Appendix F
DUCTILE END-DIAPHRAGM IN DECK TRUSS BRIDGE

F.1 DESIGN PROCEDURE

Similarly to the procedure described in Appendix E, 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. This could be the case, for example, when stiff wall-piers that can difficulty be detailed to have a stable ductile response are used as a substructure. The ductile diaphragms considered in this Article 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).

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 Figure F.1-1.

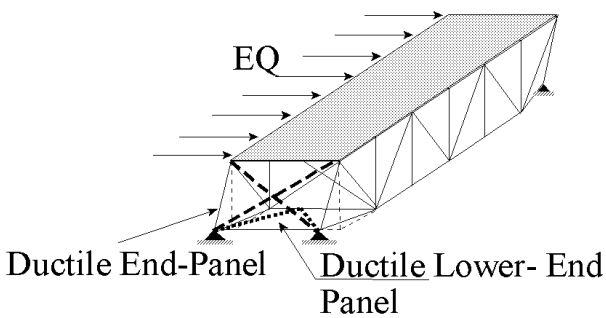

Figure F.1-1: Ductile diaphragm concept in deck trusses

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. 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 likes.

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 causes 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.

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:

$$T = 2\pi\sqrt{\frac{M}{K_{Global}}} \tag{F.1-1}$$

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

$$K_{Global} = 2\left(K_{E,S} + K_{L,S}\right) \tag{F.1-2}$$

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:

$$K_{L,S} = \frac{K^{*}K_{L,E}}{K^{*} + K_{L,E}} \tag{F.1-3}$$

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

$$K^{*} = \frac{K_{C,B} + \sqrt{K_{C,B}^{2} + 4K_{C,B}K_{L,B}}}{2} \qquad \text{(F.1-4)}$$

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 in Sarraf and Bruneau (1998a).

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 Article 7.7.8.3 or 8.7.8.3. 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:

$$1.5V_{E.S} \le Min\left(\frac{P_{Cr}b}{h}, \frac{T_r b}{h}\right) \qquad \text{(F.1-5)}$$

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 is 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:

$$1.5V_{L,E} \le \frac{\left(\sum_{i=1}^{m}(1-\xi)^{i-1} - m(1-\xi)^{m-1}\right)S_{Cr}}{1-(1-\xi)^{m-1}} \qquad \text{(F.1-6)}$$

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

$$\xi = \left(\frac{K_{C,B}}{K_{C,B} + \frac{K^{*} K_{L,B}}{K^{*} + K_{L,B}}} \right) \tag{F.1-7}$$

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 :

$$R_1' = 1.5V\xi\left(1 - \ 1 - \xi^{\ m-1}\right) \tag{F.1-8}$$

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:

$$1.5V_{\max} \le \left[Min\left(2\left(V_{L,E} \quad V_{E,S}\right), 2V_{sub}\right. \right] \tag{F.1-9}$$

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:

$$V_{\min} \le R_{total} \le V_{\max} \tag{F.1-10}$$

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.

$$R_{L,E} = \frac{R_{total}}{V_{\max}} V_{L,E} \tag{F.1-11}$$

$$R_{E,S} = \frac{R_{total}}{V_{\max}} V_{E,S} \tag{F.1-12}$$

7. Define Capacity-Based Pseudo-Acceleration and Period Limits

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

$$PSa_c = \frac{R_{total}}{M} \tag{F.1-13}$$

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:

$$T_{\min} \le T \le T_{\max} \tag{F.1-14}$$

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:

$$K_{E,S} = \frac{K_{Global}}{\alpha} \quad \text{where } \alpha = 2\left(1 + \frac{R_{L,E}}{R_{E,S}}\right) \tag{F.1-15}$$

Then:

$$\frac{4\pi^2 M}{T_{\max}^2} \le K_{Global} \le \frac{4\pi^2 M}{T_{\min}^2} \tag{F.1-16}$$

or for the end panel stiffness:

$$\frac{4\pi^2 M}{T_{\max}^2} \le K_{E,S} \le \frac{4\pi^2 M}{T_{\min}^2} \tag{F.1-17}$$

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:

$$K_{L,S} = \frac{\left(K_{Global} - 2K_{E,S}\right)}{2} \tag{F.1-18}$$

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

$$K_{L,E} = \frac{K^* K_{L,S}}{K_{L,S} - K^*} \tag{F.1-19}$$

8. Design of Ductile Diaphragm Panels

As indicated in Appendix E, many types of systems capable of stable passive seismic energy dissipation could be used as ductile-diaphragms in deck-truss bridges. Among those, eccentrically braced frames (EBF) (e.g. Malley and Popov 1983; Kasai and Popov 1986), shear panel systems (SPS) (Fehling et al. 1992; Nakashima 1995), and steel triangular-plate added damping and stiffness devices (TADAS) (Tsai et al. 1993), popular in building applications, have been studied for bridge applications (Sarraf and Bruneau 1998a, 1998b). Although concentrically braced frames can also be ductile, they are not admissible in Article 7.7.8.3 or 8.7.8.3 because they can often be stronger than calculated, and their hysteretic curves can exhibit pinching and some strength degradation.

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:

$$f_{E,S} = \frac{h^2}{2EIb}\left(\frac{a+e)^2}{3} - \frac{b^2 - 2a^2)}{6}\right) + \frac{a^2 + h^2}{2EA_b a^2}^{3/2} + \frac{h^3}{2EA_{col}a^2} + \frac{b-e)}{4EA_l} + \frac{eh^2}{2GA_s ab} \qquad \text{(F.1-20)}$$

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:

$$f_{E,S} = \frac{b(s+d/2)^2}{12EI} + \frac{2\left((h-s-d/2)^2 + b^2/4\right)^{3/2}}{EA_b b^2} + \frac{2h(h-s-d/2)^2}{EA_{col}b^2} + \frac{b}{4EA_l} + \frac{s}{A_s G} \qquad \text{(F.1-21)}$$

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:

$$f_T = f_{E,S} - \left(\frac{b\ \ \eta h + d/2)^2}{12EI} + \frac{2\left(\left(1\ \ \eta\ \ h - d/2\right)^2 + (b/2)^2\right)^{3/2}}{EA_b b^2} + \frac{2h\ \ 1\ \ \eta\ \ h - d/2)^2}{EA_{col}b^2} + \frac{b}{4EA_l}\right) \qquad \text{(F.1-22)}$$

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. (1993) recommended using η=0.10.