Seismic Behavior of Steel Girder Bridge Superstructures

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Abstract: Recent earthquakes exposed the vulnerabilities of steel plate girder bridges when subjected to ground shaking. This paper discusses the behavior of steel plate girder bridges during recent earthquakes such as Petrolia, Northridge, and Kobe. The paper also discusses the recent experimental and analytical investigations that were conducted on steel plate girder bridges and their components. Results of these investigations showed the importance of shear connectors in distributing and transferring the lateral forces to the end and intermediate cross frames. Also, these investigations showed the potential of using end cross frames as ductile elements that can be used to dissipate the earthquake input energy. The paper also gives an update on specifications and guidelines for the seismic design of steel plate girder bridges in the United States.

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Introduction

Steel bridges are generally considered to perform well in earthquakes, and the implication is often made that they should be used more frequently in seismically active regions. It seems that this argument is based on the fact that few, if any, steel bridges have collapsed in North American earthquakes, in contrast to the performance of structural concrete bridges.

If a steel bridge is defined as one with a steel superstructure and a steel substructure, there are very few of these in western North America, and even fewer have been subjected to strong ground motions in the last decade or so. However, if a steel bridge includes those with concrete substructures (piers and columns) the population increases significantly, but is still far less than that of structural concrete bridges (in western North America). Even so, performance data for these bridges is hard to find, and especially for bridges subjected to strong shaking.

Nevertheless, it can be inferred that steel bridge superstructures are susceptible to damage even during low-to-moderate shaking, and appear to be more fragile than structural concrete superstructures in this regard if not designed properly. Typical damage includes unseated girders and failures in connections, bearings, cross-frames, and expansion joints. In a few cases (notably during the Kobe earthquake) major gravity load-carrying members have failed, triggered in some instances by the failure of components elsewhere in the superstructure (a bearing, for example).

It may therefore be argued that the reputation enjoyed by steel bridges is due to the fact that very few steel bridges have been subjected to strong ground motion, and the absence of collapse may be due to a lack of exposure rather than the inherent capacity of steel bridges. Supporting this view is the observation that damage during low-to-moderate shaking shows a degree of fragility in steel bridges not seen in structural concrete superstructures.

It is important to note in this argument that seismic design specifications for bridges in the United States do not require the explicit design of bridge superstructures (concrete or steel) for earthquake loads. The assumption is made that a superstructure that is designed for out-of-plane gravity loads has sufficient strength, by default, to resist in-plane earthquake loads. This assumption appears to be justified for structural concrete superstructures, which are heavier and stiffer than their steel counterparts, but may be unfounded for certain types of steel superstructures, such as trusses or slab-and-girder superstructures, both of which may be flexible in-plane.

Improvement in the seismic performance of steel bridges is warranted, along with design guidelines for both steel sub- and super-structures. Better insight is required regarding the load path as well as the capacities of individual components and assembled systems. Applications of innovative technologies, such as ductile end cross frames (or diaphragms) and other embedded energy dissipators, deserve further study.

Performance of Steel Bridges during Recent Earthquakes

Steel plate girder bridges have generally suffered minor/moderate damage in past earthquakes compared to the significant damage suffered by structural concrete bridges. However, these earthquakes have identified critical components in the superstructure and substructure, which should be designed and detailed to resist seismic demand.

Behavior of Steel Bridges during the Petrolia Earthquakes

In 1992, three earthquakes of magnitudes 7.0, 6.0, and 6.5, respectively, occurred in a 24 h period near the town of Petrolia in

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Northern California (California Department of Transportation 1992). These earthquakes caused some notable damage to two steel plate girder bridges, the first being the Southbound Van Duzen River Bridge. In this straight steel plate girder bridge, buckling was observed at the end cross frames and the horizontal bracing. In addition, there was also spalling of concrete at the connection of the reinforced concrete deck and top flange of the steel girders at the end of one span indicating insufficient shear connectors in this region. This failure caused the bridge deck to slide over the plate girders at that location.

The second steel plate girder bridge damaged during the Petrolia earthquakes was the South Fork Eel River Bridge, a curved steel girder bridge located 49 km from the epicenters, 16 km further than any other highway structure with reported damage. It suffered considerable damage including buckling and fracture of end cross frames and their connections and also damage at the hinge locations. The damage had a large impact on the service load capacity of the bridge causing large observed deformations during the passage of trucks.

This earthquake highlighted the significance of shear connectors in transferring the lateral forces that are generated by the mass of the superstructure. These connectors should have sufficient strength to transfer the lateral force to the steel girders. In addition, it showed that the abutment and bent cross frames play an important role in transferring the lateral forces to the bearings. It also showed the potential of using these cross frames to yield and buckle in a controlled manner to dissipate the earthquake input energy.

Behavior of Steel Bridges during the Northridge Earthquake

During the 1994 Northridge earthquake several steel plate girder bridges suffered structural damage (Astaneh-Asl et al. 1994). Most of these bridges are located along Interstate 5 near the center of Newhall in Southern California. This region is located where the rupture of the hidden thrust fault would have projected to the surface. The nearest record at Newhall registered peak ground accelerations of $0.63g$ and $0.62g$, respectively, in the horizontal and vertical directions. Typical damage included anchorage failure of bearings on the abutments and bent caps, as shown in Fig. 1, causing damage to the substructure at these locations. Observed bearing damage coupled with relatively small seat widths, based on modern standards, caused the potential for unseating of the superstructure in some of these bridges. Typical damage in the superstructure included buckling of end cross frames or fracture of the connections between the end cross frames, gusset plates, and web stiffeners as shown in Fig. 2. In the case of the Pico-Lyons overcrossing there was no positive connection between web/bearing stiffeners and the bottom flange of the girders at the end cross frame locations. As a result, the web was damaged at the termination of the weld between the web and the stiffener as illustrated in Fig. 3. For these bridges there was minimal observed damage to the columns and piles indicating that much of the displacement demand was accommodated in the superstructure of each of these bridges.

Behavior of Steel Bridges during the Kobe Earthquake

The January 17, 1995 Hyogoken-Nanbu earthquake struck Kobe, a highly developed and congested modern city in a country well-known for its leading activities in earthquake engineering. Still, in spite of Japan's high level of earthquake awareness, extensive

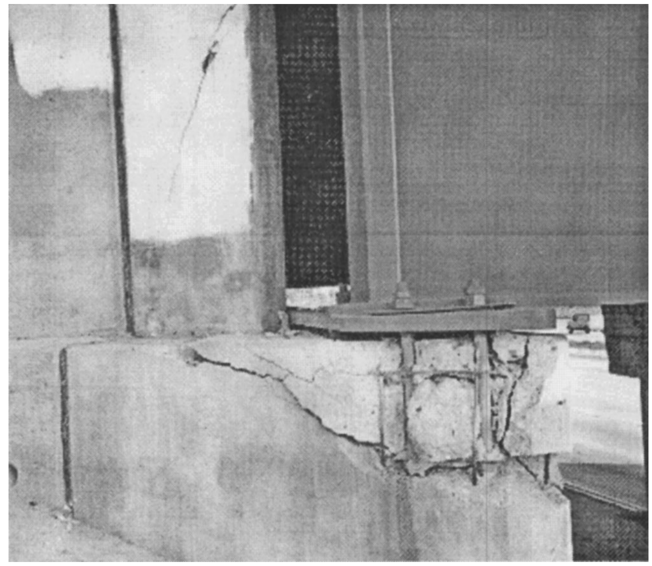

Fig. 1. Damage to bearing during the 1994 Northridge earthquake (Astaneh-Asl et al. 1994)

damage was suffered by numerous bridges in the area of severe shaking. As a result, all major roads and railways crossing Kobe were closed due to damaged or collapsed bridges.

The number of steel bridges in the area of severe shaking was considerably higher than for any previous earthquake. Damage was suffered by many steel piers, bearings, seismic restrainers, and superstructure components, and some spectacular collapses resulted from this damage (Ministry of Construction 1995; Bruneau et al. 1996). This damage is particularly relevant to Eastern North America where considerably more steel bridges exist than in Western North America where bridges exposed to past earthquakes were mostly of reinforced concrete. The damage suffered by short and medium span steel bridges can be categorized:

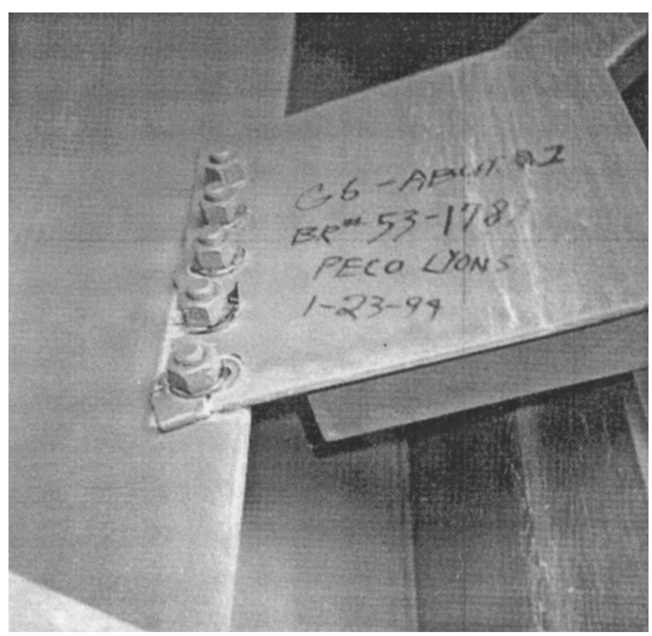

Fig. 2. Damage to end cross frames during the 1994 Northridge earthquake (Astaneh-Asl et al. 1994)

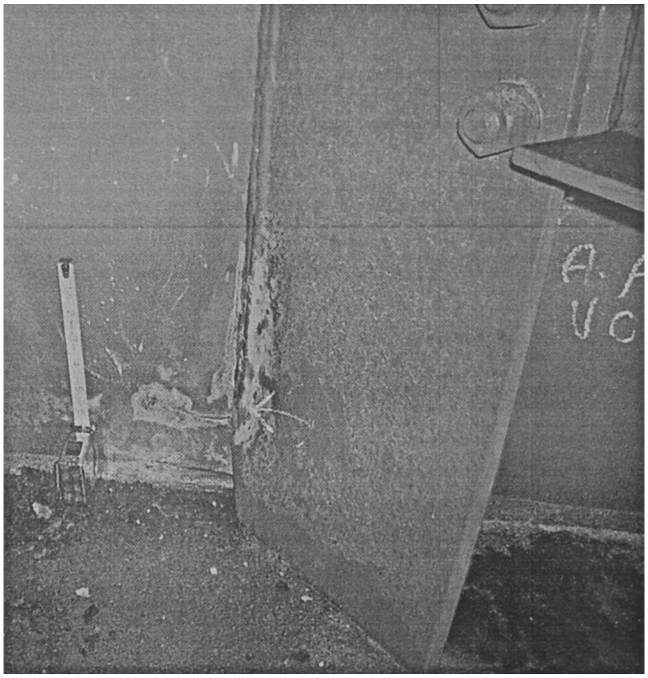

Fig. 3. Damage to web stiffeners during the 1994 Northridge earthquake (Astaneh-Asl et al. 1994)

- *Reinforced Concrete Substructure Failures*. Prior to the Hyogoken-Nanbu earthquake, many engineers alleged that steel bridges were immune from seismic damage by virtue of their lighter superstructure mass compared to concrete bridges, even if supported by nonductile substructure elements. This optimistic attitude was shattered as numerous concrete piers supporting steel superstructures failed all over Kobe during the 1995 earthquake. Failure modes germane to reinforced concrete piers and observed during this earthquake were similar to those observed in prior earthquakes (Priestley et al. 1996).
- *Steel Piers Failures*. A number of steel columns supporting portions of elevated expressways buckled, some rather severely, and collapse occurred at some locations as a result of steel column failures. In some locations the buckled plates fractured as a consequence of the large local inelastic cyclic strains. Brittle failures were also sporadically discovered in columns which otherwise showed no signs of local buckling.
- *Seismic Restrainers*. While many restrainers worked effectively by preventing spans from falling off their supports, numerous seismic restrainers showed signs of plastic yielding and/or buckling. Others were strained to their limit, often due to excessive substructure displacements, and failed.
- *Bearing Failures*. Bearings suffered a considerable amount of damage. They frequently were the second structural element to fail following major substructure damage. However, in some bridges which the superstructure remained intact, the bearings were the first to fail.
- *Bridge Girder Failures*. The lateral displacement observed for bridge spans which fell off their bearings was often impressively large, sometimes producing localized severe lateral-bending of the steel girders and even rupture of the end cross frames. Tensile fracture of the bolts connecting end cross frames to the main girders, and fracture through the cross frame extension haunch near the tip of the haunch, was typical in such cases (Fig. 4).

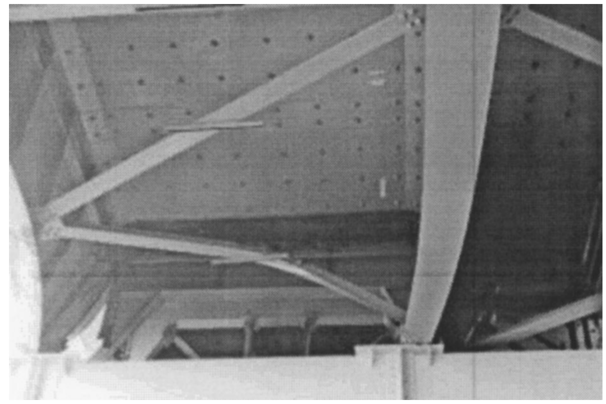

Fig. 4. Damage to end cross frames and girders during 1995 Kobe earthquake

Behavior of Steel Plate Girder Bridges under Lateral Loading

Steel plate girder superstructures consist of several components that lie in the lateral load path. These components are required to transmit the lateral forces to the supports. Any premature failure of these members may cause inadequate seismic response, and therefore, it is important to identify load path in steel plate girder bridges for earthquake response in both the transverse and longitudinal directions. Subsequently, critical components in the load path should be modeled and designed to achieve optimal performance of the system during an earthquake.

Lateral Load Path

Earthquake loading in the transverse direction causes transverse bending of the superstructure, resulting in transverse reactions at the abutments and bents. Consequently, the loads are distributed from the middle of each span to the supports. As the reinforced concrete deck and barriers in a steel plate girder bridge typically account for around 80% of the weight of the bridge, the majority of the inertia loads are generated in the superstructure. The bearing supports are at the bottom flange of the girders, therefore the inertia loads need to be distributed down through the superstructure components. Numerical analyses have shown that the loads are largely distributed through the deck to the ends of each span. The forces are then distributed vertically through bent and abutment cross frames (Itani and Rimal 1996; Zahrai and Bruneau 1998a,b). These forces are then transmitted to the bearings and shear keys at support locations. As the primary function of the bearings is to allow thermal movement, they are usually restrained from translation in the transverse direction. Thus the transverse shear forces in the bearings are transferred into the abutments and bents.

For longitudinal ground motion the inertia forces for a straight bridge are transferred from the deck into the girders using shear connectors along the length of the bridge. From the girders the loads are transferred into the bearings and substructure. Longitudinal deformation in the bearings is typically limited by the abutment once the expansion joint gap has closed and, for longer span bridges, by restraints at the bents which are activated at the design bearing deformation limits allowing forces to be transferred into the bents.

Modeling Superstructure Behavior under Lateral Loading

In the literature on the seismic evaluation and/or design of bridges (e.g., Buckle et al. 1986; Priestley et al. 1996), when the lateral period of a slab-on-girder bridge is determined, the superstructure (deck and girders) is modeled as an equivalent beam supported on columns and/or foundation springs. The effective transverse stiffness of this equivalent beam is calculated considering that the deck and girders act as a single cross section. While this approach is acceptable for concrete bridges and box-girder superstructures, it may not be for some types of existing slab-on-girder steel bridges. Typically, in such bridges, the concrete deck slab is supported on I-shaped beams interconnected by a few discrete cross frames, and the mechanism by which the seismically induced inertia forces at the concrete slab level will be transmitted to the girder bearings can be quite different from that assumed by the equivalent beam analogy. The magnitude of this difference is tied to the effectiveness of the cross frames, and can be quite large in bridges having flexible cross frames. Proper representation of the superstructure's lateral stiffness is important as it has a direct impact on the calculated bridge period, and consequently on the intensity of earthquake excitation applied to the superstructure, bearings, and substructure.

A first step in understanding the behavior of these bridges is to study the case without any effective cross frame. Such a model would be valid for bridges having severely rusted cross frames or with only nominal cross frames (e.g., single channels bolted along their web) as frequently encountered in Eastern North America. Likewise, bridges having cross frames with nonductile connection details can potentially become bridges without cross frames once brittle failures develop at those connections.

The lateral response behavior of such slab-on-girder steel bridges of various span lengths was investigated using the program *SAP90*. The calculated first lateral period of vibration as well as pseudo-spectral acceleration (PSa) are required to produce first yielding as a function of span length, presented elsewhere (Zahrai and Bruneau 1998a,b), along with comprehensive analytical expressions that capture that behavior. Although these response parameters vary nonlinearly as a function of span length in a complex manner, the general trend is that the resulting lateral periods and maximum lateral deflections are large compared to values typically reported for slab-on-girder bridges in the literature, reflecting the extreme flexibility of the structural system in the absence of cross frames. The concrete deck slab displaces laterally nearly as a rigid body, while the flexible steel girders twist and deform laterally as necessary, spanning between the slab and the supports. Closer examination of the steel beams reveals that they are most severely distorted near the supports; indeed, in each girder, the bearing supports are the only points which can counteract the lateral pull of the web to bring the lower flange under the slab.

The program *ADINA* was used to investigate the nonlinear behavior of these steel bridges and the impact of $P-\Delta$ effects (second order analysis) on this ultimate behavior. Results from push-over analyses indicate that, since lateral displacements are large in bridges without any cross frames, $P-\Delta$ effects due to the displaced weight of the deck are significant leading to inelastic overturning and structural instability.

To understand the behavior of these bridges, a bridge without end cross frames but with intermediate cross frames was investigated. Inelastic analyses showed that, in the absence of end cross frames, the presence of intermediate cross frames does not greatly improve the seismic behavior of slab-on-girder bridges. This is because the largest girder web distortions occur near the girder supports, and the contribution of intermediate cross frames in resisting the lateral load is consequently small.

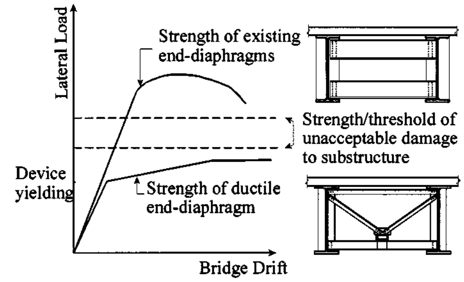

Fig. 5. Inelastic behavior of ductile end cross frames compared to existing strong diaphragms

The above analyses revealed the key role played by the end cross frames to ensure an adequate load-path in slab-on-girder steel bridges. For bridges with cross frames, analyses showed that a small end cross frame stiffness is sufficient to make the entire superstructure behave as a unit in the elastic range. However, a dramatic shift in seismic behavior could occur once rupture of the end cross frames occurs, with a sizeable period elongation, considerably larger lateral displacements, and higher propensity to damage due to instability and $P-\Delta$ effects.

Design of End Cross Frames

Given that effective end cross frames constitute critical structural elements along the main seismic load path, they should therefore be designed to resist in an elastic manner the forces induced by the maximum credible earthquake. Likewise, end cross frame members and connection details prone to fracture in existing steel bridges should be similarly retrofitted. Typical elastically designed cross frames include K-braces or X-braces located in the vertical plane transversely between the steel girders. However, as an alternative for both new designs or retrofits, the end cross frames could be designed and detailed as ductile members to preclude brittle member or connections failure and protect the substructure.

Behavior of Steel Bridges with Ductile End Cross Frames

By ensuring that the steel cross frames over abutments and piers are specially designed ductile cross frames calibrated to yield before the strength of the substructure is reached, damage can be prevented from developing in the nonductile substructural elements, foundation, and bearings (referred generically as ''substructure'' hereafter). This objective is schematically illustrated in Fig. 5. Many types of systems capable of stable passive seismic energy dissipation could be used for this purpose. 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) have received particular attention in building applications. Still, to the authors' knowledge, none of these applications has been considered for bridge structures prior to the research reported by Sarraf and Bruneau (1998a,b) and Zahrai and Bruneau (1999a,b). This may be partly attributable to the absence of seismic ductile steel detailing provisions in North American bridge codes. Zahrai and Bruneau (1999a) developed simplified analytical models as well as a step-by-step design procedure for these three types of ductile cross frame systems (SPS, EBF, and TADAS devices) in girder bridges, and validated the ductile cross frame concept using nonlinear inelastic analyses. The concepts have also been verified experimentally (Zahrai and Bruneau 1999b). Note that although concentrically braced frames can also be ductile, they are not considered here because these are often stronger than calculated, and their hysteretic curves can exhibit pinching and some strength degradation.

Note that research was also conducted to develop and experimentally validate the concept of ductile cross frames for the seismic retrofit of deck-truss bridges (Sarraf and Bruneau 1998a,b).

Effect of Composite Action on Lateral Load Transfer

In order to ensure activation of ductile end cross frames or even adequate load path for elastically designed end cross frames the loads must be transferred from the deck into the steel superstructure. For earthquake ground motion in the longitudinal direction, the inertia forces can be distributed from the deck into the steel girders through the shear connectors along the entire length of the bridge as the shear connectors run parallel to the direction of loading. However, in the transverse direction, the distribution of forces in the shear connectors varies along the length of the bridge. Numerical analysis has been performed on a typical four span, four girder steel plate girder bridge in order to investigate the effect of composite action in the transverse response of a bridge. The bridge was first modeled as fully composite along the entire length with shear connectors placed on the top flange of each girder in both positive and negative bending moment regions in accordance with the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (AASHTO 1998). Application of transverse earthquake loads showed that there were large transverse shear forces in the shear connectors within approximately 3 ft of the ends of each span while along the remaining length of each span the shear forces were negligible. This was consistent with observations from failure in an experiment on a single span bridge model (Carden et al. 2001), as shown in Fig. 6. Therefore it is apparent that the loads are transferred from the deck into the substructure at the immediate ends of the spans of this bridge, highlighting the importance of composite action in this region. Although for this bridge model, the finite element analyses showed that the maximum forces in the shear connectors were around 150% of their design strength at the ultimate limit state of the column bents, the concentration of forces may be damaging in other bridges. Further parameter studies are required to evaluate seismic demand on shear connectors in this region.

Many bridges have no shear connectors in the negative moment regions due to fatigue concerns when welding studs to the tension flange of a steel girder. A second numerical model was used to investigate the impact on the load path when there are no shear connectors in the negative moment region. In this model large forces were induced in the shear connectors at the points of contraflexure at the end of the composite regions. As additional shear connectors were placed at the points of contraflexure, in order to make the transition between a composite and noncomposite model, the forces in the shear connectors were below design levels. However, as the transverse inertia forces were distributed from the deck into the girders at the points of contraflexure, the girders were required to transfer the forces from the points of contraflexure to the bents. Weak axis bending moments were induced in each noncomposite girder resulting in stresses, which when combined with gravity load stresses, would have resulted in buckling or yielding in the girders before the appropriate limit states were reached in the ductile columns.

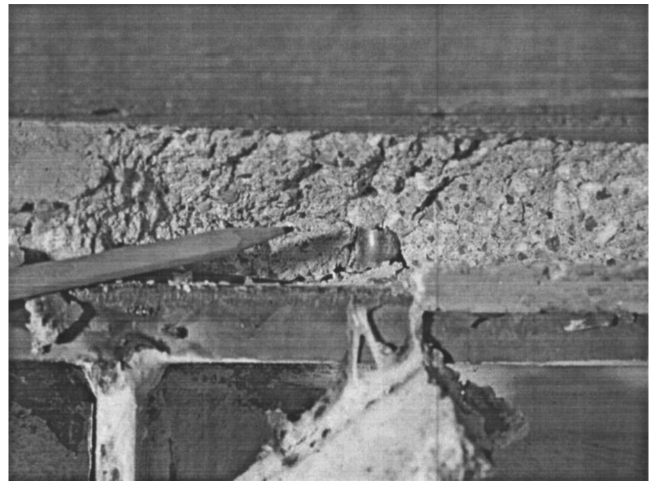

Fig. 6. Failure of shear connectors in bridge model during transverse cyclic loading

When there is no composite action between the deck and the girder in the negative moment regions then the intermediate cross frames between the ends of each span and the points of contraflexure are important in distributing some of the loads from the top flange of the girders down to the bottom flange of the girders. Therefore, in this situation, these intermediate cross frames should also be designed for a portion of the earthquake forces.

To ensure a favorable transverse load path it is recommended that adequate composite action be provided between the girders and the deck for transverse earthquake loading. Designing the top chord of the end cross frames to be composite with the deck was found to be effective in transferring the earthquake loads directly from the deck into the cross frames. This connection should be designed to carry the full earthquake shear at the abutments or column bents. However, if the top chord of the cross frames is made composite in negative moment regions while the girders are noncomposite with the deck, this chord is likely to be subjected to stresses in the longitudinal direction due to service loading on the bridge. These stresses should be accounted for in the design of the composite connection. Consequently, it is recommended that in high seismic zones the girders be made fully composite in positive and negative moment regions to provide adequate composite action.

New Seismic Design Specifications for Steel Bridges

The AASHTO LRFD specifications have limited provisions for the seismic design of steel bridges. In fact, the provisions do not offer information about analysis and the design of steel plate girder bridges. In order to overcome this shortcoming the Structural Steel Committee of the California Department of Transportation (Caltrans) initiated a study to establish guidelines for the seismic design of a steel bridge in high seismic zones. To base ‘these guidelines on a rational basis Caltrans supported an experimental testing program of a one span bridge steel plate girder bridge. This one span steel bridge was tested under lateral loading to determine the load path and establish design guidelines for the design of cross frame members (Carden et al. 2001). In addition, current experimental investigation is conducted to determine the behavior of steel plate bridges with integral bent cap (Patty et al. 2001). It is expected that Caltrans will publish their design guidelines in 2002.

In a separate effort, from 1998 to 2001, an AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) project was conducted to develop a comprehensive specification for the seismic design of bridges that would include the latest knowledge about the seismic performance of bridges, MCEER 2002. 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 subtask of this project consisted in 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 subtask, 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. Interestingly, during the course of this development, it was observed that the proposed ductile cross frame concept generated considerable interest for application in new bridges, as a cost-effective potential solution to achieve the ductile response of these types of bridges. To address this new interest, adjustments were required to revise the proposed retrofit methodologies developed earlier, and convert them into design procedures. This was done in compliance with the proposed specifications' intent to permit the use of innovative systems (such as ductile cross frames) by defining a category of ‘‘special systems’’ that can be used for steel bridges.

For these 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 stated:

Ductile end cross frames 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:

- Specially detailed cross frames capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;
- Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;
- Designers should consider the combined and relative stiffness and strength of end cross frames and girders (together with their bearing stiffeners) in establishing the cross frames strength and design forces to design for the capacity-protected elements.

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References

American Association of State Highway and Transportation Officials (AASHTO). (1998). *AASHTO LRFD bridge design specifications*, 2nd Ed., Washington, D.C.

Astaneh-Asl, A., Bolt, B., McMullin, K. M., Donikian, R. R., Modjtahedi, D., and Cho, S. (1994). ‘‘Seismic performance of steel bridges during the 1994 Northridge Earthquake.’’ *Rep. No. UCB/CE-STEEL-94/01*, Dept. of Civil Engineering, Univ. of California at Berkeley, Calif.

Bruneau, M., Wilson, J. W., and Tremblay, R. (1996). ‘‘Performance of steel bridges during the 1995 Hyogoken-Nambu (Kobe, Japan) earthquake.’’ *Can. J. Civ. Eng.*, 23(3), 678–713.

Buckle, I. G., Mayes, R. L., and Button, M. R. (1986). ‘‘Seismic design and retrofit manual for highway bridge,’’ *Rep. No. FHWA-IP-87-6*, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.

California Department of Transportation. (1992). *PEQIT Report—Highway Bridge Damage-Petrolia Earthquakes No. 1, No. 2, No. 3. of April 25–26, 1992*, California Department of Transportation, Sacramento, Calif.

Carden, L. P., Garcia-Alvarez, F., Itani, A. M., and Buckle, I. B. (2001). ‘‘Cyclic response of steel plate girder bridges in the transverse direction.’’ *Proc. 2001 Caltrans Seismic Conf.* (CD Rom), Sacramento, Calif.

Fehling, E., Pauli, W., and Bouwkamp, J. G. (1992). ‘‘Use of vertical shear links in eccentric braced frames.’’ *Proc. 10th World Conf. On Earthquake Engineering, Vol. 9, Madrid*, 4475–4479.

Itani, A. M., and Rimal, P. P. (1996). ‘‘Seismic analysis and design of modern steel highway connectors.’’ *Earthquake Spectra*, 12(2), 275–296.

Kasai, K., and Popov, E. P. (1986). ‘‘Cyclic web buckling control for shear link beams.’’ *J. Struct. Eng.*, 112(3), 505–523.

Malley, J. O., and Popov, E. P. (1983). ‘‘Design considerations for shear links in eccentric braced frames.’’ *EERC Rep. 83-24*, University of California, Berkeley, Calif.

Ministry of Construction (1995). *Investigation of damage to highway bridges from the Hyogoken-Nanbu Earthquake—Preliminary report (in Japanese)*, Rep. Ministry of Construction—Hyogoken-Nanbu Earthquake Committee on Highway Bridge Damage Prevention, Tokyo, Japan.

Nakashima, M. (1995). ‘‘Strain hardening behavior of shear panels made of low-yield steel. I: Test.’’ *J. Struct. Eng.*, 121(12), 1742–1749.

Patty, J., Seible, F., and Uang, C. (2001). ‘‘Longitudinal seismic response of concrete substructure steel superstructure intergral bridge connections.’’ *Proc. 2001 Caltrans Seismic Conf.* (CD Rom), Sacramento, Calif.

Priestley, M. J. N., Seible, F., and Calvi, G. M. (1996). *Seismic design and retrofit of highway bridges*, Wiley, New York.

Sarraf, M., and Bruneau, M. (1998a). ‘‘Ductile seismic retrofit of steel deck-truss bridges. I: Strategy and modeling.’’ *J. Struct. Eng.*, 124(11), 1253–1262.

Sarraf, M., and Bruneau, M. (1998b). ‘‘Ductile seismic retrofit of steel deck-truss bridges. II: Design applications.’’ *J. Struct. Eng.*, 124(11), 1263–1271.

Tsai, K. C., Chen, H. W., Hong, C. P., and Su, Y. F. (1993). ‘‘Design of steel triangular plate energy absorbers for seismic-resistance construction.’’ *Earthquake Spectra*, 9(3), 505–528.

Zahrai, S. M., and Bruneau, M. (1998a). ''Impact of diaphragms on seismic response of straight slab-on-girder steel bridges.'' *J. Struct. Eng.*, 124(8), 938–947.

Zahrai, S. M., and Bruneau, M. (1998b). ''Seismic retrofit of slab-on-girder steel bridges using ductile end diaphragms.'' *Rep. No. OCEERC 98-20*, Univ. of Ottawa, Ottawa, Ontario.

Zahrai, S. M., and Bruneau, M. (1999a). ''Ductile end-diaphragms for seismic retrofit of slab-on-girder steel bridges.'' *J. Struct. Eng.*, 125(1), 71–80.

Zahrai, S. M., and Bruneau, M. (1999b). ''Cyclic testing of ductile end-diaphragms for slab-on-girder steel bridges.'' *J. Struct. Eng.*, 125(9), 987–996.