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# STUDY ON MODELING OF STEEL RIGID FRAME BRIDGE FOR DYNAMIC ELASTO-PLASTIC ANALYSIS

**Tsutomu YOSHIZAWA[1], Masaru NARITOMI[2], Osamu ISHIBASHI[3] And Masahide KAWAKAMI[4]**


## SUMMARY

The modeling method of the inelastic behavior of steel members is important to analyze the response of a steel structure under strong seismic loading. In this paper, dynamic elasto-plastic analyses of a rigid frame bridge were carried out and the effects of the modeling method of inelastic steel pier elements are discussed. A three span continuous steel rigid frame bridge is chosen for the analysis. The superstructure, a two-box girder steel deck, is supported on longitudinally sliding shoes at the end piers while it is rigidly connected to the internal piers. The piers have stiffened thin-walled single box section and are modeled using elasto-plastic beam elements in the analysis. Three types of modeling method were applied to express the plastic behavior of stiffened steel box section of the piers. The first is the Hanshin Expressway Public Corporation Model. The second is the Two-Parameter Model developed by USAMI. And the third is the Japan Railways Railway Technical Research Institute Model.


## INTRODUCTION

The elastic design of a bridge against a strong earthquake with less occurring probabilities during bridge operation period is considered to be very uneconomical. Therefore, in the bridge design against strong earthquake, the plastic deformation should be allowed and the demand performance should be defined as the capacity of the plastic behavior. With this design, the calculation accuracy of the elasto-plastic dynamic analysis of the bridge must be important.

To calculate the plastic behavior of the bridge, the modeling method is important. In the analysis of the reinforced concrete piers and the steel piers filled with concrete, the modeling methods are in the process of the standardization based on the experiment results. In the analysis of the empty thin-walled steel piers, the effect of the local buckling is not ignored and the plastic behavior of the steel piers is complex. The inelastic behavior is varying with the scale of the section, the thickness of the plates, the distribution of the stiffeners and other items.

To prove the characteristics of the elasto-plastic behavior of empty thin-walled steel piers, with some single column pier models, the cyclic loading experiments and the analyses of Finite Element Method considering the geometrical nonlinearity and the material nonlinearity are carried out and the results are accumulating, nowadays. Several equations to estimate the nonlinear behavior of the steel pier are presented.

In this paper, three modeling methods were selected. The first is the Hanshin Expressway Public Corporation Model. The second is the Two-Parameter Model developed by USAMI. And the third is the Japan Railways Railway Technical Research Institute Model. The elasto-plastic dynamic analyses were carried out to calculate the response of the steel rigid frame bridge under a strong earthquake loading. And the effects of the three modeling methods are discussed.


[1] *Nippon Engineering Consultants Co., Ltd., 5-1 Shichiza-Cho, Koshigaya-Shi, Saitama-Ken, Japan, yoshizawa@ne-con.co.jp*
[2] *Kyushu Kyoritsu Univ., 1-8 Jiyugaoka, Yahatanishi-Ku, Kitakyushu-Shi, Japan, naritomi@kyukyo-u.ac.jp*
[3] *Daiichi Fukken Consultant Co., Ltd., 3-5-28 Hkata-Eki-Minami, Hakata-Ku, Kitakyushu-Shi, Japan, o_ishibashi@dfk.co.jp*
[4] *Nippon Engineering Consultants Co., Ltd., Koshigaya-Shi, Saitama-Ken, Japan, kawakami_masa@ne-con.co.jp*

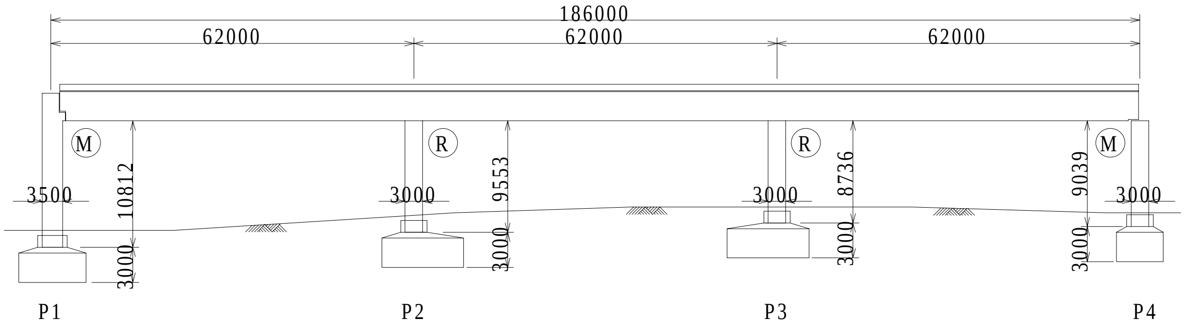



**Fig. 1 : Shape of bridge**

**Table 1 : M-Phi relations of piers of Hanshin Expressway Public Corporation Model**

| Direction of earthquake input | | | Longitudinal analysis | | Transverse analysis | | |
|---|---|---|---|---|---|---|---|
| Pier number | | | P2, P3 | | P1 | P2, P3 | P4 |
| Section of pier | | | Top | Bottom | Bottom | Bottom | Bottom |
| Compression yield | $M_{y0}$ | (kN·m) | 95549 | 64678 | 72862 | 52112 | 49430 |
| | $\phi_{y0}$ | (1/m) | 0.00132 | 0.00124 | 0.00197 | 0.00187 | 0.00196 |
| Tension yield | $M_y$ | (kN·m) | 110148 | 79179 | 82979 | 62427 | 56639 |
| | $\phi_y$ | (1/m) | 0.00174 | 0.00193 | 0.00297 | 0.00341 | 0.00296 |
| Ultimate strength | $M_u$ | (kN·m) | 122967 | 86452 | 86872 | 64889 | 59500 |
| | $\phi_u$ | (1/m) | 0.01256 | 0.01172 | 0.01717 | 0.01600 | 0.01728 |

## ANALITICAL MODEL

### Description of Bridge

The bridge selected for this study is shown in **Fig. 1**. The bridge is a three-span continuous steel rigid frame bridge. The two internal piers are connected to the girder rigidly. The shoes to allow the longitudinal sliding and to fix the transverse sliding are set on the top of two end piers. The superstructure has a two-box girder and a steel deck. The piers have the stiffened thin-walled single box section. The ground type in seismic design of this bridge is Type I [Japan Road Association, 1996], and the spread foundation was employed. In the modeling of this bridge, the thickness of the piers were reduced to show the differences of the modeling methods.

The analytical models were composed with the three dimensional frame elements. And the same models were used in the longitudinal analysis and the transverse analysis. The four piers were all modeled and the connection conditions of sliding shoes(longitudinal-sliding, transverse-fixed) were represented at the top of the end piers. The piers were modeled as the plastic elements. The superstructure was modeled as the elastic element. The boundary conditions at the bottom of the piers were fixed.

### Elasto-Plastic Model of Pier Elements

Below are three modeling methods employed in the elasto-plastic analyses to calculate the behavior of the stiffened thin-walled box sections.

#### *Model 1 Hanshin Expressway Public Corporation Model*

The Equivalent Section Method [Kitazawa, Horie and Nishida, 1997] developed by the Hanshin Expressway Public Corporation was employed. In this method, the stiffened section is substituted to the equivalent non-stiffened box section. And the tri-linear M-Phi relation is calculated for this section under the plane holding assumption. The regular tri-linear loop was adopted as the hysteresis model.

The pier elements of the analytical model are substituted to the elasto-plastic beam elements in using this

**Table 2 : P-Delta relations of piers with Two Parameters Model (longitudinal analysis)**

| Pier number | | | P2 | | P3 | |
|---|---|---|---|---|---|---|
| Section of pier | | | Top | Bottom | Top | Bottom |
| Equivalent pier height | $H$ | (m) | 4.168 | 5.368 | 3.768 | 4.968 |
| Pier axial force | $P$ | (kN) | 11768 | 12562 | 11768 | 12484 |
| Width-thickness ratio | $R_f$ | | 0.559 | 0.769 | 0.559 | 0.769 |
| Relative stiffness ratio | $\gamma/\gamma^*$ | | 1.72 | 2.95 | 1.72 | 2.95 |
| Yield strength | $H_y$ | (kN) | 20770 | 9640 | 20770 | 10434 |
| Yield displacement | $\delta_y$ | (m) | 0.0158 | 0.0165 | 0.0145 | 0.0151 |
| Maximum strength | $H_m$ | (kN) | 53948 | 17431 | 63779 | 19635 |
| Corresponding displacement | $\delta_m$ | (m) | 0.0841 | 0.0522 | 0.0849 | 0.0492 |
| Ultimate strength | $H_{95}$ | (kN) | 51250 | 16560 | 60590 | 18653 |
| Ultimate displacement | $\delta_{95}$ | (m) | 0.1276 | 0.0808 | 0.1279 | 0.0757 |

**Table 3 : P-Delta relations of piers with Two Parameters Model (transverse analysis)**

| Pier number | | | P1 | P2 | P3 | P4 |
|---|---|---|---|---|---|---|
| Section of pier | | | Bottom | Bottom | Bottom | Bottom |
| Pier height | $H$ | (m) | 12.012 | 10.735 | 9.936 | 10.239 |
| Pier axial force | $P$ | (kN) | 11072 | 12562 | 12484 | 8032 |
| Width-thickness ratio | $R_f$ | | 0.648 | 0.769 | 0.769 | 0.879 |
| Relative stiffness ratio | $\gamma/\gamma^*$ | | 1.29 | 2.91 | 2.91 | 2.88 |
| Yield strength | $H_y$ | (kN) | 5335 | 3903 | 4227 | 3579 |
| Yield displacement | $\delta_y$ | (m) | 0.0943 | 0.0659 | 0.0577 | 0.0593 |
| Maximum strength | $H_m$ | (kN) | 6559 | 4766 | 5276 | 4266 |
| Corresponding displacement | $\delta_m$ | (m) | 0.2590 | 0.1773 | 0.1562 | 0.1575 |
| Ultimate strength | $H_{95}$ | (kN) | 6325 | 4531 | 5011 | 4050 |
| Ultimate displacement | $\delta_{95}$ | (m) | 0.4166 | 0.2849 | 0.2506 | 0.2545 |

modeling method. The beam elements are divided in short length and the M-Phi relations are adopted to the divided sections. In this modeling method, the M-Phi relations of the sections are independent of the distribution of the moments along the beam length, and this method has no problem in applying for the rigid frame structure. The effect of the local buckling can not be considered. Likewise, the strain hardening was ignored in this modeling. The M-Phi relations of the piers in Model 1 are shown in **Table 1**.

***Model 2 Two-Parameter Mode***

The modeling method developed by USAMI[JSCE, 1996.] was employed. In this method, the nonlinear response of the steel piers are expressed with the P-Delta relations of a simple column pier. The P-Delta relations of the analytical model are shown in **Table 2** and **Table 3**. The regular tri-linear loop was adopted as the hysteresis model instead of the loop of the Two Parameter Model because of the limitation of the analytical program.

***Model 3 Japan Railways Railway Technical Research Institute Model***

The method developed by the Japan Railways Railway Technical Research Institute[JR RTRI, 1997.] was employed. In this method, the equations to calculated the yield strength are the same as Model 2. The equations of maximum strength and the reduction of the strength after maximum strength are different from Model 2. The P-Delta relations of the analytical model are shown in **Table 4** and **Table 5**. The regular tri-linear loop was adopted as the hysteresis loop.

Because modeling methods of Model 2 and Model 3 are developed for single column piers, to apply the methods to the inplane analysis of rigid frame structure, it is necessary to examine the modeling method to explain the behavior of the pier element with the moment distribution under the seismic loading.

In this paper, to consider the rigid frame effect, the simple method proposed by Maeno[Maeno, Inagaki, Kawano

**Table 4 : P-Delta relations of piers with JR RTRI Model (longitudinal analysis)**

| Pier number | | | P2 | | P3 | |
|---|---|---|---|---|---|---|
| Section of pier | | | Top | Bottom | Top | Bottom |
| Yield strength | $H_y$ | (kN) | 20770 | 9640 | 20770 | 10434 |
| Yield displacement | $\delta_y$ | (m) | 0.0158 | 0.0165 | 0.0145 | 0.0151 |
| Maximum strength | $H_m$ | (kN) | 69693 | 17832 | 52484 | 29886 |
| Corresponding displacement | $\delta_m$ | (m) | 0.1317 | 0.2800 | 0.1254 | 0.3823 |
| Ultimate strength | $H_n$ | (kN) | 66207 | 16940 | 49861 | 38392 |
| Ultimate Displacement | $\delta_n$ | (m) | 0.1861 | 0.1227 | 0.1777 | 0.5317 |

**Table 5 : P-Delta relations of piers with JR RTRI Model (transverse analysis)**

| Pier number | | | P1 | P2 | P3 | P4 |
|---|---|---|---|---|---|---|
| Section of pier | | | Bottom | Bottom | Bottom | Bottom |
| Yield strength | $H_y$ | (kN) | 5335 | 3903 | 4227 | 3579 |
| Yield displacement | $\delta_y$ | (m) | 0.0943 | 0.0659 | 0.0577 | 0.0593 |
| Maximum strength | $H_m$ | (kN) | 8408 | 6103 | 6676 | 5520 |
| Corresponding displacement | $\delta_m$ | (m) | 0.4271 | 0.2823 | 0.2521 | 0.2414 |
| Ultimate strength | $H_n$ | (kN) | 7988 | 5798 | 6341 | 5244 |
| Ultimate Displacement | $\delta_n$ | (m) | 0.5647 | 0.3623 | 0.3244 | 0.3136 |

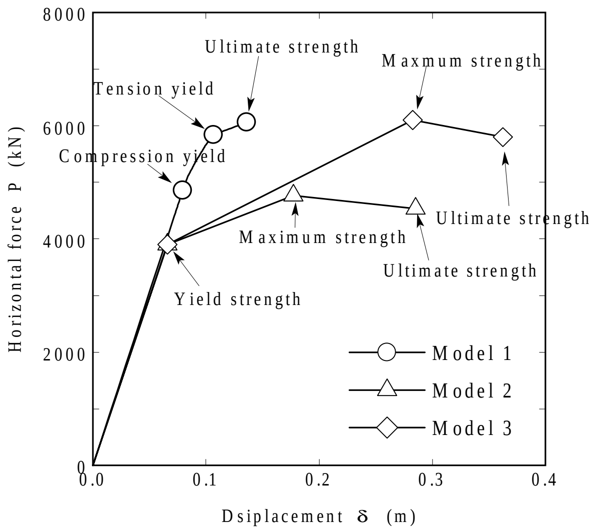



**Fig 2 : P-Delta relations of P2 in transverse loading**

and Ikeda 1998] was adopted to the longitudinal (inplane) model. The equivalent pier height is defined as the length from the middle height of the piers to the assumed plastic hinge (the bottom and the top of the piers). The P-Delta relations of the equivalent single column piers were calculated and the elasto-plastic rotational spring of the plastic hinges were defined.

***P-Delta relations of the Pier***

The P-Delta relations of the P2 calculated with these three modeling methods are shown in **Fig. 2.**

In the comparison of the P-Delta relations, the yield strength of Model 1 is the highest. And reduction of the stiffness after yielding is relatively small. The yield strengths of Model 2 and Model 3 are relatively low and the reduction of the stiffness after yielding are evident because of the effect of local buckling. And, the maximum strengths are less than Model 1.

The ultimate displacement of Model 1 is 0.14m and is the smallest. On the other hand, the ultimate state was defined as the strength reduced to 95% of the maximum strength, the ultimate displacements of Model 2 and Model 3 are 0.285m and 0.362m. The estimated plastic deformation capacity seems to be relatively large with

**Table 6 : Damping coefficient of bridge elements**

| Bridge elements | | Damping coefficient $h$ |
|---|---|---|
| Girder | | 0.02 |
| Pier | Elastic elements | 0.03 |
| | Elasto-Plastic elements | 0.01 |

**Table 7 : Maximum response of P 2 (longitudinal analysis)**

| | | Model 1 | Model 2 | Model 3 |
|---|---|---|---|---|
| Acceleration of girder | (gal) | 1241.6 | 1307.7 | 1040.2 |
| Displacement of girder | (m) | 0.115 | 0.127 | 0.121 |
| Moment of pier top | (kN·m) | 105843 | 93604 | 85700 |
| Shearing force of pier top | (kN) | 20780 | 19417 | 17279 |
| Moment of pier bottom | (kN·m) | 94340 | 93536 | 80836 |
| Shearing force of pier bottom | (kN) | 21104 | 19201 | 17574 |

**Table 8 : Maximum response of P 2 (transverse analysis)**

| | | Model 1 | Model 2 | Model 3 |
|---|---|---|---|---|
| Acceleration of girder | (gal) | 913.2 | 770.4 | 824.4 |
| Displacement of girder | (m) | 0.285 | 0.244 | 0.246 |
| Moment of pier bottom | (kN·m) | 68548 | 52868 | 61949 |
| Shearing force of pier bottom | (kN) | 7120 | 5305 | 6315 |

Model 2 and Model 3.

### Analytical Method

The standard earthquake wave of the Specifications for Highway Bridges on the Type I ground[Japan Road Association, 1997] was used. The earthquake strength is in rank with the 1995 Hyougo-ken Nanbu earthquake. In the Elasto-Plastic dynamic analysis, the direct integration with the Newmark's beta method was used. The beta value was 0.25, and the time interval of the integration was 0.001sec. The damping coefficient of the bridge elements are shown in **Table 6**.

## RESULT OF ANALYSES

### Maximum Response

#### *Longitudinal analysis*

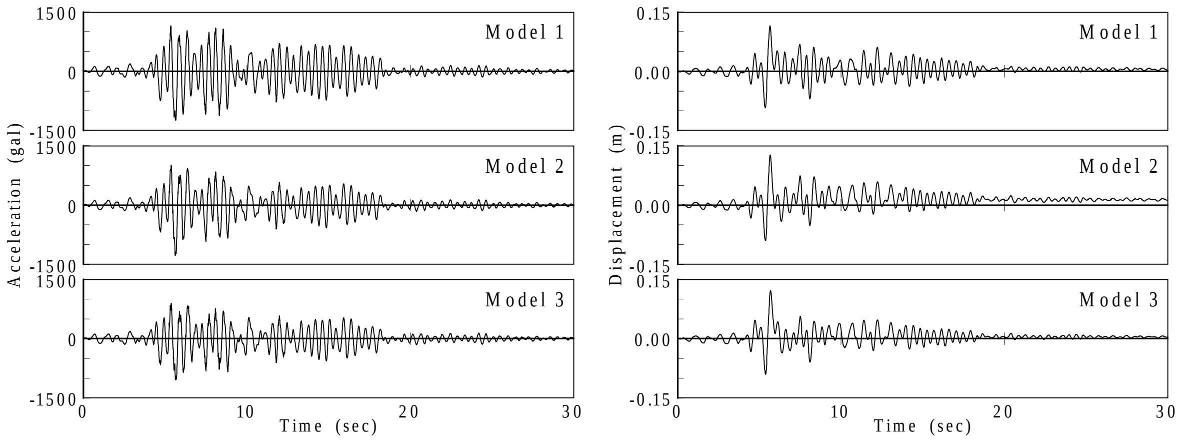
The maximum response of the longitudinal analysis is shown in **Table 7**. The response of the P2 is shown on behalf of the piers. The stiffness of the bridge is relatively high because of the rigid frame structure characteristics. The maximum response displacement of the girder was 0.127m (Model 2), and the response acceleration reached 1300gal (Model 2).

The response acceleration and the displacement of Model 2 were superior compared to the other cases. These seems to be caused by the strength reduction of the bottom of the P2.

The moment and the shearing force of Model 1 were larger than the other cases instead of the less response displacement. It is thought that these were caused by the high yield strength and the little stiffness reduction after yielding shown in **Fig. 2**.

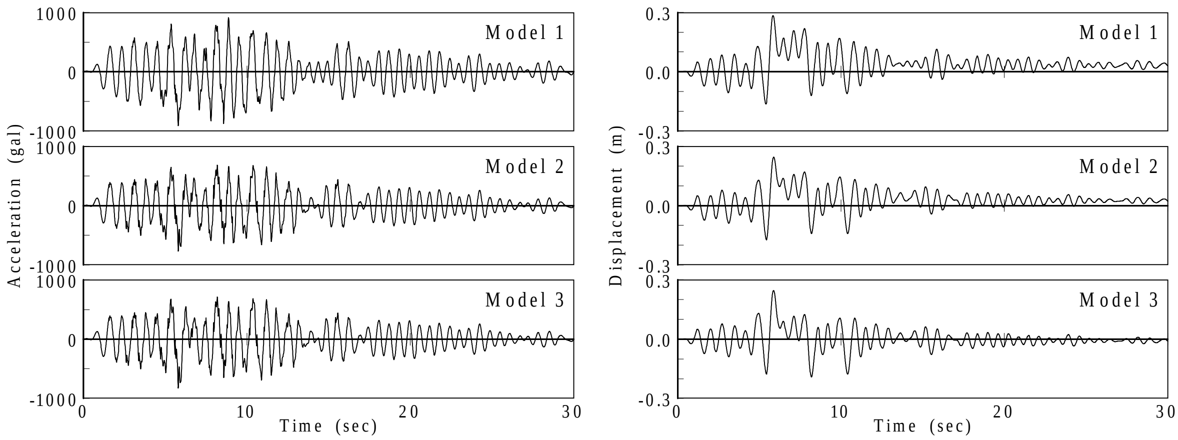
#### *Transverse analysis*

The maximum response of the transverse analysis is shown in **Table 8**. In the transverse analysis, the structure characteristics of the piers is the same as the single column pier, and the stiffness is relatively low. Hence, the




(a)Acceleration of girder (b)Displacement of girder

**Fig. 3 : Response waves of girder at P2 (longitudinal analysis)**




(a)Acceleration of girder (b)Displacement of girder

**Fig. 4 : Response waves of girder at P2 (transverse analysis)**

displacement of the girder was about twice larger than the longitudinal analysis, and the acceleration of the girder was smaller. The maximum acceleration of P2 was 913.2gal (Model 1).

The response values of Model 1 were larger than the other cases. It is thought that these was caused by the high yield strength and the little stiffness reduction after yielding.

**Response Waves**

***Longitudinal analysis***

The response waves of the girder at the P2 of the longitudinal analysis are shown in **Fig. 3**. The shapes of the waves of Model 2 and Model 3 are very similar. The residual displacement is observed at the end of the wave of Model 2. The residual displacement is considered to be caused by the reduction of the strength.

***Transverse analysis***

The response waves of the girder at the P2 of the transverse analysis are shown in **Fig. 4**. The shapes of the waves are very similar.

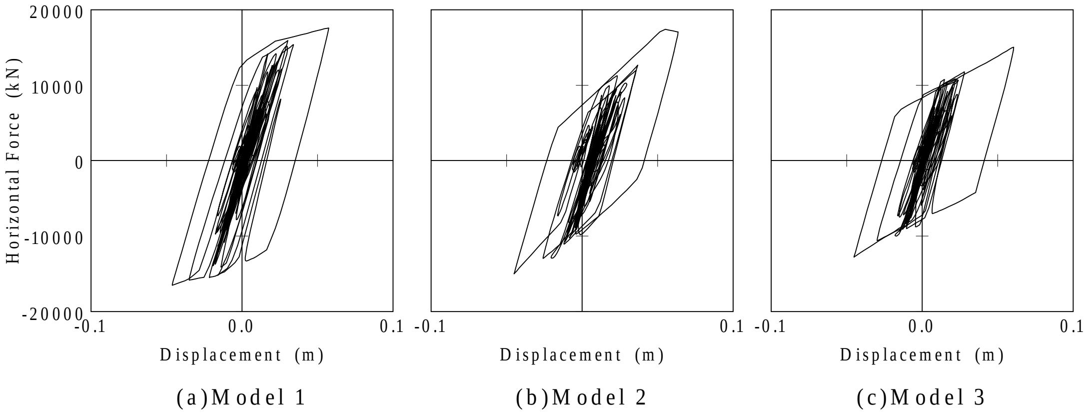



**Fig. 5 : Hysteresis loop of bottom of P2 (longitudinal analysis)**

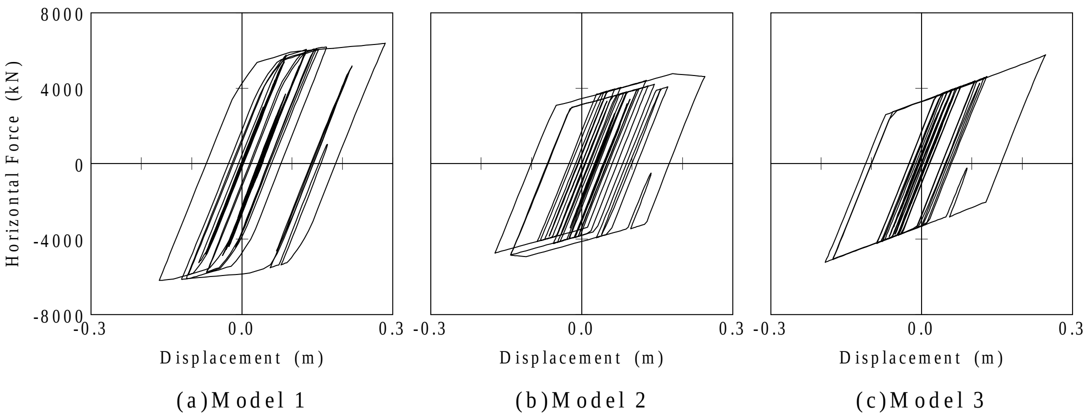



**Fig. 6 : Hysteresis loop of bottom of P2 (transverse analysis)**

The residual displacement is observed at the end of the waves of Model 1 and Model 2. It is thought that the residual displacement of Model 2 was caused by the reduction of the strength and the residual displacement of Model 1 was caused by the relatively strong seismic response shown in **Table 8**.

**Hysteresis Loop of Pier**

***Longitudinal analysis***

The hysteresis loops of the P-Delta relation of the P2 in the longitudinal analysis are shown in **Fig. 5**. The horizontal force in the graph is the quotient of the moment at the bottom of the pier divided by its equivalent height. The displacement in the graph is the value of the top of the equivalent pier. The negative slope was observed in the loop of Model 2.

In the comparison of the outline of the hysteresis loop, the height of the loop of Model 1 is relatively high and the width is relatively small, because of its high yield strength and the little stiffness reduction after yielding. The loops of Model 2 and Model 3 are relatively low and relatively wide. Therefore, the areas of the outline of the loops are similar. In the longitudinal analysis, the plastic deformation energy of the pier elements under the strong earthquake seems to be independent of the modeling method.

***Transverse analysis***

The hysteresis loops of the P2 in the transverse analysis are shown in **Fig. 6**. The horizontal force in the graph is the quotient of the moment at the bottom of the pier divided by its height. The displacement in the graph is the value of the girder. There is the negative slope in the loop of Model 2.

In comparison of the outline shape of the hysteresis loops, the height and width of Model 1 were larger than the other cases, and the area of the outline was superior. On the other hand, the areas of the loop outline of Model 2 and Model 3 were similar and smaller than Model 1. In the transverse analysis, the plastic deformation energy of the pier elements under the strong earthquake seems to be dependent on the modeling method.

## CONCLUSION

The elasto-plastic analysis of the three-span continuous steel rigid frame bridge was carried out. Below are the conclusions for each of the modeling method.

**Model 1**

The section forces of the piers under the strong earthquake were relatively large, and the ultimate displacement is relatively small. The seismic design based on this model will be conservative. In this modeling method, the inelastic properties of the piers are defined as the M-Phi relations. This modeling is useful in the modeling of the rigid frame bridge.

**Model 2**

The residual displacement was occurred in the longitudinal and the transverse analysis. It is thought that the residual displacement was caused by the negative slope of the hysteresis loop. The strength reduction after maximum strength seems to be effective for the residual displacement.

**Model 3**

The maximum response of the bridge is similar to Model 2. The yield strength of Model 3 was the same as Model 2 and different from Model 1. Hence The elasto-plastic behavior of the bridge seems to be dependent on the calculation of the yield strength of the pier elements. The negative slope did not occur in the analysis of Model 3. The strength reduction after maximum strength seems to be not effective for the maximum response.

This study is in connection with the on-going research of the “Committee on Improvement of Seismic Performance of Middle Span Length Bridges” of Japan Society of Civil Engineers Committee Western Division.

## REFFERENCE

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