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Non linear seismic response analysis of the Aratsu cable-stayed bridge

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Abstract

In this paper, the seismic capability of existing steel cable-stayed bridges was checked by nonlinear seismic response analysis using the observed records that were obtained at the 1995 Hyogoken Nanbu Earthquake. The effect of boundary condition (consideration of soil spring), modeling of bearing supports on the pier, existence of tuned mass damper at the top of the tower are investigated.

As a result, it is evident that the main steel tower, the main steel girder and the main part of the reinforced concrete pier are safe but there is some defect at the bearing supports.

In the case of longitudinal excitation, relative displacement between upper and lower part of a movable bearing becomes ten times of displacement allowance.

On the other hand for transverse excitation, reaction forces for the bearings of the girder are larger than design force, and axial forces of set bolts are larger than the tensile strength of them.

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1. Introduction

Design specifications for highway bridges in Japan[1] being based on experience of the 1995 Hyogoken Nanbu Earthquake were revised on December 1996. According to the new specification, bridges that show complicated behavior during earthquake, for example cable-stayed bridges, must be analyzed dynamically, and the results must be reflected for the design. Even existing important bridges that make great influence to the society, also must be checked by the same method. If serious damages are expected, counter measures must be taken swiftly.

This paper investigates the validity of analytical modeling for seismic response analysis, performs nonlinear seismic response analysis of a steel cable-stayed bridge, and studies the results.

2. The Aratsu Bridge

The Aratsu Bridge, Fig.1, that is the bridge analyzed in this paper is a steel cable-stayed bridge, length 346m, has been completed 1988. The design conditions are shown in Table 1.

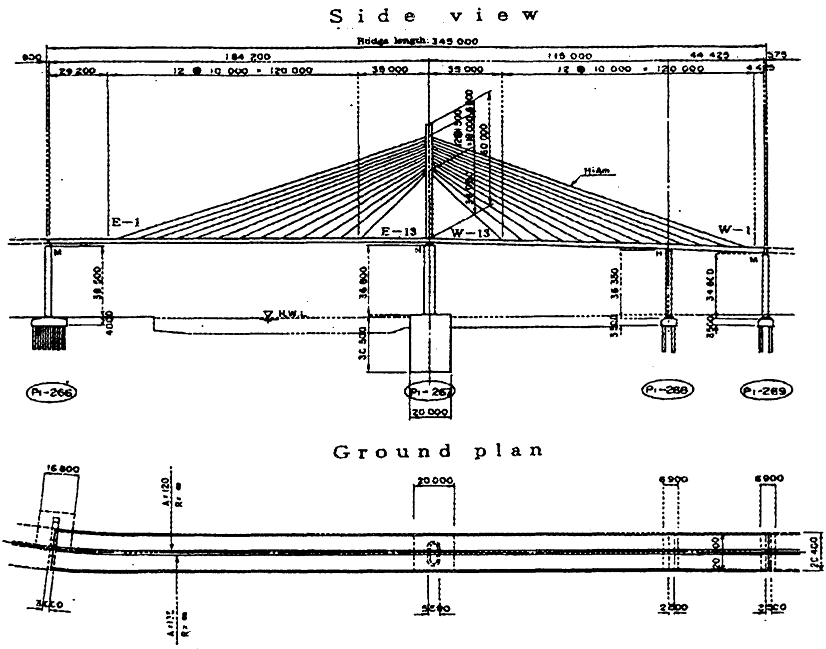

Fig.1 The Aratsu Bridge general view

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The shape of the bridge is multi-fan, single-plane cables (13 stairs) and single tower. Three pivot bearings are installed on the top of the reinforced concrete (RC) pier to sustain the steel main tower and the girder. The tower and the girder is combined rigidly on this pier. A TMD(Tuned Mass Damper) is set at the top of the tower for aerodynamic stability.

Table 1 The design conditions of the Aratsu Bridge

Superstructure	3 span continuous steel cable-stayed bridge
Bridge length	L = 345.0 m (184.2m+115.0m+44.4m)
Width	W = 8.5 m
Substructure	Pier P1-266(Move),268(Hinge),269(Move) : Steel multi-column bent
	Pier P1-267(Hinge) : RC single-column hollow bent
Foundation	Pier P1-266,268,269 : Cast-in-place concrete pile, ϕ 1200mm, n=36,12,12
	Pier P1-267 : Pneumatic caisson
Gground	Class II
Applied specification	Specifications for highway bridges (1980 edition)

n; number of piles

3. Analytical Modeling and Conditions

3.1 Basic Model (BM)

For the purpose of comparison and investigation, a most simple model named 'basic model (BM)' is created firstly. It's shown at Fig.2. The bottoms of the piers are fixed to the ground.

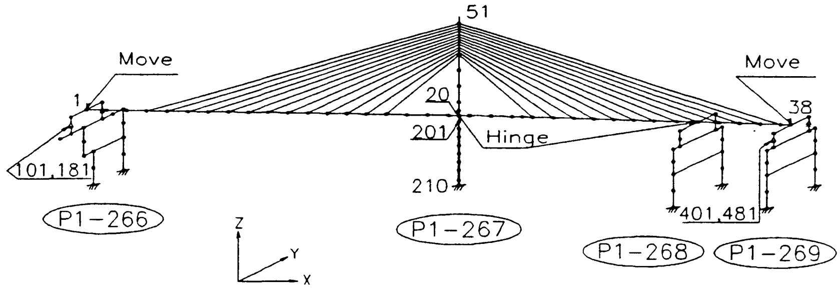

Fig.2 Basic Model (BM)

On this model, the main girder on the piers P1-266 and P1-269 is set up free for longitudinal direction, and is set up fix for longitudinal direction and free for rotation on piers P1-267 and P1-268. A couple of bearing supports are set up for the main girder on the each pier.

3.2 Soil Spring Model (SPM)

For investigation of soil-structure interaction, 'soil spring model' on which the foundation is modeled as beam and the surrounding soil is modeled as spring is created as shown in Fig3.

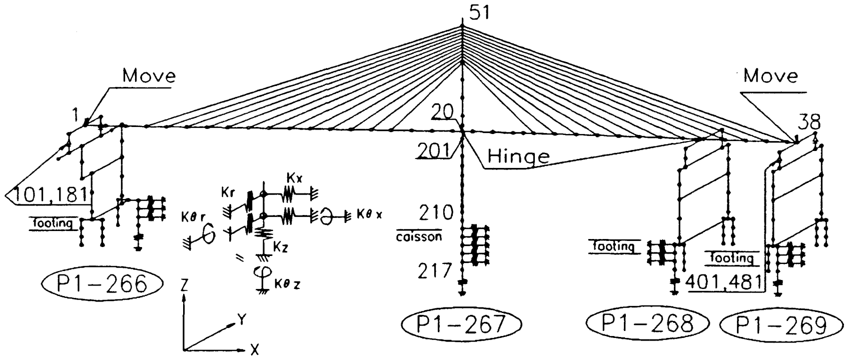

Fig.3 Soil Spring Model (SPM)

3.3 Three Bearings Model (TBM)

On the top of the P1-267 pier, three same type pivot bearings (one as tower bearing and two as girder bearings) actually line up in transverse direction. Though three bearings are concentrated into one bearing in the BM, three bearings are strictly modeled on the top of the P1-267 in the TBM. In order to investigate the accuracy of BM, the results of both models are compared. Fig.4 show the outline of this model.

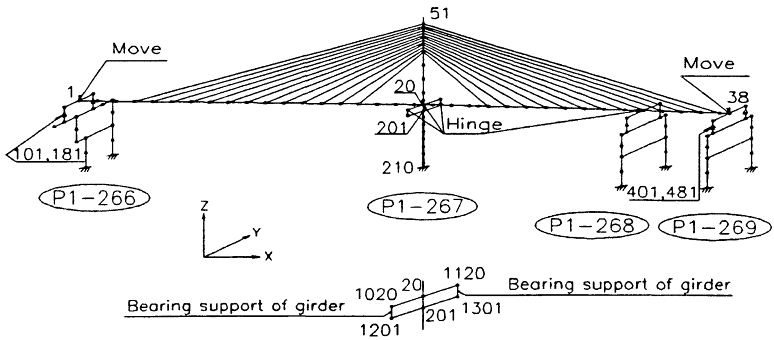

Fig.4 Three Bearings Model (TBM)

3.4 Tuned Mass Damper Model (TMDM)

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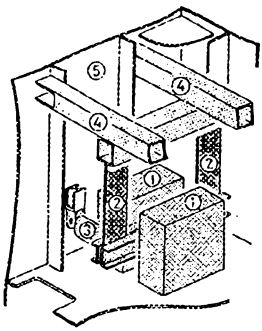
TMD is set up at the top of the tower for the purpose of controlling the tower vibration in the transverse direction of the bridge.

A sketch of this TMD is shown Fig.5. Plate springs② is supported by a beam④, it is fixed at the tower member⑤ and the weight① that is fixed at the bottom of plate springs make the swing of the pendulum. Relative velocity between the tower vibration and the weight of TMD vibration make damping force of oil damper③.



① Weight
3.58 tf (35.11 kN)
② Plate spring
③ Oil damper
0.256 tf/m/sec × 2
(2.599 N/m/sec × 2)
④ Support beam
⑤ Tower web

Fig.5 Sketch of TMD

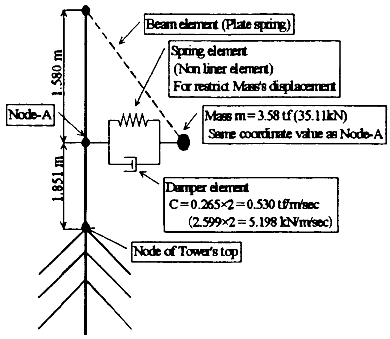


Fig.6 The Model of TMD

Analytical model of TMDM is shown in Fig.6. A spring element (nonlinear spring) is set with a damper element in order to limit maximum relative displacement of the weight less than 200mm that is the maximum stroke of the oil damper. The results of time history response analysis by ground excitation in the transverse direction for BM and TMDM are compared.

3.5 Integrated Model (IM)

Integrated model is integrated soil spring, three bearings on the P1-267 and TMD with BM.(Fig.7) On analysis of this case, dead load and initial tensions of cables are considered.

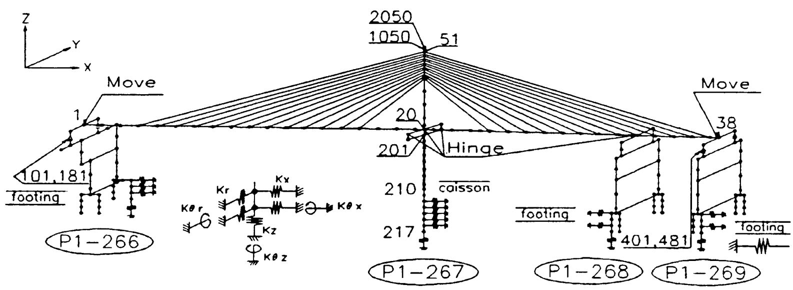


Fig.7 Integrated Model (IM)

3.6 Analytical Condition

Table 2 Conditions of the analysis

Input seismic wave	JR-Takatori EW component for longitudinal direction of the bridge
	JR-Takatori NS component for transverse direction of the bridge
	JR-Takatori UD component for vertical direction
Modification factor for zone	0.7
Damping factor	0.02(Rayleigh damping)
Nonlinear force-displacement relation	tri-linear Takeda model for RC pier
Calculation time of response	20 second
Time intervals of numerical integration	0.01 second

JR; Japan Railroad

4. Results of Analyses

4.1 Comparison between BM and SPM

4.1.1 Acceleration and displacement

Acceleration and displacement are compared at the top of the tower(node 51) and the top of the RC pier(node 201). Table 3 shows the results. In the case of ground excitation in the longitudinal direction there are almost no difference between two models. In the case of ground excitation in the transverse direction, response values of SPM are greater than that of BM.

Table 3 Maximum response acceleration and displacement

Ground excitation in longitudinal direction of the bridge

Node number	Basic model		Soil spring model	
	Gmax(gal)	δ max(m)	Gmax(gal)	δ max(m)
51 (Top of the tower)	401.0	-0.422	-592.7	-0.418
201 (Top of the P1-267)	381.3	-0.433	399.0	-0.431

Ground excitation in transverse direction of the bridge

Node number	Basic model		Soil spring model	
	Gmax(gal)	δ max(m)	Gmax(gal)	δ max(m)
51 (Top of the tower)	-1739.2	1.239	2155.8	1.428
201 (Top of the P1-267)	685.9	-0.049	-843.8	0.088

4.1.2 Internal force

Maximum response moments of the tower and the pier for ground excitations in the longitudinal direction and the transverse direction are compared. Fig.8 and Fig.9 show the maximum moment distribution of the tower in the longitudinal excitation and the transverse excitation. Maximum moment of the tower base of SPM in the transverse excitation is greater 17% than that of BM. This tendency is the same in acceleration and displacement.

Fig.10 and Fig.11 show the maximum moment distribution of the RC pier(P1-267) under the tower in the longitudinal excitation and the transverse excitation. These show the same tendency as the moment of the tower.

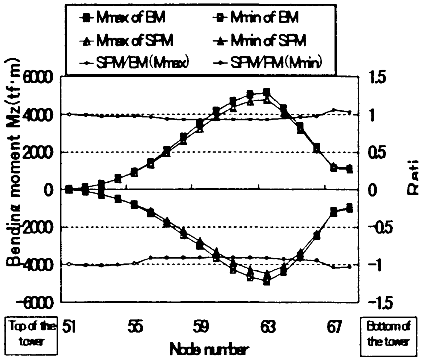


Fig.8 Tower bending moment in the longitudinal excitation

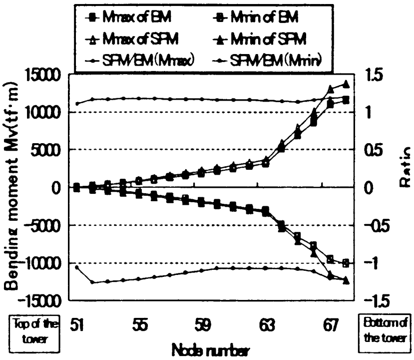


Fig.9 Tower bending moment in the transverse excitation

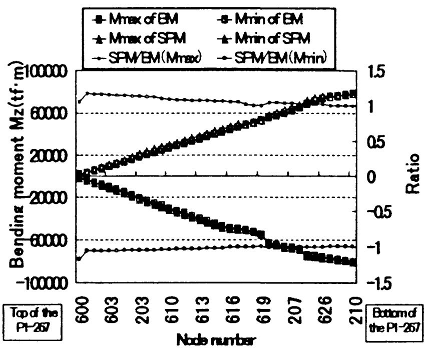


Fig.10 RC pier bending moment in the longitudinal excitation

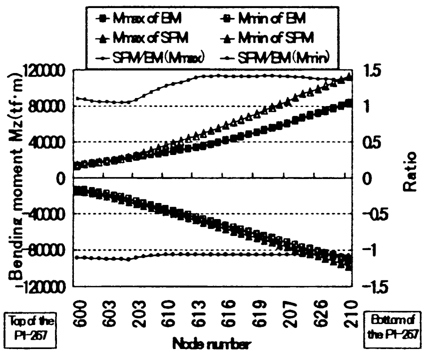


Fig.11 RC pier bending moment in the transverse excitation

4.2 Comparison between BM and TBM

Axial forces of bearings of the P1-267 are compared because the difference is remarkable. The time history of the axial force of BM in the case of the transverse excitation is shown Fig.12. And that of TBM are shown Fig 13. It is distinct that the fluctuation width of the axial force at the bearing of the tower (element 650) is narrow, that of girder (element1 650,1750) is very wide. The results show some response values are larger than the design force at the compressive side and that are over the tensile strength of the set bolts at the tensional side. According to this results TBM is necessary for the design around the bearings.

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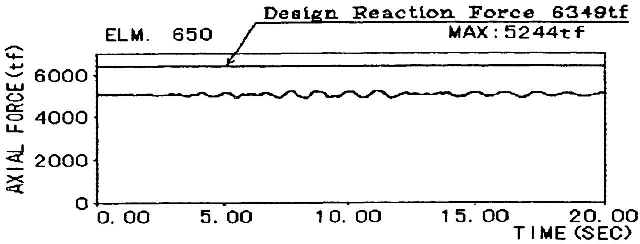

Fig.12 time history response axial force of BM

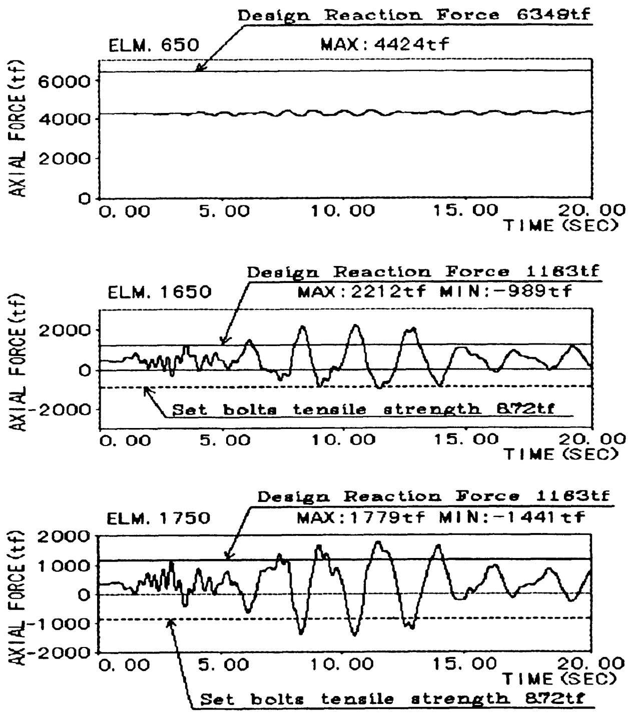

Fig.13 time history response axial force of TBM

4.3 Comparison between BM and TMDM

Bending moments of the tower are shown in Fig.14. Displacements of the main tower in the transverse direction are shown in Fig.15. These values are the maximum response of each node. There is almost no difference between the response values of BM and TMDM. The response values of the bending moment of TMDM are larger than that of BM from the top to the middle of the main tower. And it is hard to say that it is effective, though there is slightly decrease on the response displacement. From these results, the effect of the TMD can not be expected for ground excitations.

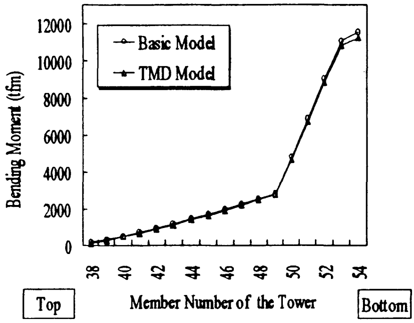

Fig. 14 Maximum bending moment of the main tower in the transverse direction

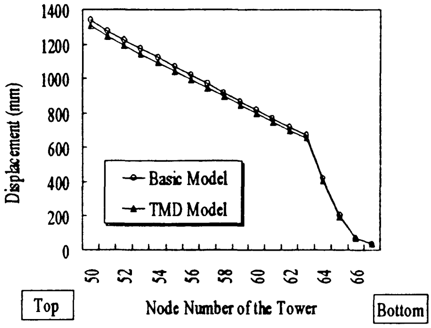

Fig. 15 Maximum displacement of the main tower in the transverse direction

4.4 Analytical results of IM

4.4.1 Displacement and acceleration

The analytical results of response displacements and accelerations of IM by longitudinal, transverse, vertical and three direction simultaneous ground excitations are shown in the following. The points of the observation are the top of the RC pier (node number 201) and the top of the main tower (node number 2050) for maximum response acceleration and displacement.

The maximum response acceleration and displacement of the top of the RC pier are 829gal and 7cm respectively, but those of the top of the main tower are 3,536gal and 178cm in transverse direction of the bridge.

In the longitudinal direction of the bridge, the maximum response acceleration and displacement of the top of the RC pier are 395gal and 42cm respectively, and those of the top of the main tower are 519gal and 69cm.

From these facts, it is proven that only the main tower of which the stiffness is small compared to the RC pier greatly vibrates in the transverse direction.

The analytical results of the direction component of three directions simultaneous excitation are almost equal to that of single direction excitation.

On the movable bearings, relative displacement between the upper bearing and the lower bearing is investigated. Fig. 16 shows time history response displacement of the pier P1-269 for the period of 20 seconds. Maximum relative displacement is 0.9m, it is very large value.

Residual displacement is calculated by 40-second free damped oscillation after the 40-second ground excitation. These values include the initial displacement by the dead load and the initial tension of the cable. Therefore the residual displacement by the plastic deformation is the value that is subtracted the initial displacement.

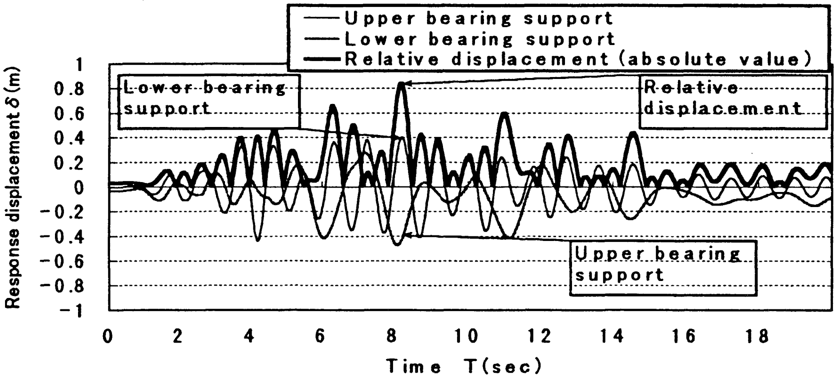

Fig. 16 time history response displacement of the pier P1-269

The points of the observation are the top of the RC pier (node number 201) for the residual displacement. Table 4.4 shows the residual displacement for the ground excitation in each direction. That of the longitudinal direction becomes large values, because the plasticity of the RC pier is generated. However maximum residual displacement is 3.7cm, it is merely 10% of the allowance 36.8cm for the pier height 36.8m that is regulated by specifications for highway bridges in Japan part V[1].

Table 4 residual displacement at node number 201

Direction of ground excitation	Residual displacement δ (m)		
	δ x	δ y	δ z
Longitudinal	-0.029	0.000	0.000
Transverse	0.000	0.002	0.000
Vertical	0.000	0.000	0.000
Simultaneous in 3 di.	-0.037	0.000	0.000

4.4.2 Internal force

Fig 17 and Fig 18 show the maximum moment of the pier P1-267 in the longitudinal and transverse direction respectively. Cracking strength, first yield strength and ultimate strength are also shown in the same figure. Fig. 17 shows that the response moment in the longitudinal direction exceeds the cracking strength in most part of the pier, and exceeds also the first yield strength at the base of the pier, however it doesn't exceeds the ultimate strength. Fig. 18 shows

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that the response moment in the transverse direction exceeds the cracking strength from the base of the pier to the place of about 2/3, however it doesn't exceeds the first yield strength.

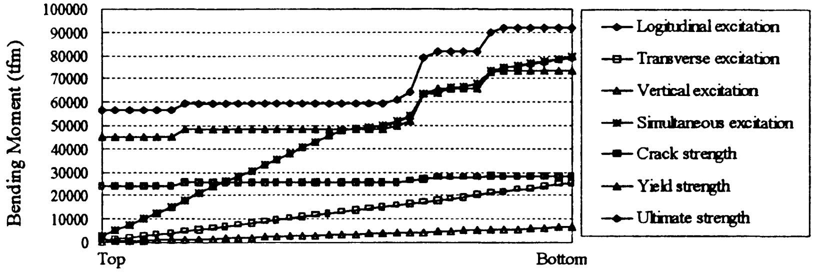

Fig. 17 maximum moment of the pier P1-267 in the longitudinal direction

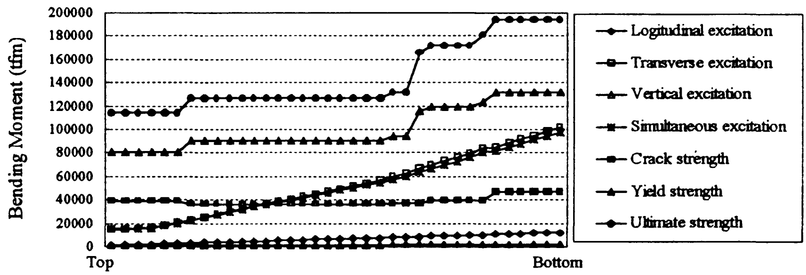

Fig. 18 maximum moment of the pier P1-267 in the transverse direction

Fig. 19 shows moment-curvature hysteresis loop at the base of pier P1-267 in the longitudinal direction, and Fig.20 shows that in the transverse direction. Fig.19 shows large loop because the response bending moment has exceeded the first yield strength. However ductility factor of the base of the pier is 3.09, it's smaller than the allowable ductility factor 5.78. Fig.20 shows small loop because the response bending moment has not exceeded the first yield strength.

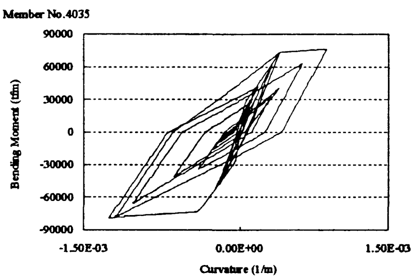

Fig.19 moment-curvature hysteresis loop in the longitudinal direction

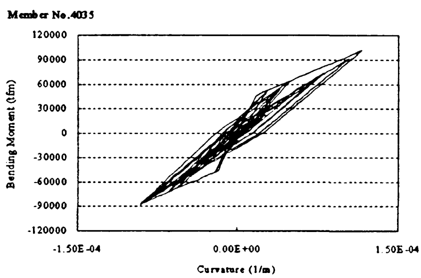

Fig.20 moment-curvature hysteresis loop in the transverse direction

5. Conclusions

The displacement, acceleration, internal force for the transverse excitation tends to increase, when the soil is modeled in the spring.

All the bearings must be modeled. Especially large axial force occurs at the bearings of the girder by the transverse excitation. Same results have been obtained in other studies[2,3,4] for urban viaduct, so the attention is necessary for the design of the bearing support.

TMD setting at the top of the tower for aerodynamic stability has no effect for strong earthquake, inversely sometimes increase section force.

The maximum displacement of the top of the tower is 70cm in the longitudinal direction, exceeds 2m in the transverse direction.

The maximum relative displacement of the movable bearings between upper and lower bearings is 90cm at the pier P1-269. This must be considered for the design of the bearings.

The maximum bending moment of the pier P1-267 exceeds the first yields strength, but does not exceed the ultimate strength yet in the longitudinal direction. And that exceeds the cracking strength, but does not exceed the first yields strength in the transverse direction.

The maximum residual displacement is very small, so it would not become a problem.

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