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Tall bridge pier with superstructure-pier-foundation interaction

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ABSTRACT : A method for the design of a tall bridge pier is presented which considers the pier as divided into two parts. The upper portion is designed as developing a plastic hinge. The lower portion is designed to be subject to only 70% of its moment capacity. This results in an economical foundation. Also, the damage due to the plastic hinge is controllable and repairable. The superstructure slip-off is prevented by controlling the relative rigidities of the upper and lower portions of the pier. Design equations are presented.

1 INTRODUCTION

AASHTO Guide Specifications for Seismic Design of Highway Bridges (1983) permits and recommends the design of bridge piers by accepting plastic hinges to occur in the pier.

Since design forces such calculated are significantly smaller than the modified design forces, the design engineer normally prefers the "Plastic Hinging Method" to achieve an economical design. This judgement may be true for bridge piers of ordinary heights (7000-8000 mm), but when applied to tall bridge piers (45000-65000 mm), safety and economics may be seriously challenged.

Obviously, the cross-section of a tall bridge pier will be large. If this pier is designed to hinge, the damage will be very severe due to the size of the plastic hinge that results, and it may not even be possible to repair this damage per AASHTO requirements. More importantly, depending on the location of the plastic hinge, longitudinal sway of the superstructure may get out of hand which may cause the superstructure to fall-off of its supports, as has been observed in the San Fernando Earthquake. Furthermore, because the moment required to hinge a large pier cross-section is also large, the footing which must receive this moment will necessitate uneconomic dimensions and number of piles.

One possible approach to this design problem is to combine the "Plastic Method" and the "Elastic Method" by considering the pier in two segments, divided somewhere over the height. At this stage, the design engineer is confronted with the following question: What is the height of the optimum location that divides the pier in two segments? This question must be answered by considering the amount of longitudinal sway that occurs after hinging, the P-Δ effect and the magnitude of the moment at the pier-footing connection by which the foundation is to be designed.

2 DESIGN CONSIDERATIONS

The pier will be considered as divided into two segments as shown in Fig.1. The above portion has a reduced cross-section as compared to the lower portion and is expected to hinge at the junction of the top and bottom pier segments. Since the cross-section of the top portion is small, the moment causing plastic hinging will also be small. Consequently, the damage due to hinging will be repairable per AASHTO Specifications. Furthermore, the shear force that will be transferred to the lower portion will be limited by the magnitude of the hinging moment, Fig.2. Thus, the design engineer acquires an effective tool to control the moment on the lower portion of the pier. Now, the lower portion can be subjected to

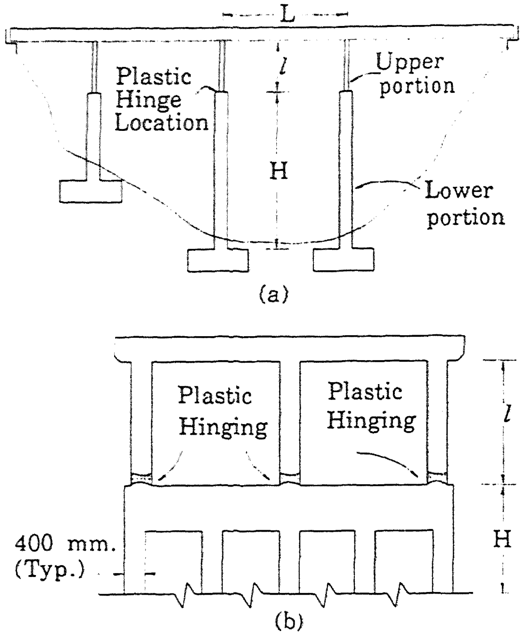

Figure 1. Tall bridge pier composed of two segments. (a) elevation (b) plan

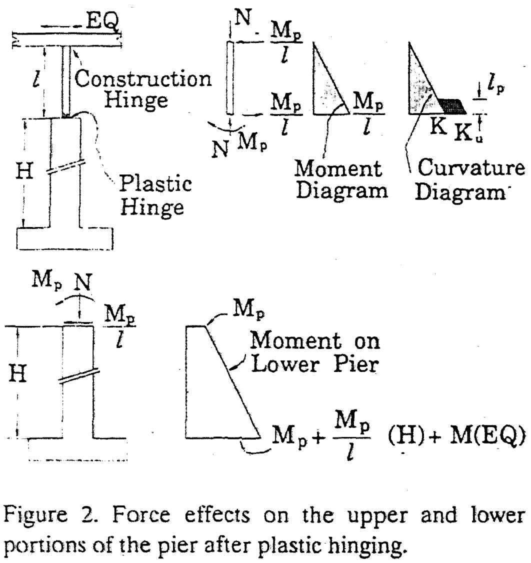

Figure 2. Force effects on the upper and lower portions of the pier after plastic hinging.

only say 70% of the yield moment of its cross-section. The damage due to hinging is avoided and the magnitudes of the force effects on the foundation are controlable.

Table 1. Curvature ductility demands for $\mu = 4$

l_p/l	0.05	0.1	0.15	0.20	0.25	0.30	0.35
K_u/K_y	21.5	11.5	8.2	6.6	5.6	4.9	4.5

2.1 Design length of the upper segment of the pier

Since plastic hinging is accepted to occur at the base of the upper portion, the superstructure will sway considerably, producing the severest P-Δ effect on this upper part. To reduce the magnitude of the P-Δ effect and to avoid excessive damage during the formation of the plastic hinge, it is recommended that the level of the axial load N/N_o be kept at or below 0.2, as stated by Park and Paulay (1975).

The sway of the superstructure after the formation of the plastic hinge can be expressed by using Fig.2. Consequently, the corresponding curvature ductility demands are calculated and tabulated accepting a displacement ductility ratio of $\mu = \Delta_u / \Delta_y = 4$, as done by Park and Paulay (1975), Table 1,

$$\Delta_u = \left(\frac{K_y}{2} \cdot \frac{2l^2}{3}\right) + (K_u - K_y)\, l_p\, (l - 0.5\, l_p) \qquad (1)$$

where Δ_u=sway at the end of the post-yielding range, Δ_y=sway corresponding to yielding, K_y=curvature corresponding to yielding, K_u=curvature at the end of the post-yielding range, l = length of the upper portion of the pier, l_p=equivalent length of the plastic hinge.

Considering that the equivalent length of the plastic hinge l_p is typically in the range of 0.5-1.0 times the depth of the cross-section h, and K_u / K_y ratio for a well detailed cross-section is between 10-20, assume $K_u / K_y = 11.5$ as a conservative value, Table 1. Assume also

$$l_p \cong h \qquad h / l = 0.1 \qquad l / h = 10 \qquad (2)$$

where h = cross-sectional dimension in the bending direction. The other design criterium that $N/N_o = 0.2$ can now be made use of to obtain the gross concrete area of the upper portion of the pier.

$$N_o = 0.85\, f_{ck}\, A_c \left(1 + \rho \frac{f_{yk}}{f_{ck}}\right) \qquad (3)$$

$$A_c = \frac{N}{0.17\, f_{ck}\,(1+\rho \frac{f_{yk}}{f_{ck}})} \qquad (4)$$

where A_c = gross concrete area of the upper portion of pier, f_{ck} = characteristic strength of concrete, f_{yk} = characteristic yield stress of steel, ρ= ratio of steel,

The axial load on the column is known and thus the gross concrete area A_c and consequently the dimensions of the desired cross-section can be calculated by Eq.(4). The length of the upper portion of the pier is readily determined by Eq.(2).

2.2 Design of the lower portion of the pier

The lower portion of the pier is subject to a plastic moment M_p and a shear force of M_p/ℓ as shown in Fig.2. The design moment at the pier-foundation intersection will be composed of M_p', the moment produced by the constant shear force $M_p(H)/\ell$ and the moment M(EQ) due to own mass of the pier,

$$M(\text{Design}) = M_p' + \frac{M_p'}{\ell}\,(H) + M(EQ) \qquad (5)$$

where M_p' = moment that causes plastic hinging of the upper portion increased by the overstrength factor, ℓ = length of the upper portion, H = height of the lower portion of the pier and M(EQ)= moment produced by the design earthquake due to own mass of the pier.

This design moment should not force the lower portion of the pier to yielding for reasons of damage control, stability and foundation economics. If permitted to yield, the level of damage due to hinging of a large cross-section will be unacceptable and almost impossible to repair. Also, the second plastic hinge will severely challenge the overall stability because the bridge is already a mechanism due to the plastic hinge already formed at the base of the upper portion of the pier. Furthermore, the foundation will have to be designed for the yield moment of the lower portion increased by the overstrength factor which will result in a more expensive foundation.

Now that the design moment on the bottom portion of the pier is known, the cross- section can be designed. At this stage the designer has to make a decision about the level at which the lower pier moment capacity is to be taxed. This is obviously a choice which depends on the safety margin and economics desired. The author prefers to use M(Design) / M(pier) = 0.7 which is a comparable value of 0.75 as has been used for the Pine Valley Creek Bridge in California (1979).

To fix the cross-sectional dimensions and the steel area for the bottom portion of the pier, some trial and error is necessary. It is more productive to fix the cross-sectional dimensions in conformance with the method of construction (usually slip forming) and play with the amount and distribution of reinforcement. In this effort, an efficient thrust-moment-curvature computer program is indispensable, that will also yield a measure of the available curvature ductility of the cross-section.

2.3 Design of the foundation

The foundation is designed for 70% of the yield moment of the pier and the axial forces resulting from the proper load group per AASHTO (1989). The danger of plastification at the pier-foundation intersection is eliminated. Severe congestion of steel required by AASHTO to confine the hinging zone is not needed. No failure can occur under the grade level where it can go unnoticed. All in all, a safe and an economic foundation has been achieved.

2.4 Design against superstructure slip-off

Many bridges have fallen off of their supports during the past San Fernando Earthquake as reported by Fung, et.al (1971). In order to design against superstructure slip-off, the longitudinal sway must be carefully calculated and consequently controlled.

Considering the pier as divided into two segments over the height, the longitudinal sway is made up of the contributions from the upper and

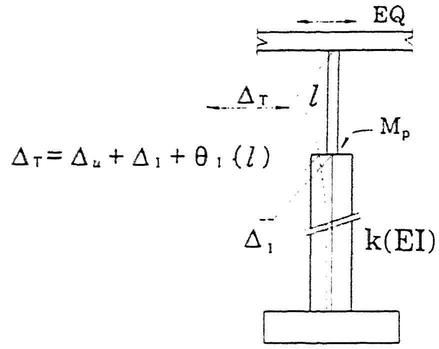

Figure 3. Components of the total sway of the superstructure

Table 2. The rigidity of the bottom portion as a multiple of the upper portion.

H/ℓ	1.0	2.0	3.0	4.0	5.0
k	18	65	158	311	339

H/ℓ	6.0	7.0	8.0	9.0	10.0
k	857	1281	1825	2504	3333

For the definition of k, see Eq.4.

lower portions, Fig.3. The upper portion will hinge at its base and will constitute the main part of the longitudinal sway. The lower segment is the enlarged portion on which no plastification occurs and can be considered in the "elastic" range. As stiff as it may be, it also shows a longitudinal displacement which must be limited to a fraction of the total sway of the upper part if it is to act as a restraint.

Sway of the upper part due to the plastic hinge at the base is a follows :

$$\Delta_u = \left(\frac{K_y}{2} \cdot \frac{2\ell^2}{3}\right) + (K_u - K_y)\ell_p (\ell - 0.5\ell_p) \tag{6}$$

$$K_y = \frac{M_p}{EI} \tag{7}$$

$$K_u - K_y = K_y \left(\frac{K_u}{K_y} - 1\right) \tag{8}$$

Using the conditions of Eq. (2) and simplifying

$$\Delta_u = 1.33 \frac{M_p}{EI} (\ell)^2 \tag{9}$$

The bottom part of the pier has a rigidity of k(EI) and is subject to the force effects as shown in Fig.2. The total superstructure sway can be calculated as follows where Δ_1 and Θ_1 are the sway and slope at the top of the lower portion of the pier, respectively.

$$\Delta_T = \Delta_u + \Delta_1 + \Theta_1 (\ell) \tag{10}$$

$$\Delta_1 = \frac{M_p (H)^2}{2EI\,(k)} + \frac{M_p (H)^3}{3EI(\ell)\,(k)} \tag{11}$$

$$\Theta_1 (\ell) = \frac{M_p H(\ell)}{2EI\,(k)} + \frac{M_p (H)^2}{2EI\,(k)} \tag{12}$$

$$k = \frac{\text{EI (bottom portion)}}{\text{EI (top portion)}} \tag{13}$$

The design engineer wants to limit the magnitude of the sway due to the lower portion ($\Delta_1 + \Theta_1\ \ell$) to a fraction of the sway of the upper portion, Δ_u. The authors prefer to use

$$\frac{\Delta_1 + \Theta_1 (\ell)}{\Delta_u} = 0.1 \tag{14}$$

as a reasonable ratio. This condition yields the required rigidity of the bottom portion as a multiple of the rigidity of the upper portion as expressed by k, Table 2.

$$k = 2.5 \left[\left(\frac{H}{\ell}\right) + \left(\frac{H}{\ell}\right)^2 + \left(\frac{H}{\ell}\right)^3\right] \tag{15}$$

The bottom portion will be designed to have the rigidity as indicated in Table 2. This will ensure that the bottom portion will contribute only a small amount to the total sway of the superstructure.

3 CONCLUSIONS

A procedure for the design of tall bridge piers is presented. The procedure considers superstructure-pier-foundation interaction. The basic features of the design method are as follows:

a) The tall pier is divided into two segments.
b) The upper portion has a reduced cross-section and is designed to hinge at its base.
c) The required cross-sectional area of the upper

portion will have a mechanical slenderness ratio of about 10 and level of axial load of 0.2.

d) The magnitudes of the force effects on the bottom portion are limited by the formation of the plastic hinge.

e) The bottom portion will be designed such that the design moment is only 70% of the cross-sectional moment capacity.

f) Superstructure slip-off is prevented by providing enough stiffness to the lower pier segment by making use of the relationships presented. This also ensures structural stability.

g) The resulting size of the foundation is small and economical.

REFERENCES

Fung, G.G., Le Beau, R.F., E.D. Belvedere and Goldschmit A.G. 1971. Field investigation of bridge damage in the San Fernando earthquake. State of California Business and Transportation Agency.

Podolny, W., Jr., and Muller, J.M., Construction and design of prestressed concrete bridges. John Wiley & Sons, New York: 46-52.

Atimtay, E., 1991. Seismic design of bridges to relate ductility and sway. ACI International Conference on Rehabilitation of Concrete Structures and Innovations in Design, Hong Kong: 1243 - 1255.

Park, R., and Paulay, T., 1975. Reinforced concrete structures. John Wiley & Sons, New York.

American Association of State Highway and Transportation Officals, 1983. Guide specifications for seismic design of highway bridges, Washington D.C.