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Displacement-based energy dissipation systems for steel bridges diaphragms

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Abstract

A seismic design strategy that relies on ductile end-diaphragms inserted in the steel superstructure can be, in some instances, an effective alternative to energy dissipation in the substructure. This could be the case, for example, when stiff wall-piers that can difficultly be detailed to have a stable ductile response are used as substructures. Such a ductile diaphragms concept was originally developed for the seismic retrofit of steel slab-on-girder and deck-truss bridges. For application in new bridges, the proposed retrofit methodologies were revised and convert into design procedures. This paper provides an overview of these design procedures developed as part of an NCHRP project. © 2002 Elsevier Science Ltd. All rights reserved.

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1. Introduction

Research was conducted to develop and experimentally validate the concept of ductile diaphragms for the seismic retrofit of steel slab-on-girder and deck-truss bridges [1–6]. Detailed retrofit design methodologies were formulated as part of this

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work. Interestingly, the proposed ductile diaphragm concept generated a considerable interest for application in new bridges, as a cost-effective potential solution to achieve the ductile response of these types of bridges. However, to address this new interest, adjustments were required to revise the proposed retrofit methodologies and convert them into design procedures.

The opportunity to conduct such work was provided to the first author as part of an AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) project to develop a comprehensive specification for the seismic design of bridges that would include the latest knowledge about the seismic performance of bridges. This comprehensive specification was prepared by a team of practicing engineers and researchers under a joint-venture partnership of the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER). One sub-task of this project consisted of the development of seismic design requirements for steel bridges. Note that the current AASHTO Specification does not have seismic requirements for these bridges, except for the provision of a continuous load path to be identified and designed (for strength) by the engineer. Consequently, within the scope of this project sub-task, a comprehensive set of special detailing requirements for steel components expected to yield and dissipate energy in a stable and ductile manner during earthquakes was developed. These included, in compliance with the proposed Specifications' intent to permit the use of innovative systems, provisions for a few "special systems", such as ductile diaphragms.

It is the objective of this paper to provide an overview of the design procedures for ductile diaphragms in slab-on-girder bridges and deck-truss bridges developed as part of this NCHRP project. However, the reader is cautioned that these remain only provisional at the time of this writing. Because the proposed new Seismic Provisions for the AASHTO LRFD Bridge Design Specifications (which include the design procedure presented here, in Appendices to the seismic steel design provisions) contain a considerable amount of changes from the current AASHTO seismic provisions, it is expected that a separate non-mandatory Guide Specification prepared from these proposed Specifications will be used for a few years until the States' Departments of Transportation develop enough familiarity to adopt the proposed seismic provisions as a mandatory part of the AASTHO Design Specifications.

2. Design specifications

For special steel energy dissipation systems less familiar to bridge engineers, the approach taken in the proposed Specifications (in accordance with AASHTO's rules) has been to provide in *Articles* only the minimum considerations that must be addressed for their design. The *Commentary* provides some explanations on the purpose of these minimum considerations, and *Appendices* provide detailed step-by-step procedures for the design of these systems.

As such, for slab-on-girder bridges, articles state:

Ductile end-diaphragms in slab-on-girder bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

(a) *Specially detailed diaphragms capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;*
(b) *Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;*
(c) *Design considers the combined and relative stiffness and strength of end-diaphragms and girders (together with their bearing stiffeners) in establishing the diaphragms strength and design forces to consider for the capacity-protected elements;*
(d) *The response modification factor to be considered in design of the ductile diaphragm is given by:*

$$R = \left| \frac{\mu + \dfrac{K_{DED}}{K_{SUB}}}{1 + \dfrac{K_{DED}}{K_{SUB}}} \right| \qquad (1)$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrate otherwise, μ should not be taken greater than 4;
(e) *All details/connections of the ductile end-diaphragms are welded.*
(f) *The bridge does not have horizontal wind-bracing connecting the bottom flanges of girders, unless the last wind bracing panel before each support is designed as a ductile panel equivalent and in parallel to its adjacent vertical end-diaphragm.*
(g) *An effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the slab to the diaphragm.*

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be based on available experimental research results.

The Commentary indicates that ductile diaphragm strategy is not effective when the substructure is significantly more flexible than the superstructure, and that bridges having wide piers, wall-piers, or other substructure elements of similar limited ductility, would be good candidates for the implementation of the ductile diaphragm system. In these cases, the ductile diaphragms could also be designed to yield instead of the bridge piles, thus preventing the development of damage below ground level where it cannot be easily inspected following an earthquake. Because the contribution of girders can be significant and cannot be neglected, ductile diaphragm are generally more effective in longer span bridges, and may be of limited benefit for short span bridges.

It is also emphasized that the inertia forces attributable to the mass of the pier-cap will be resisted by the substructure, in spite of the presence of ductile diaphragms, and that refined analyses should consider this condition if that mass is a significant portion of the total superstructure mass.

For deck-trusses, sensibly the same procedure is proposed, with the exception that items (c), (e), (f), (g) are replaced by the requirements that:

(a) *The last lower horizontal cross-frame before each support is also designed as a ductile panel equivalent and in parallel to its adjacent vertical end-diaphragm;*
(b) *Horizontal and vertical energy dissipating ductile panels are calibrated to have a ratio of stiffness approximately equal to their strength ratio;*
(c) *The concrete deck is made continuous between supports (and end-diaphragms), and an effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the deck to the diaphragms;*
(d) *All capacity-protected members are demonstrated able to resist without damage or instability the maximum calculated seismic displacements.*

This recognizes that while ductile diaphragms in slab-on-girder and deck-truss bridges share many conceptual similarities, seismic forces in deck-trusses follow a more complex and redundant load-path. This requires the use of ductile diaphragms vertically over the supports as well as horizontally in the last lower horizontal cross-frame before each support.

The proposed design procedures for ductile diaphragms in both bridge systems considered follow. Note that further research may eventually allow some of the limits currently imposed to be relaxed.

3. Design procedure for ductile end-diaphragms in slab-on-girder bridges

The ductile diaphragms considered here are therefore those that can be specially designed and calibrated to yield before the strength of the substructure is reached (substructural elements, foundation, and bearings are referred generically as “substructure” here). Many types of systems capable of stable passive seismic energy dissipation could be used for this purpose. Among those, eccentrically braced frames (EBF) [7,8], shear panel systems (SPS) [9,10], and steel triangular- plate added damping and stiffness devices (TADAS) [11], popular in building applications, have been studied for bridge applications [1,2,3,4]. These are illustrated in Figs. 1–3. Although concentrically braced frames can also be ductile, they are not admissible here because they can often be stronger than calculated, and their hysteretic curves can exhibit pinching and some strength degradation.

Note that the plate girders can also contribute to the lateral load resistance, making the end-diaphragm behave as a dual system. Therefore, the lateral stiffness of the stiffened girders, ΣK_g, must be added to the stiffness of the ductile diaphragms,

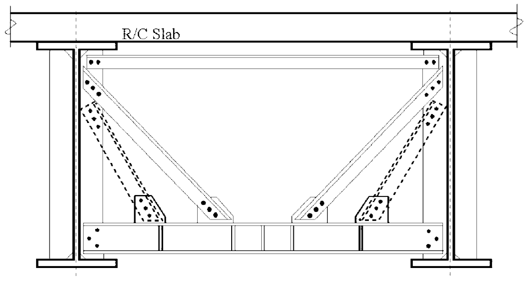

Fig. 1. EBF ductile diaphragms.

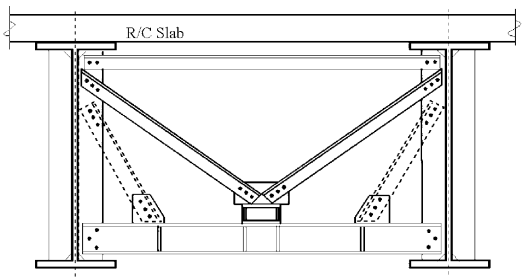

Fig. 2. SPS ducitle diaphragms.

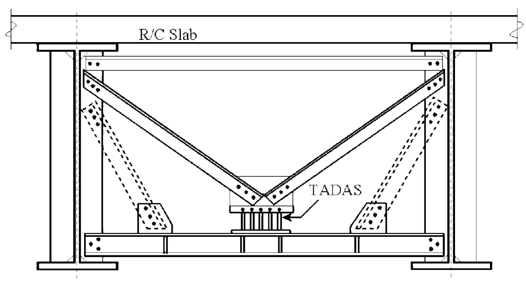

Fig. 3. TADAS ductile diaphragms.

ΣK_{DD} (usually much larger than the former), to obtain the lateral stiffness of the bridge end-diaphragms (adding the stiffnesses of both ends of the span), K_{ends}, i.e:

$$K_{ends} = \Sigma K_{DD} + \Sigma K_g \tag{2}$$

The stiffness contribution of a plate girder is obviously a function of the fixity provided to its top and bottom flanges by the deck slab and bearing respectively. If full fixity is provided at both flanges of the plate girder,

$$K_g = \frac{12EI_g}{h_g^{\,3}} \tag{3}$$

where I_g is the moment of inertia of the stiffened stub-girder (mainly due to the bearing web stiffeners) in the lateral direction, and h_g is its height. If one end is fully fixed, the other one pinned,

$$K_g = \frac{3EI_g}{{h_g}^3} \tag{4}$$

If both ends effectively behave as pin supports, K_g=0. Full fixity at the deck level in composite bridges is possible if shear studs are closely spaced and designed to resist the pull-out forces resulting from the moments developed at the top of the girders under lateral seismic forces. As for fixity at the bearing level, it obviously depends on the type of bearings present. However, even when infinitely rigid bearings are present, full fixity is still difficult to ensure due to flexibility of the girder flanges, as revealed by finite element analyses of subassemblies at the girder-to-bearing connection point.

It is the engineer's responsibility to determine the level of fixity provided at the ends of the girders. However, contrary to conventional design, the most conservative solution is not obtained when zero fixity is assumed because fixity also adds strength to the diaphragms, and the role of the ductile diaphragms is to limit the magnitude of the maximum forces that can develop in the substructure.

The lateral stiffness of the ductile diaphragms, $K_{DD,}$ depends on the type of ductile device implemented. For example, if a ductile SPS is used, the stiffness of one such end-diaphragm in a slab-on-girder bridge, K_{SPS}, can be obtained by:

$$K_{SPS} = \frac{E}{\frac{l_b}{2A_b\cos^2\alpha} + \frac{L_s}{4A_{bb}} + \left(\frac{{h_l}^3}{3I_l} + \frac{2.6h_l}{A_{s,l}}\right) + \frac{L_s(h_l + d_{bb}/2)^2}{12I_{bb}} + \frac{H\tan^2\alpha}{2A_g}} \tag{5}$$

where E is the modulus of elasticity, l_b and A_b are the length and area of each brace, α is the brace's angle with the horizontal, L_s is the girder spacing, d_{bb}, A_{bb} and I_{bb} are the depth, cross sectional area and moment of inertia for the bottom beam, h_l, I_l and $A_{s,l}$ are the length, moment of inertia and shear area of the link, and H and A_g are the height and area of the stiffened girders.

Similarly, lateral stiffness of the EBF and TADAS implemented as end-diaphragms of slab-on-girder bridges, K_{EBF} and K_{TADAS}, can be computed as follows:

$$K_{EBF} = \frac{E}{\frac{l_b}{2A_b\cos^2\alpha} + \frac{a}{2A_l} + \frac{e^2H^2}{12L_sI_l} + \frac{1.3eH^2}{aL_sA_{s,l}} + \frac{H\tan^2\alpha}{2A_g}} \tag{6}$$

$$K_{TADAS} = \frac{E}{\frac{l_b}{2A_b\cos^2\alpha} + \frac{L_s}{4A_{bb}} + \frac{6{h_T}^3}{Nb_T{t_T}^3} + \frac{L_s(h_T + d_{bb}/2)^2}{12I_{bb}} + \frac{H\tan^2\alpha}{2A_g}} \tag{7}$$

where a is the length of the beam outside the link, e, I_l, A_l and $A_{s,l}$ are the length, moment of inertia, cross sectional and shear areas of the link, N, h_T, b_T, and t_T are the number, height, width and thickness of the TADAS plates, and all other parameters are as defined previously. Note that of the five terms in the denominator of Eqs. (5)–(7), the second and fifth which account for axial deformations of bottom beam and stiffened girders could be ignored, and the fourth (accounting for the rotation of bottom beam at midspan in SPS and TADAS) could have a small impact if the bottom beam was a deep and stiff beam, which is not however always the case.

For a bridge having a given number of girders, n_g, number of end-diaphragms implemented at each support, n_d, and girder spacing, L_s, the design procedure for a ductile diaphragm consists of the following steps (illustrated in Fig. 4):

1. Determine the elastic seismic base shear resistance, V_e, for one end of the bridge (half of equivalent static force).
2. Calculate $V_{inel} = V_e / R$, where V_{inel} is the inelastic lateral load resistance of the entire ductile diaphragm panel at the target reduction factor, and R is the force reduction factor calculated as per Eq. (1). Note that μ in that equation represents the ductility capacity of the ductile diaphragm as a whole, not the local ductility of the ductile device that may be implemented in that diaphragm.
3. Determine the design lateral load, V_d, to be resisted by the energy dissipation device (e.g. link beam or TADAS) at the target ductility level, by:

$$V_d = \frac{V_{inel} - n_g V_g}{n_d} \tag{8}$$

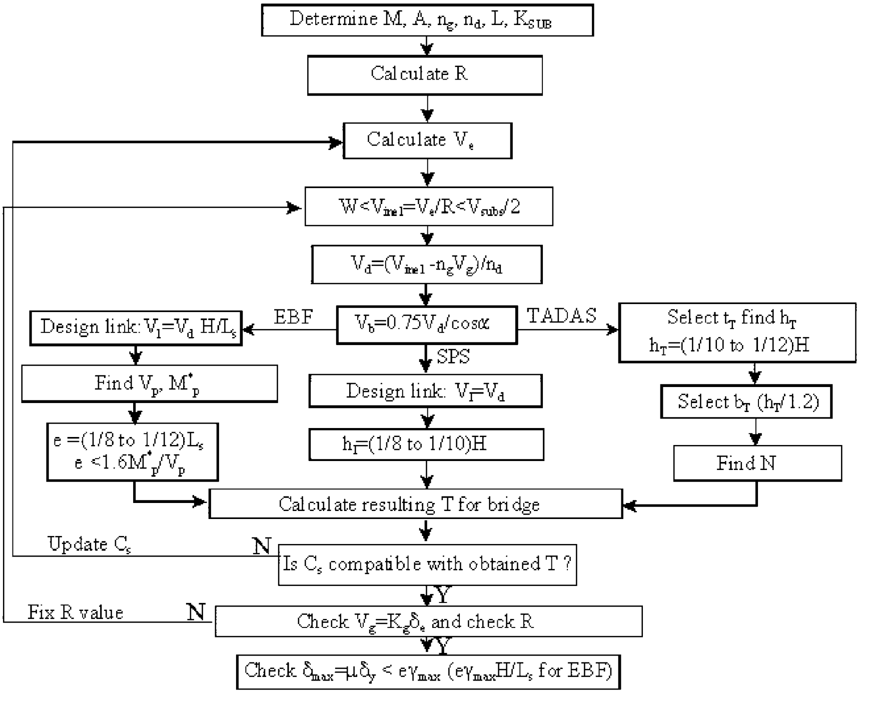

Fig. 4. Flow chart of design proces for ductile diaphragm.

where V_g is the lateral load resistance of one stiffened girder. Note that in short bridges, V_g can be a dominant factor that could overwhelm the resistance contribution provided by the special ductile diaphragm elements. In that perspective, it is recommended in this procedure that the bearing stiffeners at the support of these girders be trimmed to the minimum width necessary to satisfy the strength and stability requirements. Ideally, the braced diaphragm assembly should also be 5 to 10 times stiffer than the girders with bearing web stiffeners (even though ductility demand tends to be larger in stiffer structures) to prevent, or at least minimize, yielding in the main girders under transverse displacements. Note that in longer bridges, particularly those with a lesser number of girders per cross-section, the contribution of the girders to lateral load resistance is nearly insignificant.

4. Design all structural members and connections of the ductile diaphragm, with the exception of the seismic energy dissipation device, to be able to resist forces corresponding to $1.5V_d$ to account for potential overstrength of the ductile device due to strain hardening, strain rate effects and higher than specified yield strength. For example, braces should be designed to resist an axial compression force, V_b, equal to:

$$V_b = 1.5\left(\frac{V_d}{2\cos\alpha}\right) = 0.75\frac{V_d}{\cos\alpha} \tag{9}$$

Likewise, for the SPS and TADAS systems, the bottom beam should be designed to resist a moment equal to $1.5\ V_d\ \mathrm{h_l}$ or $1.5\ V_d\ \mathrm{h_T}$. Moreover, for a given SPS or TADAS device, it is also advantageous to select a flexurally stiff bottom beam to minimize rigid-body rotation of the energy dissipating device and thus maximize hysteretic energy at a given deck lateral displacement.

5. Design the energy dissipating device. For the link beam in an EBF end-diaphragm, the shear force V_l in the link is:

$$V_l = \frac{H}{L_s}V_d \tag{10}$$

The plastic shear capacity V_p of a wide flange steel beam is given by:

$$V_p = 0.58F_y t_w d_l \tag{11}$$

where F_y is the yield stress of steel, t_w is the web thickness, and d_l is the depth of the beam. The moment simultaneously applied to the link must be less than the reduced moment capacity, $M_p{}^*$, of the link yielding in shear and equal to [7]:

$$M_p^* = t_f b_f F_y (d_l - t_f) \tag{12}$$

Since shear links are more reliable energy dissipators than flexural links [8,12], shear links are favored and their length is therefore limited by the equation below:

$$e < e_{\max} = 1.6\frac{M_p{}^*}{V_p} \tag{13}$$

A link length, e, of 1/8 to 1/12 of the girder spacing, L_s, is recommended for preliminary design, the less restrictive value preferred for practical reasons (i.e. detailing constraints) in the presence of closely spaced girders. Deeper link beams are also preferred as the resulting larger flexural stiffness enhances the overall stiffness of the ductile device, ensuring that its yield displacement is reached much before onset of yielding of the stiffened girders.

For an SPS, the above procedure would be followed with the obvious exception that V_l=V_d and the height of panel should be limited to half of the value obtained by the above equation since the yielding link is only in single curvature, as opposed to double curvature for the EBF. A link height of 1/8 to 1/10 of the girder depth is recommended for preliminary design. However, for a TADAS system, replace step 5 with step 6:

6. Select a small plate thickness, t_T, based on available plate size. The shear strength, V_T, and the stiffness, K_T, of a TADAS device can be determined from [11]:

$$V_T = \frac{Nb_T t_T^2 F_y}{4h_T} \tag{14}$$

$$K_T = \frac{NEb_T t_T^3}{6h_T^3} \tag{15}$$

where N, b_T, t_T and h_T are the number, base width, thickness and height of the triangular steel plates. The ratio of the above equations directly provides a relationship between h_T and t_T:

$$h_T = \sqrt{\frac{2Et_T V_T}{3F_y K_T}} \tag{16}$$

Here, V_T =V_d and a h_T of *H/10* to *H/12* is recommended. Hence, if a reasonable estimate of the desirable K_T for the TADAS device is possible, t_T can be determined directly from h_T. In turn, b_T can be chosen knowing that triangular plates with aspect ratio, h_T/b_T, between 1 and 1.5 are better energy dissipators, based on experimental results [11]. Finally, N can then be calculated. Small adjustments to all parameters follow as N is rounded up to the nearest whole number. Incidentally, many different yet appropriate TADAS systems could be designed within these constraints. Systems with thinner steel plates perform better.

7. Calculate the stiffness of the ductile end-diaphragm by using the equation presented earlier in this commentary. Review the assumed lateral period of the bridge, T, and update calculation as necessary.
8. For the maximum lateral drift of the bridge at the diaphragm location, δ_{max}, check that the maximum ductility capacity of ductile device is not exceeded. For shear links, this is commonly expressed in terms of the maximum link deformation angle, $\gamma_{\max}$ (easily obtained by dividing the maximum relative displacements of link ends by the link length), the maximum drift for the SPS and EBF diaphragms is respectively limited to:

$$\delta_{\max} < e\gamma_{\max} \tag{17}$$

$$\delta_{\max} < \frac{eH}{L_s}\gamma_{\max} \tag{18}$$

with generally accepted $\gamma_{\max}$ limits of 0.08 [12]. Note that, for the SPS diaphragms, the following alternative equation accounting for the rotation of bottom beam at the link connection may be more accurate when this factor has an important impact:

$$\delta_{\max} < e\left(\gamma_{\max} + \frac{V_d L_s(h_l + d_{bb}/2)}{12EI_{bb}}\right) \tag{19}$$

Should these limits be violated, modify the link's depth and length as well as the stiffness of the EBF or SPS diaphragm as necessary, and repeat the design process. Finally, a maximum drift limit of 2% of the girder height is also suggested here, at least until experimental evidence is provided to demonstrate that higher values are acceptable.

Note that the ductile energy dissipating elements should be laterally braced at their ends to prevent out-of-plane instability. These lateral supports and their connections should be designed to resist 6% of the nominal strength of the beam flange [12], i.e. $0.06F_y\ t_f\ b_f$. In addition, to prevent lateral torsional buckling of beams in the SPS, EBF, and TADAS end-diaphragms, the unsupported length, L_u, of these beams shall not exceed $200b_f/\sqrt{F_y}$ where b_f is the width of beam flange in metre and F_y is the yield strength of steel in MPa.

4. Design procedure for ductile end-diapragms in deck-trusses

Similarly to the procedure described above, a seismic design strategy that relies on ductile end-diaphragms inserted in the steel superstructure of deck-truss bridges can be, in some instances, an effective alternative to energy dissipation in the substructure. Instances for which this solution would be effective are the same as for the slab-on-girder case above (i.e. wall-piers, etc.). Again, ductile diaphragms considered here are only those that can be specially designed and calibrated to yield before the strength of the substructure is reached.

Seismically generated inertia forces in deck-trusses can follow two possible load paths from the deck to the supports. As a result, to implement the ductile diaphragm strategy in such bridges, it is necessary to locate yielding devices in both the end-cross frames and in the lower end panels adjacent to the supports. This is illustrated in Fig. 5.

The methodology described in this Appendix is limited to simply supported spans of deck trusses. Until further research demonstrates otherwise, the design concept currently also requires stiffening of the top truss system, which can be achieved by making the concrete deck continuous and composite. This stiffening of the top truss system has two benefits. First, for a given deck lateral displacement at the supports, it reduces mid-span sway, resulting in lower forces in the interior cross-frames.

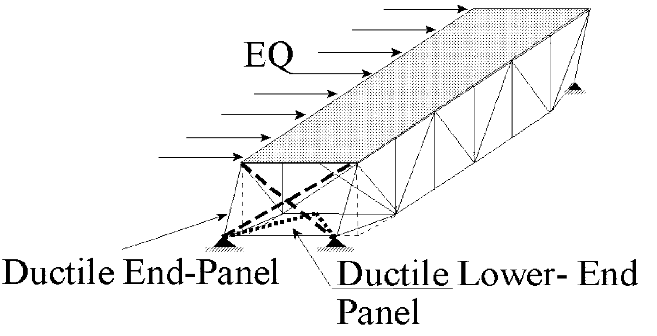

Fig. 5. Ductile diaphragm concept in deck-trusses.

Second, it increases the share of the total lateral load transferred through the top load path.

Note that the design strategy presented here only provides enhanced seismic resistance and substructure protection for the component of seismic excitation transverse to the bridge, and must be coupled with other devices that constraint longitudinal seismic displacements, such as simple bearings strengthening, rubber bumpers and the like.

Under transverse earthquake excitation, end-diaphragms are designed to be the only energy dissipation elements in these bridges. The remaining structural components must be designed to remain elastic (i.e. capacity protected). Some restrictions on stiffness are necessary to prevent excessive ductility demands in the panels and excessive drift and deformations in other parts of the superstructure. The engineer must identify the displacement constraints appropriate to specific bridges; these will vary depending on the detailing conditions germane to the particular bridge under consideration. Generally, among those limits of important consequences, the maximum permissible lateral displacement of the deck must not exceed the values at which:

- P-Δ effects cause instability of the end verticals during sway of the end panel or damage to the connections of the end verticals;
- Unacceptable deformations start to develop in members or connections of the deck-truss, such as inelastic distortion of gusset plates, premature bolt or rivet failures, or damage to structural members;
- The energy dissipating devices used in the ductile panels reach their maximum deformation without loss of strength. This requires, for each type of energy dissipating devices considered, engineering judgement and experimental data on the device's ultimate cyclic inelastic performance, often expressed by a consensus opinion. For a given geometry, the ductility demand on the energy dissipating elements is related to the global ductility demand of the deck-truss. Therefore, global stiffness of the structure must be determined so as to keep global ductility and displacement demands within reasonable limits. Stiffness of the ductile

devices has dominant effect on the overall stiffness, and this provides the control necessary for design.

Finally, it is recommended that the stiffness of the ductile panels be kept proportional to their respective capacity, as much as possible, to ensure that yielding in all ductile panels occurs nearly simultaneously. This should enhance energy dissipation capability and minimize the differences in the local ductility demands between the various yielding devices. It also helps prevent sudden changes in the proportion of the load shared between the two load paths, and minimize possible torsion along the bridge axis resulting from the instantaneous eccentricity that can develop when the end ductile panels yield first while the lower end ductile panels are still elastic.

4.1. General design methodology

Conceptually, any type of ductile energy dissipation system could be implemented in the end panels and lower end panels of the deck-truss, as long as its stiffness, ductility, and strength characteristics satisfy the requirements outlined is this appendix. The design methodology is iterative (initial properties must be assumed), and contains the following general steps.

1) Calculate fundamental period of vibration

The fundamental period for the transverse mode of vibration is given by:

$$T = 2\pi\sqrt{\frac{M}{K_{Global}}} \tag{20}$$

where M is the total mass of the deck, and K_{Global}, is given by:

$$K_{Global} = 2(K_{E,S} + K_{L,S}) \tag{21}$$

where $K_{E,S}$ is the stiffness of the ductile end cross-frames, taking into account the contribution to stiffness of the braces, verticals, horizontal, and ductile energy dissipation device/system, and $K_{L,S}$ is given by:

$$K_{L,S} = \frac{K^{*}\cdot K_{L,E}}{K^{*} + K_{L,E}} \tag{22}$$

where $K_{L,E}$ is the stiffness of the ductile last lower lateral panel, and

$$K^{*} = \frac{K_{C,B} + \sqrt{K_{C,B}^{2} + 4K_{C,B}\cdot K_{L,B}}}{2} \tag{23}$$

where $K_{L,B}$ represents the lateral stiffness of each panel of the lower lateral system (considering only the contribution of the braces to the panel stiffness) and $K_{C,B}$ represents the stiffness of the cross bracing panels (considering only the contribution of the braces to the panel stiffness).

The above equations are valid for a truss having at least 6 panels along its length. Otherwise, other equations can be derived following the procedure described by Sarraf and Bruneau [1,2].

2) Determine design forces

Although use of the capacity spectrum or push-over analysis is recommended for the design of such bridges, design is also possible using the R-factor approach. In that case, from the elastic seismic base shear resistance, V_e, for one end of the bridge (half of equivalent static force), it is possible to calculate $V = V_e / R$, where V is the inelastic lateral load resistance of the entire ductile diaphragm panel at the target reduction factor, and R is the force reduction factor calculated as indicated in Eq. 1. Note that μ in that equation represents the ductility capacity of the ductile diaphragm as a whole, not the local ductility of the ductile device that may be implemented in that diaphragm.

3) Determine strength constraints for ductile diaphragms in end panels

The upper limit for the transverse shear capacity of each end cross-frame panel, $V_{E,S,}$ can be determined from the following:

$$1.5V_{E,S} \leq Min\left(\frac{P_{Cr}\cdot b}{h}, \frac{T_r\cdot b}{h}\right) \tag{24}$$

where, P_{cr}, is the critical buckling load of the end verticals including the effect of vertical gravity as well as vertical inertia force due to earthquake, T_r, is the tensile capacity of the tie down device at each support, h, and b are height and width of the end cross-frame panel, respectively, and 1.5 is an overstrength factor.

4) Determine strength constraints for ductile diaphragms in lower end panels

Analyses showed that the force distribution in the interior cross-frames along the span is non- linear and of a complex shape. The model used to develop the equations presented here gives a conservative value of the lower end panel capacity, $V_{L,E}$, i.e. it ensures that $V_{L,E}$ is reached before any damage develops in any of the interior cross-frame.

The lower end-panel capacity shall not exceed the maximum end-panel force attained when the first sway-frame force reaches its strength limit state, S_{cr} (corresponding to buckling of its braced members, fracture of a non-ductile connection, or other strength limit states), and defined by:

$$1.5V_{L,E} \leq \left(\frac{\sum_{i=1}^{m}(1-\xi)^{i-1} - m\cdot(1-\xi)^{m-1}S_{Cr}}{1-(1-\xi)^{m-1}}\right) \tag{25}$$

where m is the number of *interior* cross-frames from the support to mid-span, 1.5 is the overstrength factor, and where:

$$\xi = \left(\frac{K_{C,B}}{K_{C,B} + \dfrac{K^{*}\cdot K_{L,B}}{K^{*} + K_{L,B}}}\right) \tag{26}$$

Note that if the total number of interior cross-frames, k, in a deck-truss is an even number (i.e $m=(k+1)/2$, is not an integer), m can be conservatively taken as $k/2$.

Interior cross-frames shall be designed to resist the force R'_1, given by:

$$R'_1 = 1.5V\xi(1-(1-\xi)^{m-1}) \tag{27}$$

where V is the total seismic force at one end of the deck-truss superstructure.

5) Determine total superstructure capacity

Given the above limits, the maximum total capacity of the superstructure will be the sum of the capacity of each ductile diaphragm, but not exceeding the substructure capacity, i.e:

$$1.5V_{\max}\leq[Min(2(V_{L,E}+V_{E,S}),2V_{sub})] \tag{28}$$

where V_{Sub} is the largest shear that can be applied at the top of the abutment without damaging the substructure (connections, wind shoes, etc.), and 1.5 is the overstrength factor. The above equation can be easily modified for bridges having multiple simply-supported spans. Furthermore, a minimum strength, V_{min}, must also be provided to resist the winds expected during life of the structure. Therefore, the yield capacity of the overall deck-truss system, R_{total}, should satisfy the following:

$$V_{\min}\leq R_{total}\leq V_{\max} \tag{29}$$

6) Distributed total system capacity

The chosen total capacity of the system can then be divided proportionally between the lower end and end panels according to the following equations which ensure the same safety margin for both panels.

$$R_{L,E} = \frac{R_{total}}{V_{\max}}V_{L,E} \tag{30}$$

$$R_{E,S} = \frac{R_{total}}{V_{\max}}V_{E,S} \tag{31}$$

7) Define capacity-based pseudo-acceleration and period limits

A corresponding *Capacity-Based Pseudo Acceleration*, PSa_C, can be calculated as:

$$PSa_c = \frac{R_{total}}{M} \tag{32}$$

This value can be drawn on a capacity spectrum, or compared with the required design values. Structural period of vibration directly ties this strength to the ductility and displacement demands. For example, in the intermediate period range, the ductility demand of systems having a constant strength decreases as the period increases (i.e. as stiffness decreases), while their displacement response increases. Therefore, a range of admissible period values can be located along the capacity-based pseudo-acceleration line, based on the permissible values of global ductility and displacement of the system corresponding to a particular ductile system.

Design iterations are required until a compatible set of strength and period are found to provide acceptable ductility and displacement demands. In other words, for a desired structural system strength, a range of limiting periods can be defined by a lower bound to the period, $T_{\min}$, to limit system ductility demands, and an upper bound, $T_{\max}$, to limit displacement demands (note that in some instances, $T_{\min}$ may not exist). As a result of these two constraints:

$$T_{\min} \leq T \leq T_{\max} \tag{33}$$

Note that it may be more convenient to express these limits in terms of the global stiffness of the entire structural system, or of the end panel. Since:

$$K_{E,S} = \frac{K_{Global}}{\alpha} \text{ where } \alpha = 2\left(1 + \frac{R_{L,E}}{R_{E,S}}\right) \tag{34}$$

Then:

$$\frac{4\pi^2 M}{T_{\max}{}^2} \leq K_{Global} \leq \frac{4\pi^2 M}{T_{\min}{}^2} \tag{35}$$

or for the end panel stiffness:

$$\frac{4\pi^2 M}{\alpha T_{\max}{}^2} \leq K_{E,S} \leq \frac{4\pi^2 M}{\alpha T_{\min}{}^2} \tag{36}$$

This can be used to select proper values of stiffness for the end panel. To calculate the stiffness of the lower end ductile panel, $K_{L,E}$, stiffness of the lower load path system is first determined as:

$$K_{L,S} = (K_{Global} - 2K_{E,S})/2 \tag{37}$$

and $K_{L,E}$ is given by:

$$K_{L,E} = \frac{K^* \cdot K_{L,S}}{K_{L,S} - K^*} \tag{38}$$

8) Design of ductile diaphragm panels

As for slab-on-girder bridges, systems capable of stable passive seismic energy dissipation must be used as ductile-diaphragms in deck-truss bridges. EBF, SPS, and TADAS have been studied for the deck-truss bridges [1,2]. Concentrically braced frames are deemed not admissible for the same reasons presented previously.

For convenience, the flexibility (i.e. inverse of stiffness) of panels having ductile diaphragms is provided below for a few types of ductile systems.

The flexibility of an eccentrically braced end panel, $f_{E,S}$, is expressed by:

$$f_{E,S} = \frac{h^2}{2EIb}\left(\frac{(a+e)^2}{3} - \frac{(b^2 - 2a^2)}{6}\right) + \frac{(a^2+h^2)^{3/2}}{2EA_b a^2} + \frac{h^3}{2EA_{col}a^2} + \frac{(b-e)}{4EA_l} + \frac{eh^2}{2GA_S ab} \tag{39}$$

where $a = (b-e)/2$, b is the panel width, h is the height, A_{col} is the cross-sectional area of a vertical panel member, A_b is the cross-sectional area of a bracing members, A_l, A_S, and I are, respectively, the cross-sectional area, shear area, and moment of inertia of the link beam, and e is the link length.

The flexibility, $f_{E,S}$, of a ductile VSL panel can be expressed by the following equation:

$$f_{E,S} = \frac{b(s+d/2)^2}{12EI} + \frac{2(h-s-d/2)^2+b^2/4)^{3/2}}{EA_b b^2} + \frac{2h(h-s-d/2)^2}{EA_{col} b^2} + \frac{b}{4EA_l} + \frac{s}{A_s G} \tag{40}$$

where, s is the height of the shear panel, I, is the bottom beam moment of inertia, and, d, is the depth of the bottom beam. The other parameters are as previously defined.

The required flexibility of the triangular plates alone for a TADAS system, f_T, expressed in terms of an admissible flexibility value of the end panel and other panel member properties, is given by:

$$f_T = f_{E,S} - \left(\frac{b(\eta\cdot h+d/2)^2}{12EI} + \frac{2(((1-\eta)h-d/2)^2+(b/2)^2)^{3/2}}{EA_b b^2} + \frac{2h((1-\eta)h-d/2)^2}{EA_{col} b^2} + \frac{b}{4EA_l}\right) \tag{41}$$

where η is the ratio of height of triangular plates to the height of the panel and other parameters correspond to the panel members similar to those of VSL panel. Tsai et al. [11]] recommended using η=0.10.

5. Conclusions

Step-by-step design procedures for the use of specially detailed ductile diaphragms have been presented for slab-on-girder bridges and deck-truss bridges. Although specific equations were presented for three types of structural systems (EBF, SPS (a.k.a. VSL) and TADAS), many other types of ductile diaphragms can be implemented provided that they possess a yield strength that can be accurately assessed, and can sustain repeated cycles of inelastic deformations in a ductile manner without significant strength degradation. While the procedures presented here were implemented as part of seismic design provisions for steel bridges as part of comprehensive proposed seismic design specifications, they have not yet been presented to AASHTO at the time of this writing.

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