Mem. Fac. Eng., Osaka City Univ., Vol. 39, pp. 39−51 (1998)

A Seismic Retrofitting Method and Trial Design for Stiffened Plates in Existing Steel Bridge Piers

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(*Received September* 30, 1998)

Synopsis: Presented in this paper are a seismic retrofitting method and a trial design for stiffened plates in an existing steel bridge pier based on the design guidelines drafted in the Hanshin Expressway Public Corporation in response to the revision of the seismic design method in the Japanese Specifications for Highway Bridges after the Hyogo-ken Nanbu Earthquake. The design guidelines are utilized in the case that the strength of their steel pier columns increases substantially and subsequently the applied seismic load exceeds more than that decided by the strength of the basement structures, if their steel pier columns are filled with concrete as a retrofitting method.

Keywords: stiffened plate, existing bridge pier, retrofitting method, trial design, seismic design

1. Introduction

As steel among structural materials is very ductile in comparison with concrete, it had been considered that the steel structures, designed against an earthquake (level 1) with the maximum acceleration of 150∼200 gals at the surface level of the ground, never collapse against a strong earthquake (level 2) which rarely occurs during their design life, although they may lose some of their functions. The Hyogo-ken Nanbu Earthquake (one of the level 2 earthquakes) which occurred on January 17th in 1995, however, caused various kinds of serious damage to steel bridge piers such as the collapse of bridge bearings, failure of bridge piers, local buckling, occurrence of brittle cracks, etc.

After the earthquake, many energetic investigations have been carried out for developing the ductile steel bridge piers, which can support superstructures without increasing their elastic strength against such a strong earthquake as the Hyogo-ken Nanbu Earthquake, in research laboratories in universities, governments, public corporations, steel mill companies and bridge fabricators. On the basis of the results of these researches, the seismic design method in the Japanese Specifications for Highway Bridges (JSHB) was revised in December 1996. In this seismic design method, the composite bridge piers of which steel columns are filled with concrete are recommended as one of the most effective and economical ones. No recommendable structures seem to be specified except for the rectangular cross section with the corner plates with regard to ductile steel cross sections [1].

Design guidelines [2] were drafted in the Hanshin Expressway Public Corporation (HEPC) for retrofitting the existing steel bridge piers by referring to the seismic design method in JSHB and these recent research results [3]. According to the seismic design guidelines, the HEPC is strengthening the existing steel bridge piers in which the basement structures of their pier columns are supported with anchor bolts and footing concrete and can not bear against the strong earthquakes (Types Ⅰ and Ⅱ of the level 2) specified in the JSHB if their steel pier columns are filled with concrete, because the strength of their pier columns

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increases substantially and subsequently the applied seismic load exceeds more than that decided by the strength of the basement structures.

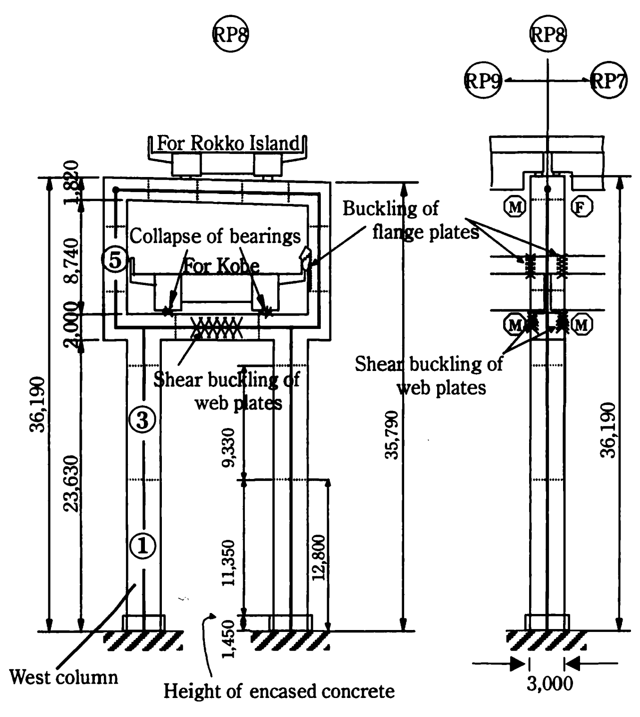
Demonstrated in this paper are the concept of this seismic retrofitting method in the HEPC and a trial design example according to this method by using an actual steel bridge pier of rigid framed structure with two stories and one span [4] in which several web panels around the center part of the lower beam member with box cross section buckled, the bearings supporting the lower bridge collapsed due to the Hyogo-ken Nanbu Earthquake, and subsequently the stiffened flange plate of the west column member in the second story was dented by the collision of the lower bridge.

This seismic retrofitting method is almost verified through the experiment and elasto-plastic finite displacement analyses carried out by the authors. The experimental and analytical results are reported in Ref. 5. The additional investigation to verify the seismic retrofitting method completely is being executed in this year.

2. Bridge pier under consideration and damage due to the Hyogo-ken Nanbu earthquake

The bridge pier [4] under consideration is located on the type Ⅲ ground (weak ground) and designed according to the specifications [6] for bridges provided be HECP in 1992. Concrete is filled into the lower part of the pier columns with rectangular cross section up to the height of 1.45 m from the basement to prevent serious damage caused by the collision of a vehicle.

Figure 1 illustrates the front and side elevations of the pier together with their damage. As this bridge pier is almost symmetric in the front elevation, almost the same axial force due to the earthquake in the direction of the bridge axis is applied to both pier columns. Consequently, treated in this report is only the west column having three types of cross section with different thickness of the component plates; cross section ①, ③ and ⑤.



(a)Front elevation (b)Side elevation

Fig. 1 Bridge pier under consideration and damage due to the Hyogo-ken Nanbu Earthquake (unit:mm)

The main types of damage due to the Hyogo-ken Nanbu Earthquake are summarized as follows:

i) Shearing buckling occurred in the both sides of web plates at the center part of the lower lateral beam with box cross section.

ii) 4 bearings and their sole plates supporting the lower bridge collapsed at Kobe side.

iii) Owing to the above reason, the lower bridge slipped sideways, struck against one of the side column members and caused local buckling on it.

3. Trial check for safety and trial retrofitting of objected bridge pier

The method for retrofitting the bridge pier is based on the Guidelines for Seismic Retrofitting Methods for Existing Steel Bridge Piers against Strong Earthquakes (draft) [2] (afterwards referred to as only the Guidelines) of HEPC.

Mentioned in this section is the outlines of the Guidelines for retrofitting the existing steel bridge pier under consideration.

3.1 Selection of Retrofitting Method

The selection of a retrofitting method is decided according to the Manual (Draft) on Restoration of Highway Bridges under Disaster by Hyogo-ken Nanbu Earthquake [7] as follows:

i) The method of filling concrete should have superior priority for economical and operational reasons.

ii) The weakest part should never be located at the pier base structures.

The method of retrofitting should be, therefore, decided by comparing $M_{u,anc}$ with M_{uc} and M_p , in which

M_{uc} : the ultimate bending moment of the composite cross section consisting of the steel plates and encased concrete, which is calculated on the basis of the Manual (Draft) and the new JSHB, Part V. Seismic Design [1],

M_p : the fully plastic bending moment of the steel cross-section without encased concrete,

$M_{u,anc}$: the ultimate bending moment at the pier base structure, which is calculated as a RC structure consisting of the anchor bolts and the paty of footing concrete subjected to compression according to Ref. 6.

3.2 Restrictions Concerning Cross-sectional Dimensions

To obtain expected ductility in the pier columns, the following conditions on their cross-sectional dimensions shall be regulated [3].

i) In order to prevent the local buckling, the following conditions are specified with regard to the plate slenderness parameters of overall stiffened plate panels R_F, plate panels between longitudinal stiffeners R_R, and longitudinal stiffeners and R_S, respectively.

$$R_F = \frac{B}{t}\sqrt{\frac{\sigma_Y}{E}\frac{12(1-\mu^2)}{\pi^2 \cdot k_F}} \leqq 0.4 \tag{1}$$

$$R_R = \frac{b}{t}\sqrt{\frac{\sigma_Y}{E}\frac{12(1-\mu^2)}{\pi^2 \cdot 4}} = 0.526\frac{b}{t}\sqrt{\frac{\sigma_Y}{E}} \leqq 0.4 \tag{2}$$

$$R_S = \frac{h_s}{t_s}\sqrt{\frac{\sigma_Y}{E}\frac{12(1-\mu^2)}{\pi^2 \cdot 0.425}} = 1.163\frac{h_s}{t_s}\sqrt{\frac{\sigma_Y}{E}} \leqq 0.5 \tag{3}$$

where

μ : Poisson's ratio of steel material (=0.3)
B : width of stiffened plate panel
b : spacing of longitudinal stiffeners ($= B/n$)
n : number of plate panels divided by longitudinal stiffeners
k_F : buckling coefficient of stiffened plate panel defined by

$$\left.\begin{aligned} k_F &= \frac{\left(1+\alpha_t^{\,2}\right)^2 + n\gamma_s}{\alpha_t^{\,2}\left(1+n\delta_s\right)} && (\alpha_t \leqq \alpha_0) \\ k_F &= \frac{2\left(1+\sqrt{1+n\gamma_s}\right)}{1+n\delta_s} && (\alpha_t > \alpha_0) \end{aligned}\right\} \tag{4)a,b}$$

$$\alpha_t = \frac{a}{B} \text{ : aspect ratio of stiffened plate panel} \tag{5}$$

a : spacing of transverse stiffeners (or diaphragms)

$$\delta_s = \frac{A_s}{Bt} \text{ : cross-sectional area ratio of longitudinal stiffener to plate panel} \tag{6}$$

$$\alpha_0 = \sqrt[4]{1+n\gamma_s} \text{ : limit aspect ratio of stiffened plate panel} \tag{7}$$

A_s : cross sectional area of longitudinal stiffener

ii) To satisfy a criterion concerning reinforcement at the corners of the box cross section according to the new JSHB, the next formula is also considered in the Guidelines.

$$\lambda \cdot R_F \cdot \sigma_c / \sigma_Y \leqq 0.02 \tag{8}$$

where

λ :slenderness parameter [8], given by

$$\lambda = \frac{1}{\pi}\sqrt{\frac{\sigma_Y}{E}}\frac{l_e}{r} \tag{9}$$

l_e : effective buckling length (two times of the height of the bridge pier in case of this bridge pier)
r : radius of gyration
σ_c : axial compressive stress due to dead load of superstructures
σ_Y : yield stress of steel material

3.3 Selection of cross section to be retrofitted

Figures 2 and **3** show the cross-sectional configurations ① and ③, for reference [4]. In this paper, the stiffened plate panels parallel to the bridge axis are defined as the flange plate, and the stiffened plate panels perpendicular to the bridge axis as the web plate.

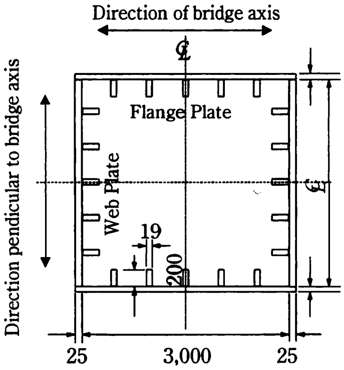


Fig. 2 Cross section ①(unit: mm)

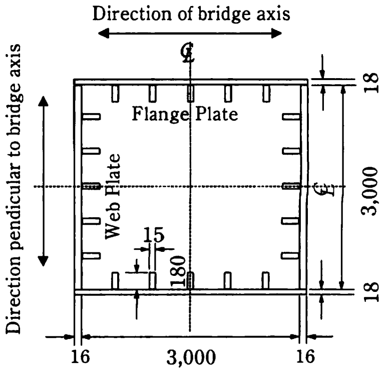
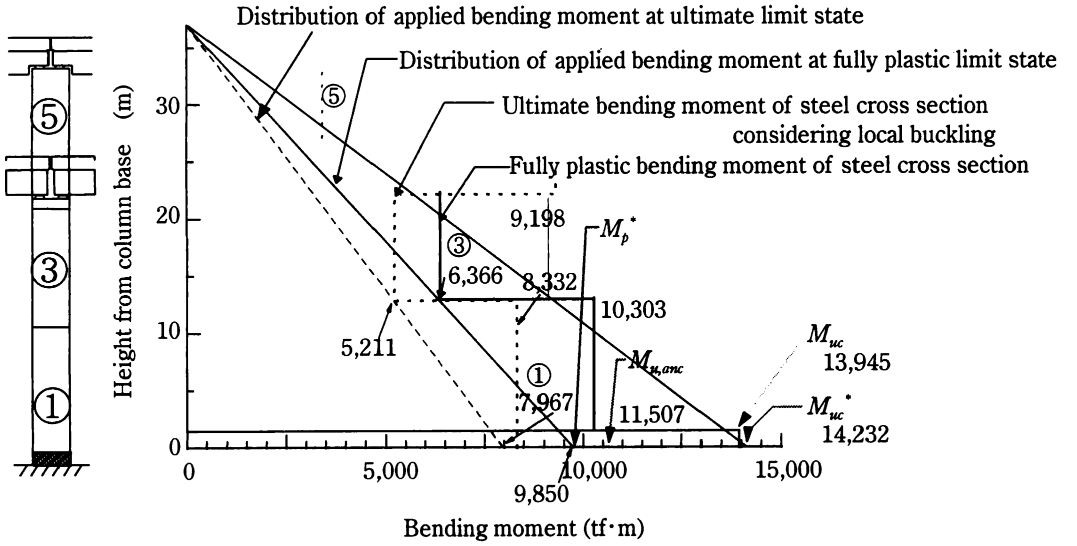


Fig. 3 Cross section ③(unit: mm)

Listed in **Table 1** is the ultimate bending moment of the cross sections together with the other limit strengths and the parameters on the ductility of the cross sections. Besides, **Fig. 4** shows the distributions of the applied bending moment due to two different load levels, the ultimate bending moment and fully plastic bending moment along to the column axis, respectively.

Table 1 Ultimate bending moment, other limit strengths and parameters on ductility of cross sections

Number of cross section / Parameters	Unit	①	③	⑤
R_R	-	0.312	0.489	0.764
R_F	-	0.448	0.488	0.537
σ_c	kgf/cm²	246	347	368
σ_c/σ_Y	-	0.103	0.144	0.102
M_Y	tf·m	8,332	5,211	5,433
σ_u	kgf/cm²	2,400	2,400	3,084
M_u	tf·m	8,332	5,211	4,565



(a) Side elevation (b) Distributions of bending moment

Fig. 4 Distributions of applied, ultimate and fully plastic bending moments

It can be seen from **Fig. 4** that the cross section ③ is the weakest one against earthquake loading compared with the other ones. Even if the bending strength of the cross section ③ increases by about 22 % due to retrofitting and the ultimate bending moment of the section becomes equal to its fully plastic moment, the applied bending moment of the basement after being retrofitted can be less than $M_{u,anc}$ (=11,507 tf·m). Because the relationship $M_p^* < M_{u,anc} < M_{uc}^*$ (=9,850 tf·m) is concluded where the superscript * means the bending moment of the cross section ③ equivalently modified at the level of the column base as shown in **Fig. 4**, a retrofitting method not to fill up concrete into the pier column, i.e., to strengthen the steel cross section must be adopted according to the method mentioned above.

The cross section ③ can be, therefore, decided as the one to be retrofitted. Design calculations for retrofitting the cross section ③ are detailed below.

Table 2 Buckling parameters of cross section ③

Buckling parameters	Web plate	Flange plate	Restriction values
R_R	0.556	0.489	0.4
R_F	0.474	0.488	0.4
R_S	0.653	0.653	0.5
σ_c/σ_Y	0.145		-
λ	0.336	0.625	-
$R_F\lambda\ \sigma_c/\sigma_Y$	0.023	0.043	0.02

Table 2 shows the buckling parameters of the cross section ③, and indicates that all the parameters R_R, R_F, R_S and $R_F\lambda\ \sigma_c/\sigma_Y$ do not satisfy the expected values corresponding to the parameters and that the cross section should be retrofitted.

3.4 Design of Corner Reinforcement

To prevent cracks at the corner parts of the box cross section, the following 5 restrictions can be regulated according to the new JSHB, Part V - Seismic Design Code [1].

i) Two times the width of a plate panel, h_c, in a reinforced corner, shown in **Fig. 5**, shall be taken more than 25 % of the width of the stiffened plate panel, B on each side as follows:

$$2h_c/B \geqq 0.25 \tag{10}$$

where h_c is the length between a corner of the box cross section and the center line of high strengthened bolts used for connecting the corner reinforcing plates.

ii) The slenderness parameter of the corner reinforcing plate, R_C shall satisfy the following condition.

$$R_C = 0.526\frac{b_c}{t_c}\sqrt{\frac{\sigma_Y}{E}} \leqq 0.4 \tag{11}$$

where t_c is the thickness of the corner reinforcing plate as shown in **Fig. 5**.

iii) The geometric moment of inertia of the corner reinforcing plates, I_c with respect to the neutral axis of the steel box cross section shall be taken about 20 % in comparison with that of the overall steel cross section, I_B.

iv) The thickness of the corner reinforcing plate shall be less than 22 mm.

v) The slenderness parameter of the corner plate panels, R_{CR} shall satisfy the following restriction given by Eq. (12), which is regulated newly in this study.

$$R_{CR} = 0.526\frac{h_c}{t}\sqrt{\frac{\sigma_Y}{E}} \leqq 0.4 \tag{12}$$

By using the cross-sectional dimensions shown in **Fig. 5**, the restricting conditions mentioned above are satisfied as listed in **Table 3.**

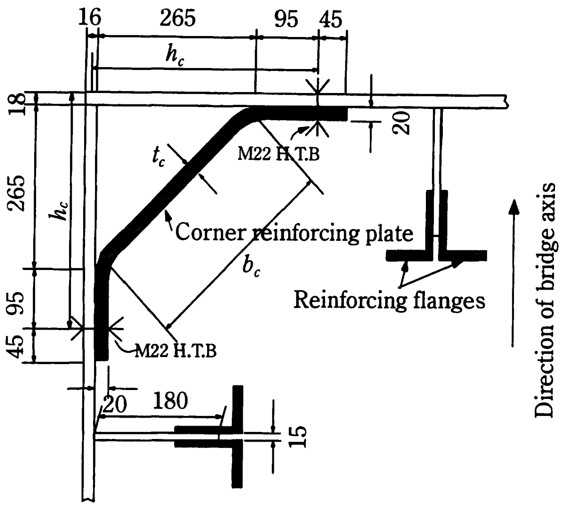


Fig. 5 Corner reinforcement (unit: mm)

Table 3 Parameters on corner reinforcement (cross section ③)

Parameters	Calculated value	Restrictions
$2h_c/B$	0.251	More than 0.25
R_C	0.333	Less than 0.4
I_c/I_B	0.248	Almost equal to 0.2
t_c	20mm	Less than 22mm
R_{CR}	0.369	Less than 0.4

3.5 Retrofitting Based on Restrictions Concerning Cross-Sectional Dimensions

Figure 6 shows the cross section retrofitted on the basis of the above restrictions concerning cross-sectional dimensions.

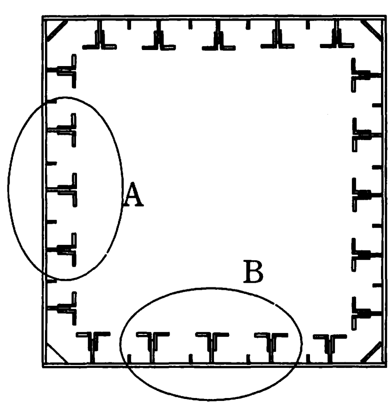


Fig. 6 Cross section of retrofitted bridge pier
(Details of parts A and B are indicated in Figs. 7 and 11, respectively)

In order to make R_R less than 0.4, new longitudinal stiffeners with smaller cross section than the existing ones are added among the existing ones. These small longitudinal stiffeners are defined as the additional longitudinal stiffeners in this paper. Moreover, to make R_S and R_F less than 0.5 and 0.4, respectively, the existing stiffeners are stiffened by new flange plates at their outstanding edges. These additional flange plates are referred to as the reinforcing flange plates. In case that the rigidity of the existing transverse stiffeners becomes insufficient by stiffening the existing longitudinal stiffeners, the transverse stiffeners have to be stiffened by adding new cover flange plates at their tops (hereafter called as the covering flange plates).

(1) Retrofitting calculation of web plates

Figure 7 illustrates the details of the part A shown in **Fig. 6.**

Retrofitting design calculations to the web plates are carried out in the following manner; ① Additional longitudinal stiffeners, ② Reinforcing flange plates, and ③ Covering flange plates.

As the additional longitudinal stiffeners intend to increase the elastic buckling strength of the plate panels among the existing longitudinal stiffeners, and the reinforcing flange plates to increase the elastic buckling strength of both the existing longitudinal stiffeners as a plate and the overall existing stiffened plate panels without substantially increasing the ultimate strength of the pier column, so that they are not connected with the transverse stiffeners or diaphragms (See **Fig. 10**).

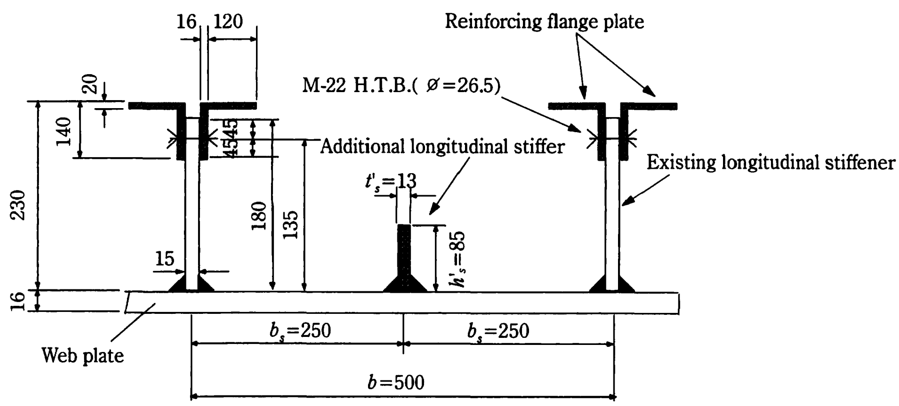


Fig. 7 Details of part A (unit: mm)

a. Additional longitudinal stiffener

i) Check for plate slenderness parameter of additional longitudinal stiffener, R'_S

The plate slenderness parameter R'_S is calculated by Eq. (13) and can be checked to be less than 0.5.

$$R'_s = \frac{h'_s}{t'_s}\sqrt{\frac{\sigma_Y}{E}\frac{12(1-\mu^2)}{k\pi^2}} = 1.613\frac{h'_s}{t'_s}\sqrt{\frac{\sigma_Y}{E}} \quad (\leqq 0.5) \tag{13}$$

where

h'_s : plate width of additional longitudinal stiffener

t'_s : plate thickness of additional longitudinal stiffener

k : buckling coefficient ($k = 0.425$)

ii) Check for plate slenderness parameter of plate panel between existing and additional longitudinal stiffeners, R'_R

The plate slenderness parameter R'_R can be checked through the following equation.

$$R'_R = 0.526\frac{b_s}{t}\sqrt{\frac{\sigma_Y}{E}} \quad (\leqq\ 0.4) \tag{14}$$

where

b_s : spacing between existing and additional longitudinal stiffeners

iii) Check for plate slenderness parameter of sub-stiffened plate panel among existing longitudinal stiffeners, R'_F

The plate slenderness parameter R'_F can be calculated by the following equation and is checked to be less than 0.4.

$$R'_F = \frac{b}{t}\sqrt{\frac{\sigma_Y}{E}\frac{12(1-\mu^2)}{k'_F\pi^2}} \quad (\alpha'_0 < \alpha'_t) \tag{15}$$

where

$\alpha'_t = a\,/\,\mathrm{b}$: aspect ratio of sub-stiffened plate panel (16)

$\alpha'_0 = \sqrt[4]{1+n'\gamma'_s} \quad (\leq \alpha'_t)$: limit aspect ratio of sub-stiffened plate panel (17)

b : spacing among existing longitudinal stiffeners

n' : number of plate panel divided by additional longitudinal stiffeners ($n'=2$ in this design example, because one additional longitudinal stiffener is adopted among the existing longitudinal stiffeners)

γ'_s : flexural rigidity ratio of an additional longitudinal stiffener to plate panel, defined by Eq. (18)

I'_s : geometric moment of inertia of additional longitudinal stiffener with respect to web plate surface, which is defined by Eq. (19)

$$\gamma'_s = \frac{EI'_s}{Db_s} \tag{18}$$

$$I'_s = \frac{t'_s \times h'^3_s}{3} \tag{19}$$

$$D = \frac{Et^3}{12(1-\mu^2)} \quad \text{: flexural rigidity of plate panel} \tag{20}$$

$$k'_F = \frac{2(1+\sqrt{1+n'\gamma'_s})}{1+n'\delta'_s} \quad \text{: buckling coefficient of sub-stiffened plate panel} \tag{21}$$

$$\delta'_s = \frac{A'_s}{bt} \quad \text{: cross-sectional ratio of additional stiffener to plate panel} \tag{22}$$

A'_s : cross sectional area of additional stiffener

b. Reinforcing flange plates

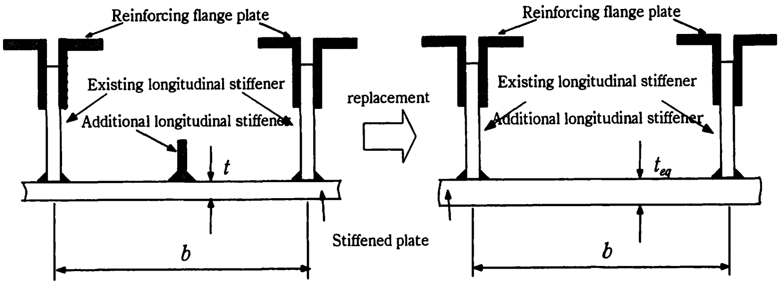
i) Replacement of sub-stiffened plate panel into equivalent unstiffened plate panel

In order to calculate the plate slenderness parameter of the overall retrofitted stiffened plate panel, R_F as easily as possible, it is idealized into a stiffened plate panel with the equivalent thickness of plate panel, t_{eq}, but without the additional longitudinal stiffeners as shown in **Fig. 8.** Then by using this equivalent thickness t_{eq} and Eq. (15), R_F can be checked to be less than 0.4.

The equivalent thickness t_{eq} can easily be derived by equating the plate slenderness of the plate panels of the idealized stiffened plate, R_R to the plate slenderness ratio of the sub-stiffened plate panel as follows:

$$R_R = \frac{b}{t_{eq}}\sqrt{\frac{\sigma_Y}{E}\frac{12(1-\mu^2)}{4.0\pi^2}} = R'_F \tag{23}$$

$$t_{eq} = \frac{b}{R'_F}\sqrt{\frac{\sigma_Y}{E}\frac{(1-\mu^2)}{4.0\pi^2}} \tag{24}$$



(a) Original stiffened plate panel (b) Idealized stiffened plate panel

Fig. 8 Replacement of retrofitted stiffened plate panel into idealized stiffened plate panel

ii) Check for R_F

The plate slenderness parameters R_F can be calculated by using Eq. (25) together with k_F defined by Eq. (4) and $\alpha_t = a / B$, in adoption of the equivalent thickness of idealized stiffened plate panel and is checked to be less than 0.4.

$$R_F = \frac{B}{t_{eq}}\sqrt{\frac{\sigma_Y}{E}\frac{12(1-\mu^2)}{\pi^2 \cdot k_F}} \tag{25}$$

iii) Check for plate slenderness parameters of retrofitted longitudinal stiffener, R_S

The plate slenderness parameters R_S can be checked to be less than 0.5 as follows:

$$R_s = \frac{h_s}{t_s}\sqrt{\frac{\sigma_Y}{E}\frac{12(1-\mu^2)}{k''\pi^2}} \tag{26}$$

in which k'' is given as 3.96(=0.99×4.0) according to Ref. 9 in case where the geometric moment of inertia of the covering flange plates, I_{f1}, exceeds a required value defined by the following equation.

$$I_{f1} = b_w \times t_s^3 \times (1.8315 + 0.3663 \times \frac{A_f}{A_w}) \tag{27}$$

where

$A_f = 2b_f t_f$ and $A_w = b_w t_s$ (See **Fig. 9**)

b_w : distance between the center of bolt axis and the plate surface with existing longitudinal stiffeners

t_s : plate thickness of existing longitudinal stiffener

t_f : plate thickness of reinforcing flange plate

b_f : plate width of reinforcing flange plate

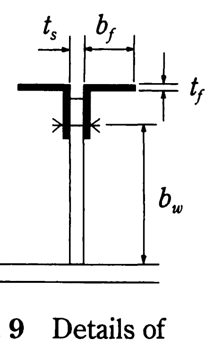


Fig. 9 Details of retrofitting of existing longitudinal stiffener

By using the above definitions, the geometric moment of inertia of the reinforcing flange plate, I_f can be calculated by the following equation.

$$I_f = 2\times\left[\frac{t_f\times b_f^{\ 3}}{12}+t_f\times b_f\times\left(\frac{b_f+t_w}{2}\right)^2\right] \tag{28}$$

c. Covering flange plates

The stiffness of the existing transverse stiffeners and diaphragms must be checked whether their geometric moment of inertia, $I_{t,R}$ and $I_{t,D}$, exceeds the required value, defined by Eq. (29) or not according to Ref. 8.

$$I_{t,req}=\frac{B\cdot t_{eq}}{11}\times\frac{1+n\gamma_s^{\ *}}{4\alpha_t^{\ 3}} \tag{29}$$

where $\gamma_s^{\ *}$ is the minimum required rigidity of the transverse stiffener on the basis of the elastic buckling theory.

In the case of $I_{t,D}$ or $I_{t,R}$ < $I_{t,req}$, the transverse stiffener or diaphragm shall be stiffened to fulfill this condition as shown in **Fig. 10**.

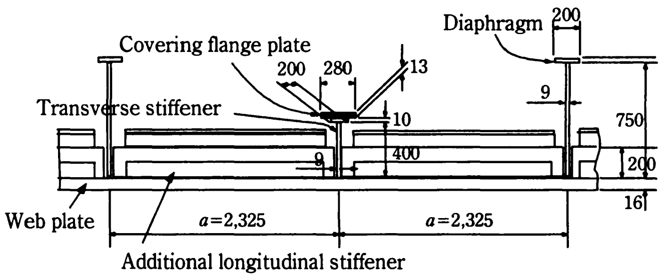


Fig. 10 Reinforcement to transverse stiffener of web plate (unit: mm)

(2) Retrofitting calculation for flange plate

The retrofitting method and their calculation adopted for the web plate can also be applied to the flange plate. The details decided through the calculation are depicted in **Figs. 11 and 12.**

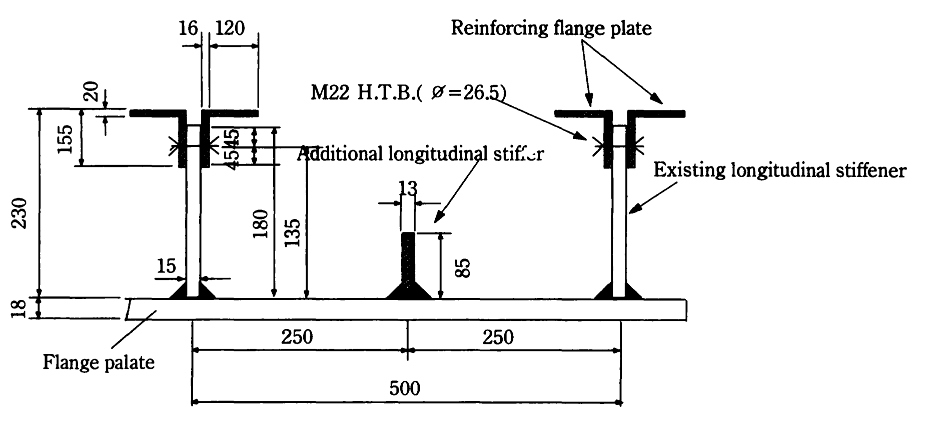


Fig. 11 Details of part B in flange plate (unit: mm)

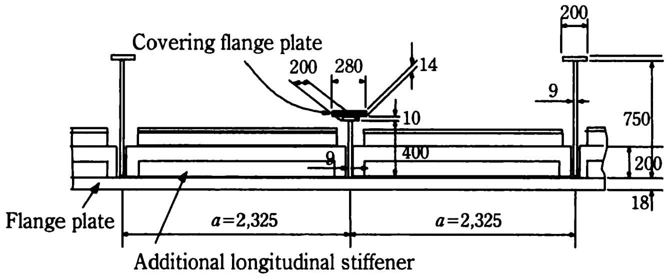


Fig. 12 Reinforcement to transverse stiffener of flange plate (unit: mm)

3.6 Conclusion on Structural Parameters of Bridge Pier Retrofitted

Table 4 indicates the structural parameters of the cross section ③ under consideration, before and after the retrofitting. It is concluded that the structural parameters after retrofitting are satisfied with the limit values of the restrictions concerning the cross-sectional dimensions. As the transverse stiffeners have less flexural rigidity than the required value, the covering flange plate is added onto the flange plate of the existing transverse stiffener to increase its flexural rigidity more than the required one.

Table 4 Structural parameters before and after retrofitting (cross section ③)

(a) Yield stress and cross-sectional constants

Parameters	Before retrofitting	After retrofitting
Yield stress (kgf / cm^2)	2,400 (SS400)	2,400 (SS400)
Geometric moment of inertia (cm^4)	3.852×10^7	6.179×10^7
Section modulus (cm^3)	2.537×10^5	4.070×10^5

(b) Plate slenderness parameters and flexural rigidity of stiffeners

Parameters	Before retrofitting		After retrofitting		Limit value
	Web plate	Flange plate	Web Plate	Flange plate	
R_F	0.474	0.488	0.200	0.191	0.4
R'_F	-	-	0.350	0.330	0.4
R_R	0.556	0.489	0.350	0.330	0.4
R'_R	-	-	0.278	0.247	0.4
R_S	0.653	0.653	0.161	0.161	0.5
R'_S	-	-	0.357	0.357	0.5
γ_s/γ_s^*	1.385	1.001	3.097	3.025	3.0
I_f (cm^4)	-	-	1.106×10^3	1.106×10^3	I_{f1}
I_{f1} (cm^4)	-	-	173	173	-
I_{tD} (cm^4)	263,734	263,734	-	-	$I_{t,req}$
I_{tR} (cm^4)	52,007	52,007	1.202×10^5	1.152×10^5	$I_{t,req}$
$I_{t,req}$ (cm^4)	-	-	6.473×10^4	6.863×10^4	-

4. Conclusions

The concept of the seismic retrofitting method drafted in the Hanshin Expressway Public Corporation for existing steel bridge piers without adopting encased concrete is introduced in this paper. Then, a trial design example according to this seismic design method is shown by using an actual steel bridge pier of rigid framed structure with two stories and one span damaged due to the Hyogo-ken Nanbu Earthquake.

This seismic retrofitting method is almost verified through the experiment and elasto-plastic finite displacement analyses carried out by the authors. The additional investigation to verify the seismic retrofitting method completely is being executed in this year.

Acknowledgements

The authors would like to express their gratitude to the members of the Committee on Seismic Retrofitting Methods for Existing Steel Bridge Piers in the Hanshin Expressway Public Corporation, in particular Mr. T. Nishioka of the Corporation and Mr. S. Okashiro of Newjec, Inc. for their many useful advice and comments in developing the seismic retrofitting method.

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