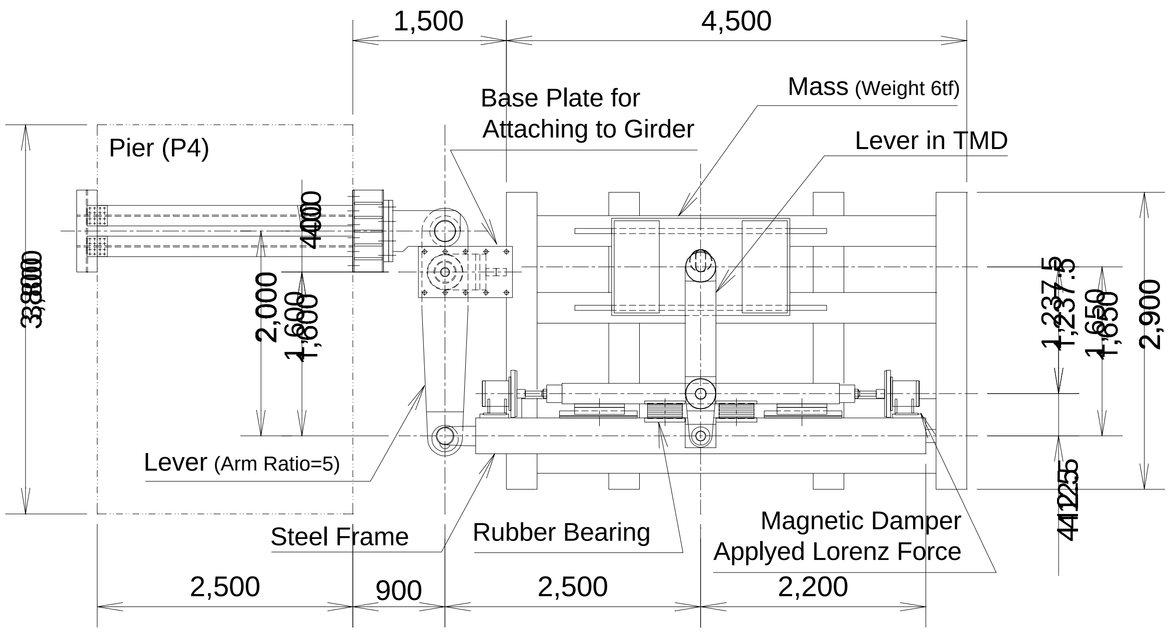

Figure 12: Structural drawing of the seismic control device designed for Bridge-2

CONCLUSIONS

In this paper, a measure for seismic retrofit of existing old bridges using the seismic control device is introduced with some examples of seismic retrofit design. The measure makes use of the effectiveness of damping of the device, so that it reduced not only the acceleration of bridge girder but displacement also. That is why this measure is a good application to existing old bridges, since it restricts the displacement within the allowable limit of the girder. Especially, since the displacements of bridges constructed on Type III ground condition are large, the measure is expected as a substitute for seismic isolation.

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