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APPLICATION OF ENERGY ABSORBING CONNECTERS TO STEEL CONTINUOUS GIRDER BRIDGES

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SUMMARY

The efficiency of a seismic energy dissipation device is studied. Steel bellows are connected between girders and abutments in order to reduce the damage of steel bridges. The effectiveness of the bellows is verified by the dynamic response analyses for three-span steel continuous girders on rubber bearings. The piers are single column type made of reinforced concrete designed under L1 or L2 earthquake condition of Japanese code. Their heights are varied in three ways. The seismic performances are examined under various combinations of designed piers. Furthermore, estimation formulas concerning the maximum displacement of superstructure and piers are developed by considering the energy balance between seismic input energy and dissipated hysteretic energy, and their validities are evaluated by comparing with nonlinear dynamic response analyses. The estimation formulas produce good results in spite of their simplicity, and therefore they are efficient for selection of suitable bellows.

INTRODUCTION

Passive base isolation such as rubber bearings can reduce damage of bridges due to the increase of the fundamental vibration period and the additional damping provided to dissipate the seismic energy. However, it becomes possible to cause collision between girders or between girders and abutments due to large longitudinal displacements of the superstructure subjected strong earthquakes. This collision results in increased failure of super- and substructures. The various kinds of mitigation devices such as rubber type and metal type have been investigated and these studies show that pounding forces can be considerably reduced [1-3]. Most of the proposed devices are effective only after collision and for compression.

Authors have proposed a design method in which steel bellows as energy absorbing devices are connected between girders in a row or between girders and abutments [4]. They are connected to the web of girders. The bellows are effective even before collision and, moreover, both in compression and tension. The

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characteristics of bellows were investigated by loading tests and finite element method, and the design formulas as to the initial rigidity, the yield strength and the yield displacement of bellows were developed. The effectiveness of steel bellows for simply supported steel girders on metallic bearings and for steel continuous girders on rubber bearings was also studied in the case of their placement between abutments at both ends [4]. Furthermore, a practical method was developed in order to decide the characteristics of bellows by using the response spectrum analysis based on the equivalent linear method. This method is effective but repeated dynamic analyses are necessary and the displacements of superstructure and piers tend to be overestimated.

In this paper, the efficiency of steel bellows for steel continuous girders on rubber bearings is investigated and a more practical and simple method for the selection of characteristics of bellows is proposed by considering energy balance between seismic input energy and dissipated hysteretic energy. Computer programs were developed using MATLAB for response spectrum analyses and non-linear time-history analyses. Dynamic analyses are carried out also by these programs.

ANALYTICAL MODELING OF A STEEL CONTINUOUS GIRDER BRIDGE

Analytical model of bridge

A model of three-span continuous girder is shown in *Fig.1*. The superstructure consists of five steel plate girders, and the total dead load is 15MN. The characteristics of the pier used for the analysis are shown in *Table 1*. The piers are single column type made of reinforced concrete. Their heights are varied in three ways.

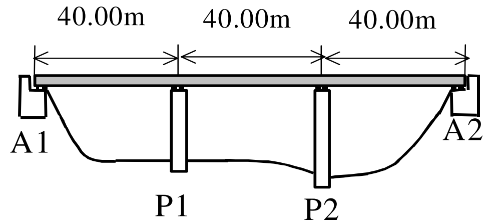


Fig. 1 A Continuous girder on rubber bearings

The pier section characteristics were decided for two strength level, small (L1 level) and medium (L2 level). In the design of pier, effective weight of the superstructure was assumed to be dead load reaction force (5.0MN) to all piers regardless of their heights. The longitudinal strengths and the displacements are shown in *Table 1* at the point of yield initiation of steel bars and at an ultimate state. The strengths are expressed by the seismic intensities. The intensities of the piers designed by L1 earthquake are about 0.4 and the intensity of the pier designed by L2 earthquake is about 0.6. *Table 2* shows the case of analysis in terms of the combination of pier type. Lead rubber bearings (LRB) are used for the purpose of seismic base isolation. The dimensions of LRB are decided so that the fundamental period coincides with the target period, and the maximum shearing strain in rubber does not exceed 250%. *Table 2* shows combination cases of piers and the maximum displacements obtained by the static design based on the ductility design method. In the first column of *Table 2*, ‘S’ means small strength pier and ‘C’ means medium strength pier. The numbers next to ‘S’ or ‘C’ show pier heights of P1 and P2. For example, S618 means the pier combination where P1 and P2 are designed by L1 earthquake and their heights are 6m and 18m, respectively. Since LRB have non-linear hysteretic

Table 1 Characteristics of concrete piers

Strength of Piers	*Height of Pier (m)*	*Weight of Piers (kN)*	*Steel Rods Area (cm^2)*	*Yield initiation*		*Ultimate state*		*Horizontal strength (Seismic intensity)*
				Strength (kN)	*Disp. (cm)*	*Strength (kN)*	*Disp. (cm)*	
Small (Level 1)	*6*	*913*	*548*	*1.80*	*1.7*	*2.24*	*14.2*	*0.40*
	12	*1900*	*1110*	*1.98*	*5.7*	*2.53*	*25.3*	*0.41*
	18	*3270*	*1550*	*2.12*	*10.7*	*2.76*	*45.9*	*0.41*
Medium (Level 2)	*6*	*1240*	*668*	*2.70*	*1.3*	*3.45*	*16.7*	*0.60*
	12	*2650*	*1330*	*3.06*	*4.3*	*3.99*	*30.8*	*0.61*
	18	*4170*	*2190*	*3.38*	*9.3*	*4.41*	*39.7*	*0.62*

Table 2 Combination cases of piers and Displacements obtained by Static Design

Name of case	Superstructure (cm)	Pier P1 (cm)		Pier P2 (cm)	
		Pier	LRB	Pier	LRB
S66	26.2	6.6	19.6	6.6	19.6
S1212	41.4	16.6	24.8	16.6	24.8
S1818	53.2	32.5	20.7	32.5	20.7
S612	33.1	6.6	8.2	26.5	24.9
S618	35.3	9.0	11.4	26.3	23.8
S1218	35.3	14.3	16.1	21.0	19.2
C66	21.6	1.0	20.6	1.0	20.6
C1212	31.0	3.5	27.5	3.5	27.5
C1818	32.6	3.2	29.4	3.2	29.4
C612	24.5	1.0	23.5	3.3	21.2
C618	32.5	1.1	31.4	6.8	25.7
C1218	31.4	3.7	27.8	7.7	23.8

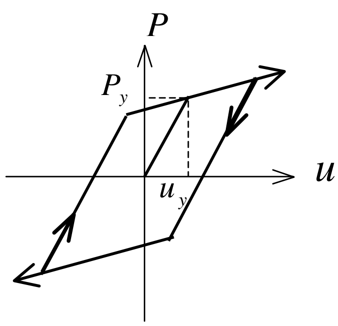


Fig. 2 Force vs. disp. relationship of hysteretic bi-linear springs

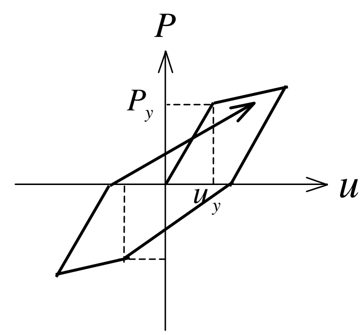


Fig. 3 Hysteretic modified Takeda model springs

characteristics, the load displacement curves of LRB are expressed by hysteretic bi-linear springs. The rigidity of unloading is equal to the initial rigidity. The hysteretic model of bi-linear type is shown in *Fig.2*. We examine here the behaviors in longitudinal direction, and the superstructure is treated as rigid bodies. The displacements of abutment are disregarded since it is rigid enough. The number of degrees of freedom is three in total (superstructure and two middle piers). The force-displacement law of piers is represented by the modified Takeda model which can treat the degradation effects under cyclic loading. *Fig.3* shows hysteretic load-displacement curve of the modified Takeda model.

Mechanical characteristics of steel bellows

The steel bellow connected between a girder end and an abutment is shown in *Fig.4*. Two steel plates are bent in semicircular shapes and are connected to the web of a girder and to the anchored plate. *Fig. 5* shows loading and unloading paths obtained by the laboratory tests and FEM analyses [4]. The width of the specimen is 25cm, the radius is 15cm and the thickness is 1.9cm. Strictly speaking, the inclinations of the loading pass after yielding of the compression side are less steep than the tension side, and as unloading-reloading advances to opposite direction, this tendency becomes more conspicuous. However, the loading and unloading paths are approximated by the hysteretic

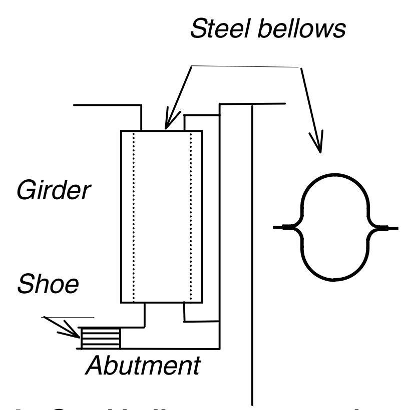


Fig. 4 Steel bellows connected to abutment and its cross section

bi-linear springs shown in *Fig.2* for simplicity. The design formulas were shown in [4] concerning the structural characteristics such as yield strength, yield displacement and maximum strength of bellows.

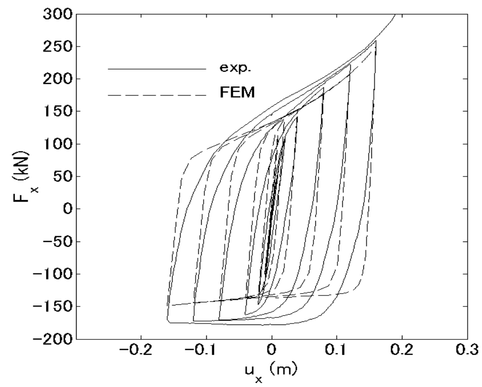
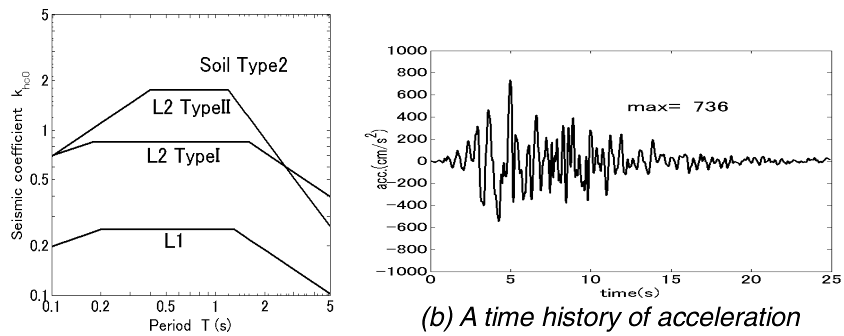


Fig. 5 Loading and unloading paths of specimen

Response spectrum and seismic wave

The response analyses are carried out in the next chapter by taking into account only the behaviors in longitudinal direction for the whole structure. *Fig. 6(a)* shows the used response spectra that are given in Japanese seismic code for soil type 2 [5]. Time-history analyses are also carried out in the next chapter by taking into account the non-linear behaviors. As an input data of seismic wave we use acceleration time histories that are obtained from the actual earthquakes and so modified as to satisfy the spectrum for L2 earthquake (Type II). The seismic wave used is shown in *Fig. 6(b)*.



(a) Response spectrum

(b) A time history of acceleration (L2,TypeII)

Fig. 6 Response spectra and a time history of acceleration

APPLICATION OF STEEL BELLOWS TO A STEEL CONTINUOUS GIRDER BRIDGE

Calculation results of dynamic analyses in case without bellows

The cases with usual restrainers (without bellows) in which the energy absorbing is not expected are calculated first for comparison. In these cases, both of stiffness and damping of the restrainers are ignored because they are not effective before the displacement of superstructure exceeds a certain value. *Fig.7* shows the maximum displacements of superstructure and piers obtained by two dynamic response analyses, the response spectrum analyses based on the equivalent linear method and the non-linear time-history analyses. The procedure of the response spectrum analyses is as follows. First, the effective displacements of pier and LRB are assumed, and then equivalent linear stiffness and the damping ratios of non-linear spring elements are calculated. The response values are calculated from the response spectrum analyses. Calculations are repeated until the differences between the obtained displacements and the assumed displacements become within 5% [4]. In *Fig.7*, ‘sp’ means results of the response spectrum analyses and ‘nl’ means results of the non-linear time-history analyses. The differences between calculated values by response spectrum analyses and non-linear time-history analyses are small. The response spectrum analyses based on the equivalent linear method can be a practical and effective means as a design method in case without bellows. The displacements of small strength piers are considerably large in comparison with those of medium strength piers. Some of the displacements of small strength piers exceed the ultimate displacement. *Fig.8* shows the time-history response of the displacements of

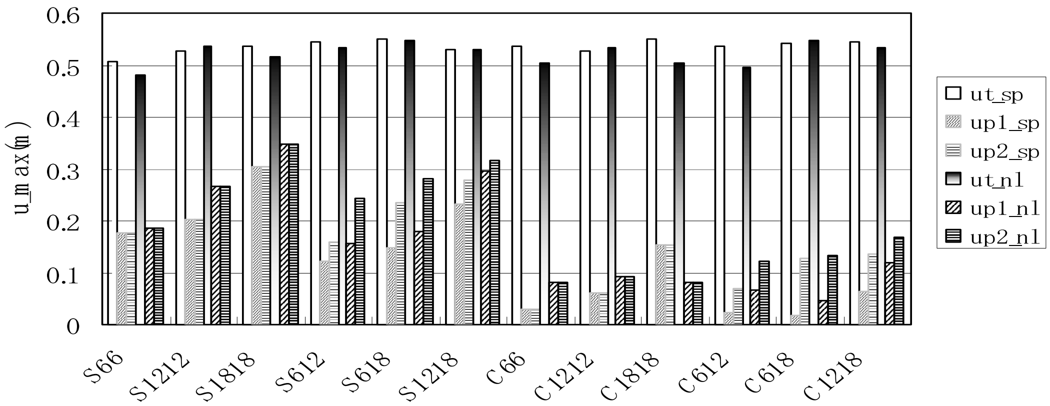


Fig.7 Calculation results of dynamic analyses (the maximum displacements without bellows)

piers and the load displacement curve of piers in case of S618 in which the difference of pier heights is largest. It can be confirmed that large energy absorbing is demonstrated from the load displacement history curve of the piers. To clarify the amounts of the energy absorbing of the piers and LRB quantitatively, the amounts of energy absorption of each set of piers and rubber bearings are calculated. *Fig.9* shows the accumulated time-history responses in terms of energy absorption of piers and of the bearing, respectively, for S618 and C618. In case of S618, the amounts of energy absorption of piers are larger than those of LRB, and in case of C618, the amounts of energy absorption of piers are less than those of LRB. The piers with small strength may cause severe damages. The maximum displacements of superstructure shown as 'ut' in *Fig.7* are almost equal regardless of pier strength. These values exceed 50cm and collision may be caused in the expansion joints in these cases.

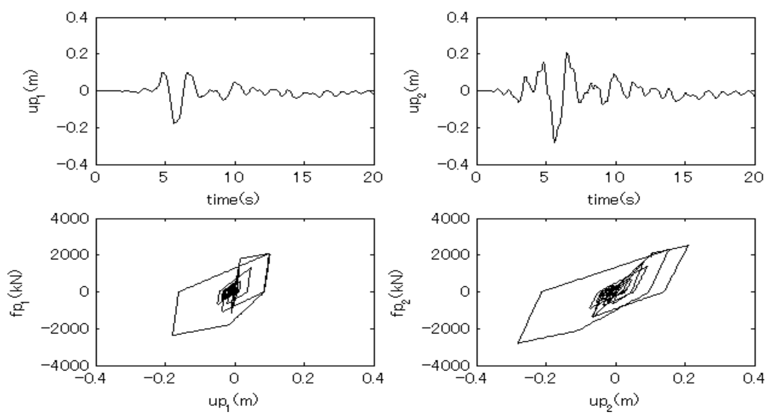


Fig. 8 Results of S618 by nonlinear time history analyses (without bellows)

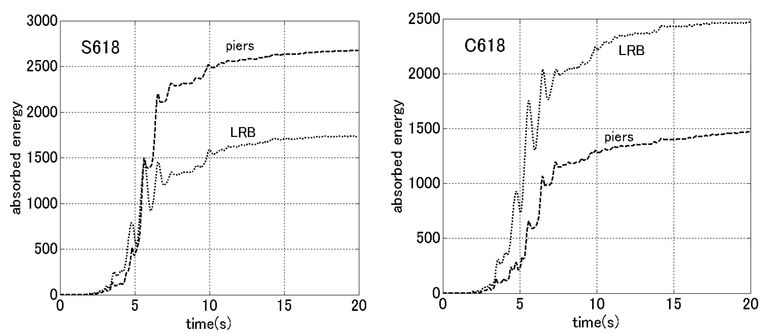


Fig. 9 Energy absorbing of bearings and piers (S618 and C618 without bellows)

Calculation results obtained by dynamic analyses give large values compared with the static design shown in *Table 2*.

Effect of steel bellows as energy absorbing connecters

In case without bellows, the maximum displacements of superstructure exceed 50cm. In this section, the effects of steel bellows are examined. The bellows are connected between a girder end and an abutment as shown in *Fig.1*. Yield strengths of steel bellows are so decided as to make the displacements of middle piers less than yield displacement, and to suppress the displacement of superstructure within ca. 10cm-15cm which can be absorbed by expansion joint comparatively easily. The yield strength of bellows is determined to be about 20%-40% of the superstructure weight that corresponds to 1.0-2.0 times the seismic force caused by L1 earthquake. The comparison of the maximum response displacement of cases with and without bellows is shown in *Fig.10* by using same height piers model. The same yield strength of bellows, $P_y\ /W\ = 0.4$ is used for all cases where P_y is the total yield strength of both side bellows and W is the total weight of superstructure. In *Fig.10*, (a) shows the results of the response spectrum analyses and (b) shows the results of the non-linear time history response analyses. The ordinate of *Fig.10* shows normalized pier displacements using yield displacement of each pier. The abscissa of *Fig.10* shows displacements of superstructure. The results with bellows are shown as BEL and their marks are painted out with black color. The results of the response spectrum analyses give relatively larger values than those of the non-linear time history response analyses of case with bellows. We observe that the displacements of superstructure and piers can be reduced to large extent by using steel bellows. We must point out here that if the same yield strength of bellows are used, the maximum displacement of superstructure becomes almost same regardless of pier strength or height on the results of the non-linear time history analyses. Since the response spectrum analyses give relatively larger displacements in case with bellows, a more precise design method is needed in order to decide the suitable characteristics of steel bellows.

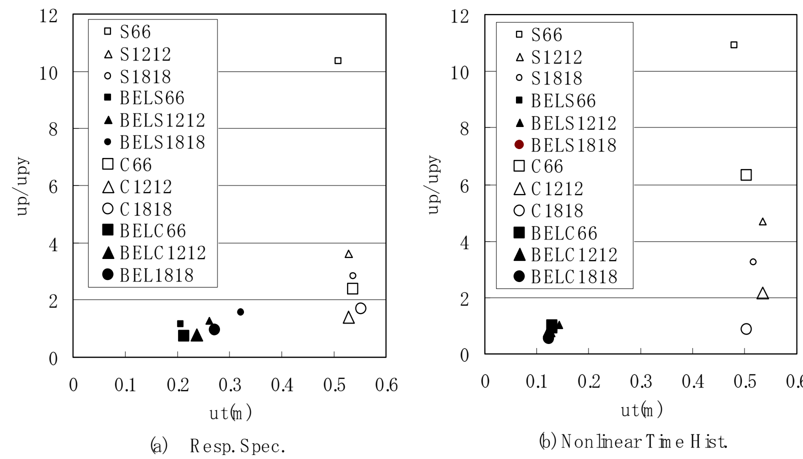


Fig. 10 Comparison of maximum displacements with and without bellows ($P_y\ /W\ = 0.4$ ***)***

A design method of steel bellows as energy absorbing connecters

We note here that the behavior of superstructure with bellows is similar in case of same yield strength of bellows. *Fig.11* shows energy absorbing of hysteretic members in case of small strength and high piers, BELS1818, and in case of medium strength and low piers, BELC66. The energy absorbing amount of bellows is dominant for both cases even though the energy absorbing of piers in BELS1818 is considerably larger than in BELC66. This fact suggests that a continuous girder with bellows can be modeled in one nonlinear hysteretic spring and one mass. Many methods have been proposed to estimate the maximum inelastic displacement of single-degree-of-freedom systems [6-7]. In most methods, the maximum displacement is estimated from the maximum displacement of linear elastic systems.

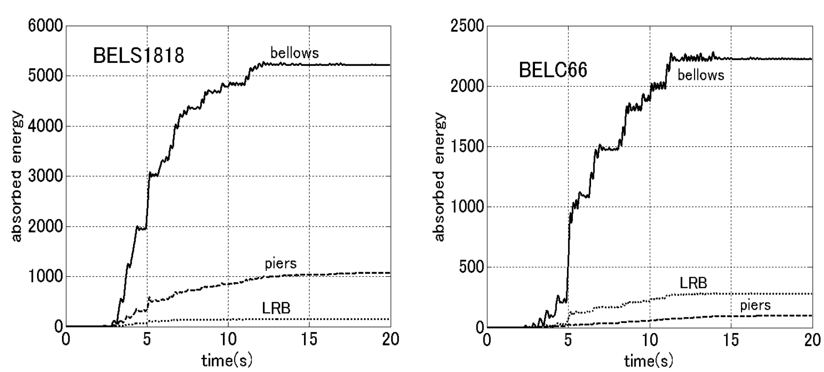


Fig. 11 Energy absorbing of bellows, bearings and piers (S1818 and C66 with bellows)

Matsuda et al. show that a estimation method based on non-linear energy absorption response spectra gives more precise values than a estimation method based on linear energy absorption response spectra [8].
Single-degree-of-freedom model used is shown in *Fig.12*. They calculated the nonlinear history absorption energy by using bi-linear model of the perfect elasto-plasticity. So as not to depend on the size of mass, the absorption energy is converted into the equivalent velocity.

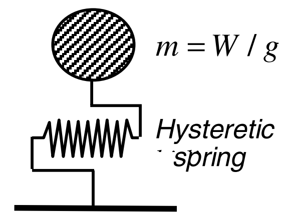


Fig.12 Single-degree-of-freedom model

$$E = \int_0^T f(u)\,\dot{u}\,dt = \int_0^T f(u)\,u\,du\,, \quad V_e = \sqrt{2E/m} \qquad (1)$$

We calculated the equivalent velocity using the seismic wave shown in *Fig.6(b)*. Since second stiffness of plastic region in bellows hysteretic curves affects the amount of absorption energy, the ratio of second stiffness to first stiffness is assumed to be 0.05 and is taken into account.
Fig.13 shows the non-linear spectra in which the equivalent velocity is plotted for the elastic period. As shown in *Fig.14*, hysteretic absorbing energy in a cycle of the bi-linear spring is expressed as follows.

$$W_{hys} = 4(1-\gamma)P_y(u_{max} - u_y) \qquad (2)$$

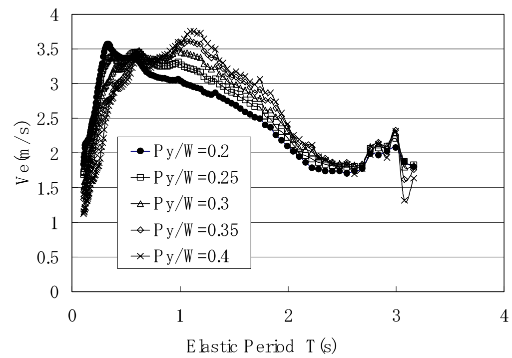


Fig.13 Non-linear spectra of Absorbing energy

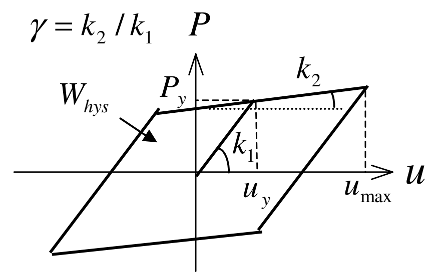
When we assume that only members of steel bellows absorb input seismic energy, total seismic energy is expressed:

$$E = n \cdot W_{hys} \qquad (3)$$

where n means hysteretic number when assuming that the bi-linear spring repeats hysteretic curve by the maximum displacement.
The maximum displacement of the superstructure with bellows is estimated from the following procedure.

1) By assuming the ratio, P_y / W, yield strength of bellows to total weight of superstructure, the equivalent velocity, V_e, is read out from the elastic period using *Fig.13*.
2) The total seismic energy, E, is counted backward from V_e.
3) Select the coefficient n.
4) Estimate the maximum displacement from the balance between E and the absorbing hysteretic energy $n \cdot W_{hys}$.

$$u_{max} = \frac{E}{4n(1-\gamma)P_y} + u_y \quad (4)$$



$$W_{hys} = 4(1-\gamma)P_y(u_{max} - u_y)$$

Fig. 14 Hysteretic absorbing energy of bi-linear springs

By this method the maximum displacement can be presumed without repeating the calculation. *Fig.15* shows the comparison of estimated maximum displacement and calculated maximum displacement from non-linear dynamic analysis.

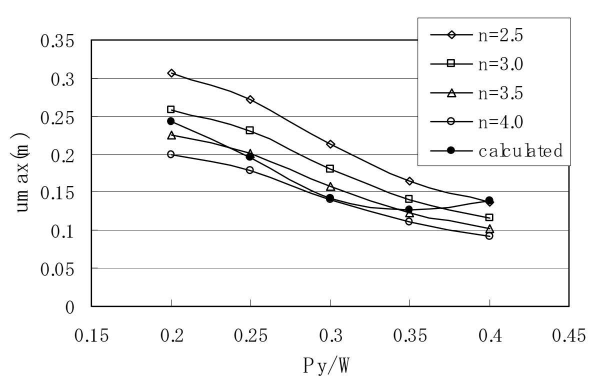


Fig. 15 Estimated maximum displacement from energy balance

The model used in the non-linear dynamic analysis is S1218 in *Table 2*. When $n = 3$ is adopted, the estimated maximum displacement gives the value of the safe side with a exception of the case $P_y / W = 0.4$.

The maximum displacements of piers are estimated from the balance of forces acting on the pier as shown in *Fig.16*.

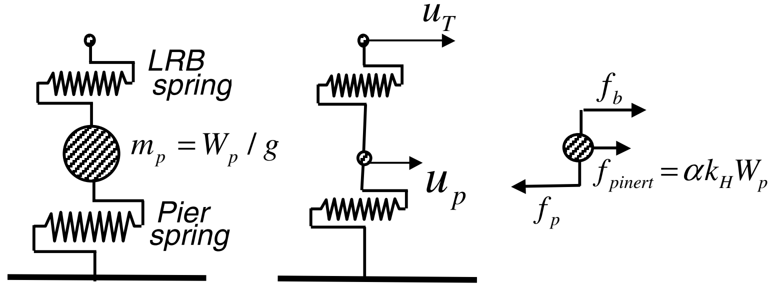


Fig.16 Forces acting to a pier

$$f_p = f_b + f_{piner} \qquad (5)$$

where f_b is the spring force of the LRB, f_p is the spring force of the pier and f_{piner} is the inertia force of the pier. Assuming piers are within elastic range:

$$f_p = k_p u_p, \quad f_{piner} = \alpha\, k_H W_p \qquad (6)$$

where k_p is pier stiffness within elastic range and k_H is seismic coefficient obtained from the response spectra (*Fig.6(a)*) according to the elastic period of the pier, $2\pi\sqrt{W_p /(gk_p)}$ where W_p is the effective weight of the pier. In Eq. (6), α is the modified coefficient smaller than 1 for the purpose of adjusting the influence that neither the displacement nor the acceleration becomes maximum at the same time. The spring force of the bearing, f_b is obtained as follows:

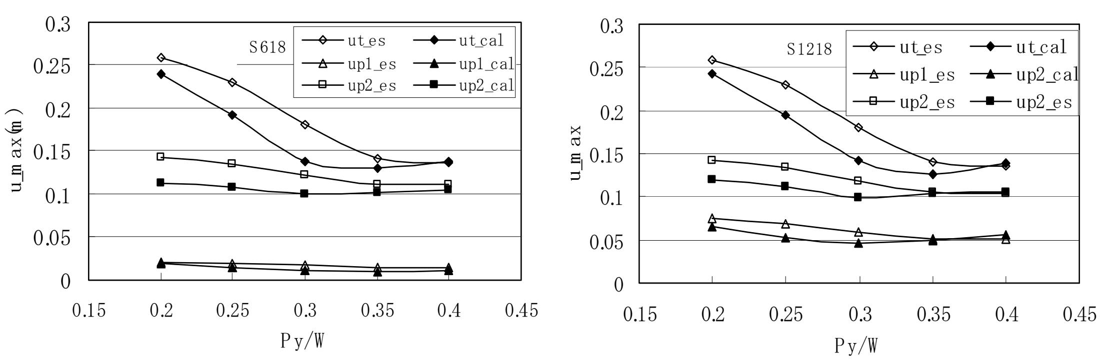
$$f_b = P_{by} + k_{b2}(u_T - u_p - u_{by}) \qquad (7)$$

where P_{by} and u_{by} is the yield strength and the yield displacement of the LRB, respectively. k_{b2} is the second stiffness of the LRB after yielding, and u_T is the displacement of the superstructure. Substituting Eqs. (6) and (7) into Eq. (5):

$$u_p = \left\{P_{by} + k_{b2}(u_T - u_{by}) + k_H W_p\right\}/(k_p + k_{b2}) \qquad (8)$$

Evaluation of the estimation method

The exact estimation of the maximum displacements of the superstructure and the piers is important in performance-based seismic design. Non-linear time history analyses are carried out in order to evaluate the proposed estimation method through the comparison with exact results. The maximum displacements



(a) S618 *(b) S1218*

Fig. 17 Maximum displacements estimated by the proposed method and obtained by non-linear analyses

estimated by the proposed method and the maximum displacements obtained by non-linear analyses are shown in *Fig.17*. The results of model S618 are shown in *(a)* and the results of model S1218 are shown in *(b)* in *Fig.17*. In *Fig.17*, 'ut' means the maximum displacements of the superstructure, and 'up1' and 'up2' means the maximum displacements of Pier 1 and Pier 2, respectively. Also, '_es' expresses the estimated values and '_cal' shows the calculated values by non-linear time history analyses. The coefficients n in Eq.(4) and α in Eq.(6) are assumed to be 3 and 0.6, respectively, for all cases. Though the proposed method tends to overestimate the maximum displacements, it gives a relatively good estimation of the maximum displacements for both superstructure and piers. Steel bellows decrease the maximum displacement of superstructure within 15cm and they enable piers to be within the range of elasticity even for the pier designed for L1 earthquake. It can be seen that the maximum displacements become small as the yield strength of the bellows is enlarged to some extent but further enlarging cannot be effective. Practically, the recommended values of the ratio of P_y / W are from 0.2 to 0.3 according to the extent of the maximum displacement which the superstructure demands in these models.

CONCLUSION

The effect of the steel bellows as girder connecters of energy absorbing type have been studied by means of the non-linear time history analyses for continuous girders on rubber bearings. Also, a practical method is proposed to estimate the maximum displacement of the superstructure and piers using energy balance when the steel bellows are attached between girders and abutments.

The main conclusions of these studies are:

Steel bellows reduce considerably the displacements of superstructure and seismic forces to piers. Calculation values obtained from the developed estimation method agree fairly well with those of the non-linear time history analyses.

Desired characteristics of steel bellows are satisfied by means of adjusting the yield strength of the bellows.

REFERENCES

1 JANKOWSKI R, Wilde K, FUJINO Y "Reduction of pounding effects in elevated bridges during earthquakes" *Earthquake Engineering and Structural Dynamics*, 2000: 29, 195-212.

2 OBATA M. et al. "Mechanical behaviors of a shock absorbing bridge restrainer plate" *JSCE Journal of Structural Mechanics and Earthquake Engineering*, No.612, 1999: 239-249. (in Japanese)

3 KOSHITOUGE M. et al. "Energy absorption performance of the cylindrical rubber shock absorber" *JSCE Journal of Structural Mechanics and Earthquake Engineering*, No.675, 2001: 99-112. (in Japanese)

4 ZUI H. et al. "Application of energy absorbing connecters to steel girder bridges" *IABSE Symposium, "Towards a better built environment - innovation, sustainability, information technology", Melbourne, September 2002.*

5 Japan Road Association "Specifications of Highway Bridges" JRA, 1996.

6 Miranda E, Ruiz-Garcia J "Evaluation of approximate methods to estimate maximum inelastic displacement demands" *Earthquake Engineering and Structural Dynamics*, 2002: 31, 539-560.

7 Riddell R., Garcia J. E., Graces E. "Inelastic deformation response of SDOF systems subjected to earthquakes" *Earthquake Engineering and Structural Dynamics*, 2002: 31, 515-538.

8 Matsuda T. et al. "A consideration on the displacement based design of seismically isolated highway bridges" *JSCE Journal of Structural Engineering*, 2003: Vol.46A, 907-916. (in Japanese)