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Overview - of Seismic Resistance of Railway Steel Trusses Bridges Using Splice Connection

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Abstract:

This paper presents overview of a seismic assessment of multi-span steel railway bridges and preventive seismic performance of steel structures. The main concept is splice connection use to steel members in railway bridge seismic behaviour and safety under seismic conditions. The newly developed splice connection in main girder , longitudinal girder trusses members used in railway bridges under reversal cyclic loading to evaluate seismic performance. Seismic performance is evaluated based on hysteretic behaviour, strength, ductility, stiffness, and energy dissipation.

key words : steel connection, splice connection , railway bridges, steel joint seismic

I Introduction

According to the recent Indian standard code on earthquake resistant design of structures, more than 60-65% of the area of our country falls under seismic zone III or above. This underlines the importance of seismic detailing. In any structure, the joints assume more importance and have to be detailed carefully so that they are able to withstand the inelastic joint rotations (in the order of 0.04 radians) and drift that may result during an earthquake. The detailing of reinforced concrete structures have been covered adequately in the Indian codes. However, until recently such detailing of joints in steel structures was not covered in the Indian code on steel structures. Though the recent version of the code, IS 800:2007, contains provisions for design and detailing for seismic loads, it does not suggest the type of connections which are suitable for high or intermediate seismic zones. **A Jayaraman et al., (2018)** in this research work, the design based on IS 800:2007, IS 801:1975 and IS811:1975 the study is carried out to earthquake code book IS1893 (Part1):2002 and analysis is done by a industrial structure in both conventional steel and cold formed steel using splice connections. The parameters mainly focused in this study is the deflection and load carrying capacity of industrial structures, member reduction, cost of steel structures, seismic performance in both conventional and light gauge cold formed steel with or without splice connections. **Kaar et al. (1960)** carried out the load tests on the connection detail where the deformed rebar in the deck slab is made continuous over the supports and resists the negative bending moment. This detail also included the use of a diaphragm over the piers extending laterally between the T- girders on the side. The width of the diaphragms was greater than the spacing between the ends of the T-girders, which helped to provide lateral restraint to the compressive strength of concrete. The results from this study found that this continue connection detail was desirable as it permits sufficient redistribution of moment and an easy to construct and relatively economical. **Mattock et al., (1960)** carried out additional tests on the continue connection for precast, prestressed concrete bridge T- girders with introduction of all details for resisting the (+) moments resulting from shrinkage and creep of concrete. They conducted static and dynamic load tests on 1/2-scale component specimens of a double-span continue connection between the Tgirders with CIP deck and diaphragm. The results from all the static load tests to confirmed the results determined by **Kaar et al., (1960).** From the dynamic test using repeated moving loads applied to the free ends of the girders, the researchers found that the connection can potentially resist a definite number of all applications of design loads without failure. However, the breath of the cracks and the resulting flexible of the connection were found to large. They tested two connection details of all (+) moment resistance: (i) Fillet arc welding the projecting of all ends to the reinforcement bars toa structural steel angle, and (ii) bending the projecting ends of the reinforcement to form right angle hooks and lapping them with the longitudinal diaphragm reinforcement. This project deals with the RCC design of an earthquake resistant bridge. The location is near railway station in THANJAVUR which is facing major traffic problem due to the train moving & the Public felt inconvenient to cross the busy Track. We have done a traffic survey and designed all the structural parts for the project. The bridge is having 10m span Length .The slab is designed by Limit stress method as per the recommendation of IRC:-21-2000 and IS 456-2000.Dead & live load for the Pire Column, slab, beam. All the elements are designed by using M25 Concrete Grade, & fe 415 I. The every major earthquake, to increase the capacity demand of the structure to counteract. The last decade that new Planning have been developed to manage this problem economically. The current IS Code Practice has towards a performance- based engineering design, wherein it's on serviceability & safety under different level of magnitude of earthquakes. There is an increasing realization that apart from techniques for improving ductility. There have been methods of preventing dislodgement of superstructure at the severe earthquake. This is used in economical earthquake resistant design of bridge superstructure. Our project deals with the RCC design of an earthquake resistant bridge. The location is near railway station in Thanjaur which is facing major traffic problem due to the train moving & the Public felt inconvenient to cross the busy Track. We have done a traffic survey and designed all the structural parts for the project. Grade steel. All the Drawings are draft by using AutoCAD 2016 and analysis by STAAD- PRO and Manual Design calculation for RCC structural design. **Chin-Tung Cheng et al.,(2003)** To develop an effective repair technique for rapid bridge restoration after an earthquake, four hollow bridge columns were cyclically loaded to failure, repaired and retested. The repair process includes using dog-bone shape bars to replace the fractured longitudinal bars in plastic hinges and using FRP (Fiber Reinforced Plastic) wraps to enhance the deformation capacity of columns. The repair aims to restore seismic capacity in terms of strength and ductility. Test results indicate that the fractured longitudinal bars can be completely repaired and the deformation capacities of the columns were enhanced by FRP wraps. However, due to concrete deterioration and the buckling of the longitudinal bars in the inner layer of the hollow sections, the test results also indicate the repaired column strengths are less than anticipated. Test results of four full-size hollow bridge columns show that the hollow columns reinforced with two layers of longitudinal rebar can achieve the desired strength and with excellent ductility, provided sufficient hoops are used. Test results also show that the flexural damage caused by the fracture of outer layer longitudinal bars were successfully repaired by dog-bone shape bars. While buckled rebars in the inner layer, which were left without repair due to construction feasibility, resulted in lower restored column strength. Test results indicate that FRP wraps not only increase the ultimate displacements for the columns, but also reduce column shear deformation. In addition, flexural-shear failure mode in the circular column was upgraded to flexural dominance. Among the models for columns without FRP, the Priestley shear model is the best fit with experiments in terms of strength and prediction of failure modes. For repaired columns, Seible's evaluation overestimated the shear capacity due to the poor efficiency of concrete repair in the column wall. The energy model proposed by Dutta and Mander made a reasonable prediction of strength deterioration for the lap-splice column. **Dongzhi guan et al., (2016)** The newly developed beam-to-column connection combining both longitudinal bar anchoring and lap splicing used in precast construction has been investigated under reversal cyclic loading to evaluate seismic performance. A new precast concrete beam-to-column connection for moment-resisting frames was developed in this study. Both longitudinal bar anchoring and lap splicing were used to achieve beam reinforcement continuity. Three full-scale beam-to-column connections, including a reference monolithic specimen, were investigated under reversal cyclic loading. The difference between the two precast specimens was the consideration of additional lap-splicing bars in the calculation of moment-resisting strength. Seismic performance was evaluated based on hysteretic behavior, strength, ductility, stiffness, and energy dissipation. The plastic hinge length of the specimens is also discussed. The results show that the proposed precast system performs satisfactorily under reversal cyclic loading compared with the monolithic specimen, and the additional lap-splicing bars can be included in the strength calculation using the plane cross-section assumption. Furthermore, the plastic hinge length of the proposed precast beam-to-column connection can be estimated using the models for monolithic specimens. **Mehmet F et al., (2018)** this study presents a seismic assessment of multi-span steel railway bridges in the Turkish railway system. The main concept was to determine bridge seismic behavior and safety under seismic conditions. Bridge PSDMs were obtained for the example the Alasehir bridge from 60 nonlinear time history analyses, and bridge component demands were used to derive component and overall bridge fragility curves. Several IMs were considered to characterize the seismic event, and their relative practicality, efficiency, and proficiency was compared. Calculated fundamental period of Alasehir bride was obtained 0.38 and 0.59 s at x and y direction, respectively. **Alessandro Palermo et al., (2012)** the paper provides a brief overview of international trends of Seismic Accelerated Bridge Construction (ABC). The United States through FHWA, Caltrans and AASHTO seems to strongly believe in the benefits of ABC and therefore a massive collaborative programme has been on-going for 6-8 years. The researchers in the United States are currently working on new ways of improving the seismic performance of ABC bridges looking at both emulative cast in place and low damage controlled rocking solutions. New Zealand has always been a world leader for pioneering design concepts in earthquake engineering. **Ahmad et al.,** (2004) he reported that 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. Recent earthquakes exposed the vulnerabilities of steel plate girder bridges when subjected to ground shaking. This paper discusses the behaviour 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. **Jay shen et al .,(2010)** he reported that until additional research considering combined cyclic flexural and tensile actions reduces the uncertainty inherent in reliably estimating the capacity of a welded column splice constructed using partial-joint penetration groove welds, it is recommended that a significant margin of safety be provided for column splices in seismic load-resisting structures. For special moment frames in moderately tall structures (e.g., those taller than approximately nine stories, where the effects of tensile axial loads on column splices may be significant), current requirements mandating use of complete-joint-penetration groove welds in welded column splices appears reasonable. For special moment frames in shorter structures (e.g., those less than or equal to approximately nine stories), current requirements mandating use of complete joint-penetration groove welds in welded column splices appear conservative. Welded splices using partial-joint-penetration groove welds (or the equivalent bolted splice) designed to develop at least 0.8Mp of the smaller column appear to provide a reasonable margin of safety and could be permitted. Based on the seismic demands on the column splices determined from this study, it is recommended that an experimental and analytical study be undertaken to investigate the performance of column splices using partial-joint-penetration groove welds (or equivalent bolted splices) under combined cyclic flexural and tensile axial force demands. This additional research could also investigate the reliability of the proposed height limitations proposed. **Abolhassan astaneh-Asl et al., (1998)** The paper also presents the concept of performance-based design of steel connections using a failure mode hierarchy. In this concept, all failure modes of the connection are identified and then an order of desirability is assigned to each failure mode based on its ductility. The more ductile the failure mode is the higher its place in the hierarchy. Then, for each failure mode, design equations are developed. These equations ensure that, the more ductile failure modes, such as yielding of steel, will occur first and protect the connection from experiencing the more brittle and undesirable failure modes, such as fracture of welds, bolts or net sections. As an illustration of the procedure, this paper presents application of a proposed "hierarchical" approach to failure modes of bolted top-and bottom flange plate moment connections and provides corresponding design equations. In general, designing a structure that minimizes a total cost, including not only construction but also repairing costs after severe earthquake, requires considerable number of calculations in a trial-and-error basis. In this research, a new design nomograph that gives combinations of demanding natural period, ductility and yielding coefficient in view of minimizing a total cost of structure is proposed by means of nonlinear numerical simulations. It is clarified through comparison of the nomograph and current design spectra based on past earthquake records that the structures comply with current design code meet the demands of preventing severe damage and minimizing the total cost simultaneously. The seismic vulnerability of regular bridges with continuous deck, monolithically connected to the piers or supported on elastomeric bearings is studied, as a function of the bridge length, the number, section and height of piers and the number of columns per pier. At the abutments the deck is free to translate longitudinally; transversely it is either free or constrained. Prototype bridges are designed according to Eurocodes 2 and 8 (EC2 and EC8) and their fragility curves constructed as a function of peak ground acceleration. Their seismic deformation demands are estimated by 5%-damped linear analysis and their shear forces from the plastic mechanism. Bridges designed to EC8 have satisfactory fragilities; those designed to EC2 alone show remarkably good seismic performance, yet quite higher fragilities than the bridges designed to EC8. Horizontal displacements are critical for long bridges or bridges on bearings. Hollow piers are vulnerable in shear.

II PREVENTION OF DAMAGE OF STEEL STRUCTURES

The following techniques and devices commonly used to create a low-damage building using structural steel.

2.1 Spring joints

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Shown in Figure .1 spring joints can be used in rocking structures. They use a series of high-strength springs to couple the base of the steel frame to the foundation. During an earthquake, the frame is still able to rock according to its post-tensioned design. However, as soon as the column foot begins to lift, the spring compresses and applies a restorative and damping force. The spring also remains undamaged. Like other dampers, spring joint systems protect the foundation and column foot from impact damage while the building rocks. They also act with the post-tensioning system to return the structure to its original upright position. The springs also increase the force required to produce uplift, which increases the overall stiffness of the structure and reduces frame displacements.

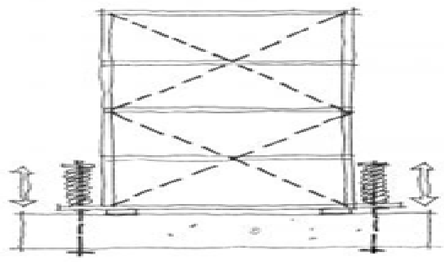

Figure .1 spring joints

2.2 Sliding Hinge joints

A sliding hinge joint is a special type of friction damper. It is a beam-column connection used in steel moment frames that adds friction damping to the structure. It consists of a top and bottom flange plate welded to the column and a friction plate. The top and bottom flange plates hold the beam in place, and the top flange plate can rotate around its connection to the column. The friction plate is sandwiched between the beam and the bottom flange plate. During an earthquake, the joint allows the beam to rotate around the top flange plate. As the frame moves, the bottom of the beam and bottom flange plate slide against the friction plate. This dampens the movement and reduces the likelihood of yield in the main structural elements. However, the technique is not entirely self-centring due to the additional friction in the system. The connection of system shown in Figure .2

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Figure .2 Sliding Hinge joints

2.3 Friction damped braces

Also a damping brace, a friction brace combines a concentric or eccentric brace with a friction damper to add energy dissipation to the system. Concentric friction braces may use a friction damper that is in line with the diagonal brace (top, bottom or midway). They may also be part of a gusset plate attached to the beam or slide directly on the bottom flange of the beam. Eccentric friction braces may use a friction damper that is part of a gusset plate attachment, a sacrificial link below the beam or in-line devices that form an asymmetrical pair. Friction braces provide a similar level of damping under both tension and compression loads, but they require careful design to ensure they do not buckle out of plane under compression. Also, friction braces have no self-centring action and must be combined with other systems if this characteristic is required in the building design. The friction damped braces system shown in Figure .3

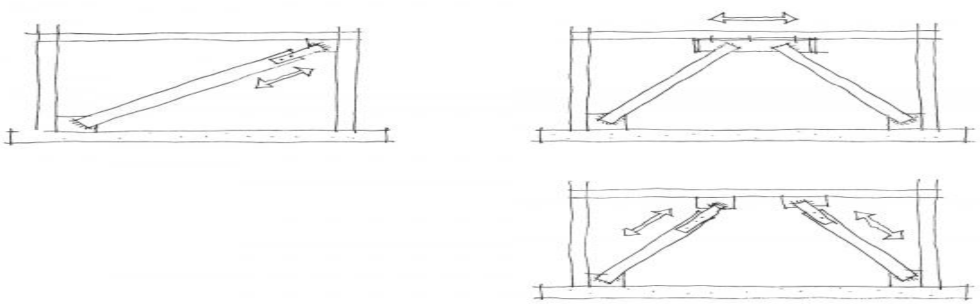

Figure .3 Varieties of friction braces.

2.4 Friction dampers

Friction dampers are a class of device that use metal surfaces in friction to dissipate seismic energy by converting it into heat as they rub together. Typically, a friction damper consists of several steel plates sliding against each other in opposite directions. The steel plates are separated by layers of friction pad material, like the brakes in a car. Friction dampers are not used by themselves but become part of other structural systems, such as diagonal bracing, to add friction damping to the structure. Friction dampers can take several forms, but there are two basic varieties – symmetrical and asymmetrical depending on the type of bracing that they are combined with steel plates. The symmetrical friction damper system shown in Figure .4

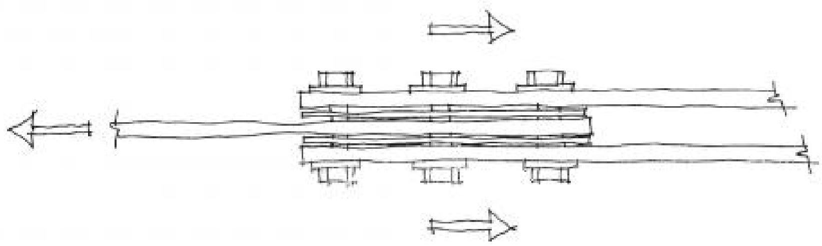

Figure .4 operation of a symmetrical friction damper.

2.5 Steel frames

Many multi-storey steel structures in New Zealand use moment frames or braced moment frames. Because of the high strength of steel, it is possible to design multi-storey steel structures with very stiff frames. Many buildings are designed to have low ductility or even an elastic response. The size of the steel members, which partially determines the strength of the frame, is often controlled by other factors, such as gravity and wind loads, rather than seismic loads. However, they must be designed with sufficient ductility to avoid a sudden failure in a large earthquake, for example, from buckling.

2.6 Steel bracing

Steel is an effective material for bracing steel frames, and eccentric braced frames are widely used in New Zealand. There are also a significant number of low to medium-rise buildings that use concentrically braced frames. The braces may be designed to work in tension or compression. Generally, steel braces have a high degree of strength and stiffness as well as high ductility , which also gives them a damping characteristic. Some braces enhance this effect by adding additional dissipative components to the brace. However, steel braces, especially eccentric steel braces, are susceptible to buckling if the brace is not sufficiently reinforced, and repeated tension and compression cycles can cause the steel to fatigue and break. The steel bracing system shown in Figure.5

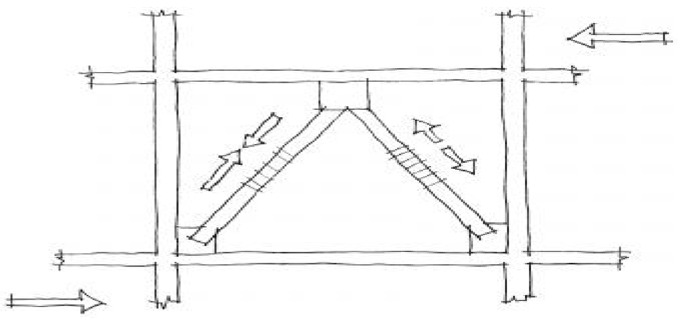

Figure.5 Eccentric bracing showing damping effect under lateral load.

2.7 Post-tensioning in steel

Post-tensioning in steel uses the same principle as the post-tensioned techniques described for concrete. The system uses a series of jointed steel members connected together with post-tensioning tendons to create a moment frame. During a minor earthquake, the additional frame stiffness provided by the tendon tensioning reduces movement of the structure. When exposed to larger lateral seismic forces, the structure will oscillate or rock as the elastic action of the tendons allows gaps to open and close between individual members within the frame. The tendons pull the structure back into its original position as the shaking subsides. Some designs place damping devices across the gaps to dissipate energy during the rocking. A relatively new idea that is still under development proposes to 'tie down' the entire structure simultaneously by running a single post-tensioned steel tendon from the top of the frame to the foundation. Post-tensioning in steel system shown in Figure.6

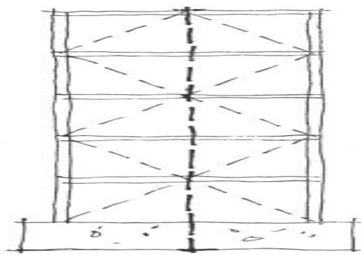

Figure.6 Post-tensioning in steel system

2.7 Flexibility and low weight

There are other advantages for steel structures in a seismic zone, namely their flexibility and low weight. Stiffer and heavier structures attract larger forces when an earthquake hits. Steel structures are generally more flexible than other types of structure and lower in weight (as discussed below). Forces in the structure and its foundations are therefore lower. This reduction of design forces significantly reduces the cost of both the superstructure and foundations of a building. Steel structures are generally light in comparison to those constructed using other materials. As earthquake forces are associated with inertia, they are related to the mass of the structure and so reducing the mass inevitably leads to lower seismic design forces. Indeed some steel structures are sufficiently light that seismic design is not critical. This is particularly the case for halls/sheds: they create an envelope around a large volume so their weight per unit surface area is low and wind forces, not seismic forces, generally govern the design. This means that a building designed for gravity and wind loads implicitly provides sufficient resistance to earthquakes. This explains why in past earthquakes such buildings have been observed to perform so much better than those made of heavy materials.

2.8 Damages to Beam-column Connections During Eathquakes

The factors that contributed to the damage include the following (FEMA, 2000):

- Stress concentration at the bottom flange weld, due to the notch effect produced by backing strips left in place,
- Poor welding practices, including the use of weld metal of low toughness
- uncontrolled deposition rates

- The use of larger members than those previously tested or the use of higher strength girders,
- Less system redundancy and higher strain demands on connections,
- Lack of control of basic material properties (large variation of member strength from the prescribed values)
- Inadequate quality control during construction, and
- The tri-axial restraint existing at the center of beam flanges and at the beam-column interface, which inhibits yielding.

III PRE-QUALIFIED MOMENT CONNECTIONS

AISC 358-2010 gives the following pre-qualified connections:

1. Reduced–Beam section connection
2. Bolted un-stiffened and Stiffened Extended End-Plate Moment Connections
3. Bolted Flange Plate moment connection
4. Welded Un-reinforced Flange-Welded Web Moment Connection
5. Kaiser Bolded Bracket Moment connection

3.1 Reduced–Beam Section Connection

In reduced beam section (RBS) moment connection (also known as the 'dog bone' connection), some portions of the beam flanges are removed in a pre-determined fashion, adjacent to the beam-column connection, as shown in Fig.7. In such a connection, yielding and plastic hinges are forced to form away from the connection at the reduced section of the beam.

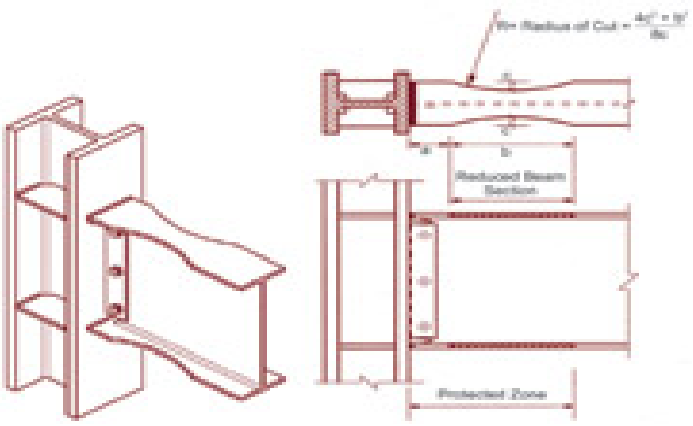

Figure .7 Reduced–Beam Section Connections

The effect of dog bone is similar to that of cover plate connections. With cover plates the connection is made stronger than the beam by strengthening the connection. In the dog bone, the connection is effectively made stronger than the beam by weakening the beam. While producing the same effect of cover plates, the dog bone connection can be constructed with relatively simpler details, resulting in a more reliable and economic solution. Moreover, the strong-column and weak-beam design can easily be achieved. flanges such as straight cut, taper cut, arc cut, and drilled flanges have been tested and the arc cut was found to provide favourable results.

3.2 shown in Figure .8 Bolted Un-stiffened and Stiffened Extended End-Plate Moment Connections

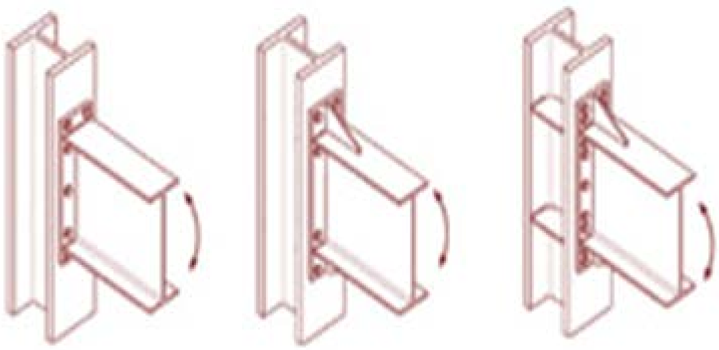

Figure .8 Bolted Un-stiffened and Stiffened Extended End-Plate Moment Connections

Bolted end plate connections are made by welding the beam section to an end plate which is in-turn bolted to the column flange. Three types of these connections are pre-qualified by AISC 358. It gives equations to check the various limit states of this type of connection such as flexural yielding of the beam section or end plate, yielding of column panel zone, shear or tension failure of the end-plate bolts, and failure of the various welded joints. These provisions are intended to ensure inelastic deformation of the connection by beam yielding.

3.3 Bolted Flange Plate (BFP) Moment Connection

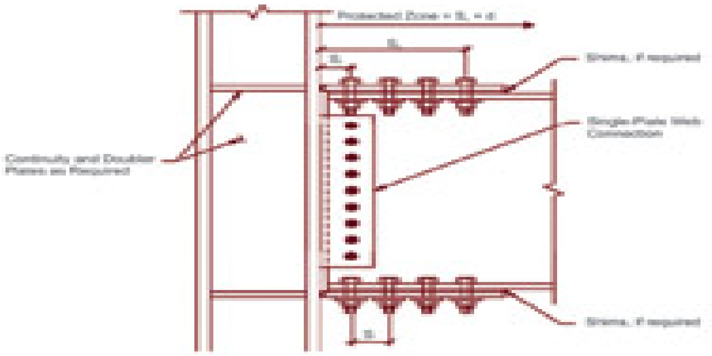

Figure.8 Bolted Flange Plate Moment Connection

These connections consist of plates welded to column flanges and bolted to beam flanges as shown in Figure.8 Identical top and bottom plates are used. Flange plates are connected to column flange by using complete joint penetration (CJP) groove welds and beam flanges are connected to the plates by using high strength friction grip bolts. The web of the beam is connected to the column flange using a bolted single-plate shear connection, with bolts in short-slotted holes. In this connection, yielding and plastic hinge formation are designed to occur in the beam near the end of the flange plates. The design procedure for this type of connection is more complex than other pre-qualified connections.

3.4 Welded Un-reinforced Flange-Welded Web Moment Connection

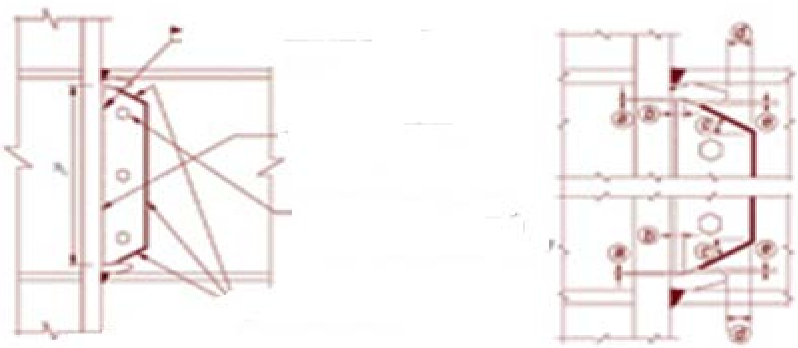

Figure.9 Welded Un-reinforced Flange-Welded Web Moment Connection

Unlike other pre-qualified connections, in the welded un-reinforced flange-welded web moment connection, the plastic hinge location is not moved away from the column face. Rather, the design and detailing features are intended to allow it to achieve Special Moment Frame performance without fracture. In this connection the beam flanges are welded directly to the column flange using CJP groove welds. The beam web is bolted to a single-plate shear connection for erection. This plate is used as a backing bar for welding the beam web directly to column flange using CJP groove weld, which extends to the full depth of the web .A fillet weld is also used to connect the shear plate to the beam web, as shown in Fig.9. A special seismic weld access hole and detailing, as shown in Figure. 9

3.5 Kaiser Bolted Bracket Moment Connection

Figure.10 Kaiser Bolted Bracket Moment Connection

In Kaiser bolted bracket moment connection, a cast steel (high-strength) bracket is fastened to each beam flange and bolted to the column flange as shown in Figure 10. The bracket can be either bolted or welded to the beam. The bracket is proportioned to develop the probable maximum moment strength of the beam, such that yielding and plastic hinge formation occurs in the beam at the end of bracket away from the column flange. This connection is designed to eliminate field welding and facilitate erection.

The design procedure and detailing requirements for these connections are given in AISC 358-2010.

A quick review of the basic eight steel frame systems tabulated indicates that they are reasonable, but some have substantially more redundancy than others: (Figure 3)

1. The simple 1890-1920 steel framework with unreinforced brick infill: The system has vertical support with an infill system, which allows brick joint movement for energy dissipation. A good inexpensive system, which allows for repair of brick after an earthquake.
2. The 1910-1930 column-to-girder gusset plate connection with nominally reinforced concrete infill walls: A good low-cost steel riveted detail with concrete providing stiffness for controlling lateral drift.

3. The 1910-1940 trussed girder wind brace providing inexpensive drift control of the frame: The encasing concrete also provided substantial lateral stiffness, and forced the column sections to be stronger and stiffer and to create girder yielding, a good contemporary concept.
4. The 1920-1940 knee braced moment frame with concrete encasement provided a nominally stiff frame system.

5. The 1930-1970 riveted (or bolted) top and bottom girder connections to the column, creating a steel moment frame: The concrete encasement on buildings through 1960 enhanced the moment frame stiffness.

6. The 1950-1970 top & bottom bolted hunched girder moment frame provided inexpensive girder stiffness and was especially strong if encased with cast-in-place concrete.

7. The more recent 1970-2000 all welded girder moment frame which only relied on the steel system for seismic resistance and the most flexible of steel frames. These steel systems were not encased in concrete and were clad with precast concrete, metal panels, or glass. After the Northridge Earthquake these conventionally welded frames were generally vulnerable. A major FEMA funded study has attempted to find solutions to this very significant problem. The current solutions tend to be expensive and suggest alternative answers.

8. The 1995-2000 steel moment frames with a dual system of dampers, or un bonded braces or eccentric braced frames, all clad with light-weight materials appear to be good solutions.

Conclusion

There seems to be an increasing earthquake activity throughout the world. The recent earthquakes have demonstrated that the damages and loss of lives will be extensive if the buildings are not designed and detailed properly. Though the recent version of steel code contained provisions for seismic design and detailing, designers are not given guidance to choose proper beam-to-column connections. Some of the research articles have given splice connection gives best seismic performance in steel connections.

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12. IS - 800 Code of Practice for General Construction in Steel, Bureau of Indian Standards, New Delhi, 2007.

13. IS - 1893 -2002 “Criteria for Earthquake Resistance and Construction of Buildings” Bureau of Indian Standards, New Delhi.