8 SEISMIC DESIGN

8.1 Principles

Steel moment frames are widely used for seismic loading. The seismic resistance and behaviour of these frames were highly regarded in USA and Japan prior to the Northridge and Kobe earthquakes. Unfortunately, some steel frames showed damages during these earthquakes, and today engineers are trying to better understand these structures. The unexpected damage effectively invalidated the then-existing design and construction procedures for pre-qualified beam-to-column connections. An extensive research programme was developed to gain a better understanding of why moment connections in special moment resisting frames performed poorly during the earthquake. There is consensus that numerous factors must have contributed to the observed failures including factors that pertain to welding detailing, and design practice that were prevalent prior to the earthquakes.

Detailing practices that appear to have played a role in the earthquake fractures include details that resulted in the development of large stress concentrations, excessive local ductility demands and high tri-axial restraint at the beam-to-column interface.

Poor welding practices include the common use of low toughness weld metal and insufficient quality control. Design practices suspected to have contributed to poor performance include the use of significantly larger members and connections than had previously been tested and the development and use of design provisions than can result in excessively weak panel zones. The standard prEN 1993-1-8 includes rules to determine the strength and stiffness of the steel joints. The influence of the seismic or dynamic loads, which may be considerable in seismic areas, is not covered. Steel structural joints designed for seismic zones must satisfy the following basic conditions:

- over-strength demand;
- ductility demand (rotation capacity);
- robustness demand (reliable detailing together with material behaviour).

8.2 Design criteria

The research leading to the development of design guidelines for new moment resisting steel frame buildings are based on a significant literature search and new analytical and laboratory research on seismic demands, base materials, welds, and fracture mechanics, as well as on full scale and subassembly testing of actual connections. In addition, the study results in new, more rigorous, requirements for quality control of materials and fabrication processes.

In the USA, the guidelines cover designs for frames with different anticipated seismic demands as permitted by the 1997 NEHRP Provisions [BSSC, 1997] and the AISC Seismic Provisions [AISC, 1997], namely Ordinary Moment Resisting Frames (OMRF), Intermediate Moment Resisting Frames (IMRF), and Special Moment Resisting Frames (SMRF). The three frame types are required to be qualified for plastic rotation capacities of *0,01*, *0,02* and *0,03* radians respectively and connections can be either fully restrained (FR), or partially restrained (PR). The guidelines provide recommendations for the selection of the appropriate system, for redundancy, for system analysis, for frame design issues, and for the design of suitable connections to meet the anticipated rotation demands of the system selected.

Moment frame performance is a function of many interrelated factors and design relationships, which affect the overall frame performance, as well as, in some cases, the connection performance. Recent studies are intended to determine the effects and importance of, and lead to guidelines for, the following:

- Strong Column/Weak Beam design principle;
- Panel zone strength;
- Connection strength and degradation characteristics;
- P-δ effects;
- Member local buckling.

Connection design guidelines are broken into certain issues generic to most or all connection design and specific design guidelines for the connection types determined to be pre-qualified. The following connection design conditions are considered to be generic, that is, they are conditions which, when they occur in a connection, are considered to perform in a similar way, or at least to have the same requirements for successful performance, irrespective of the connection type being used.

Welded Joints:

- Through-Thickness Strength;
- Base Material Notch-Toughness;
- Weld Wire Notch-Toughness;
- Weld backing and Run out Tabs;
- Reinforcing Fillet Welds;
- Cope Hole Size, Shape, Workmanship.

Bolted Joints:

- Bolt Sizing, Hole Type, Tightening;
- Net Section Strength.

In Europe, prEN 1993-1-8 gives the steel joint characteristics, as follows:

The moment resistance of a joint $M_{j,Rd}$ should be determined from prEN 1993-1-8 for welded or bolted connections.

The rotational stiffness of a joint S_j should be determined from the flexibilities of its basic components, each represented by its elastic stiffness coefficient k_i. Provided that the axial force N_{Sd} in the connected member does not exceed 10% of the resistance $N_{pl,Rd}$ of its cross-section, the rotational stiffness S_j of a joint, for a moment $M_{j,Sd}$ less than the moment resistance $M_{j,Rd}$ of the joint may be obtained with sufficient accuracy from:

$$S_j = \frac{E\,z^2}{\mu \sum_i \frac{1}{k_i}} \tag{8.1}$$

Only some provisions concerning the Rotation capacity of steel joints are given in prEN 1993-1-8, and insufficient for a realistic evaluation of the rotational capacity of a joint, especially for a seismic analysis, where rotational capacity is very important. Other important provisions concerning steel joints subject to seismic loads are given in Eurocode 8.

8.3 Beam-to-column typologies

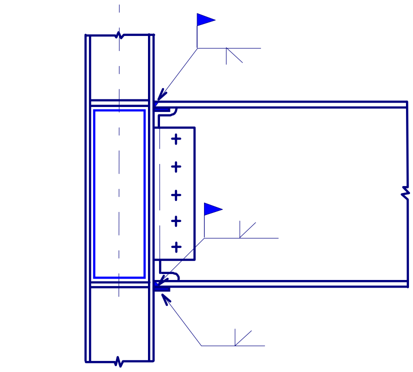
In USA the FEMA/SAC test programmes provided sufficient data to permit reliable design guidelines to be developed for at least ten types of welded connections. Each connection type is classified as suitable for certain ranges of member size, and of certain ranges of plastic rotation angle. The types of connections listed below are pre-qualified for use. Figures are provided to show the general configuration of each connection type.

- Welded Unreinforced Flange (WURF), similar to Figure 8.1a, but with design improvements;
- Welded Cover Plated Flange (WCPF);
- Welded Flange Plates (WFP), see Figure 8.1b;
- Welded Vertical Ribbed Flange (WVRF);
- Welded Column Tree with Bolted Beam (WCT/BB), see Figure 8. 1c;
- Welded Single Haunch (WSH);
- Welded Double Haunch (WDH).

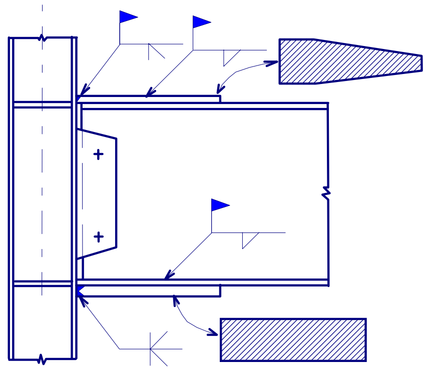
In addition to the welded connection types noted above, several field bolted types of connections are included in the guidelines as pre-qualified for certain conditions of use. The types, which may be selected, are listed below.

- Bolted End Plate (BEP), see Figure 8.1d;
- Welded Flange Plates with Bolted Beam (WFPBB), see Figure 8.1e;
- Bolted Single Haunch (BSH), see Figure 8.1f;
- Bolted Double Haunch (BDH), see Figure 8.1f.

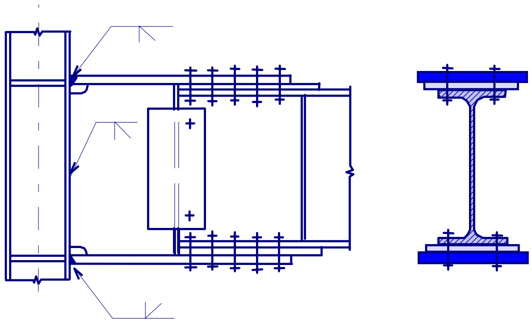
Some specific joints utilised in Japan are presented in Figure 8.2. Figure 8.3 shows some usual joints utilised and tested in Europe [Mazzolani, 2000].



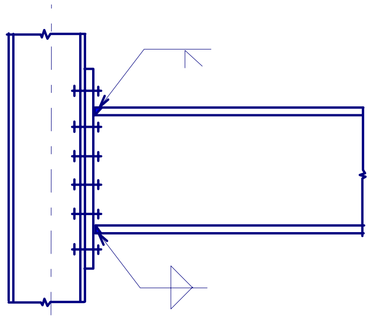
a) *Prescriptive Moment Frame Connection*



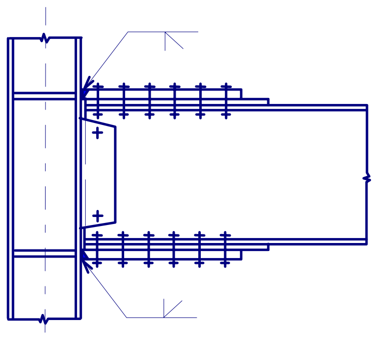
b) *Welded Flange Plate Connection*



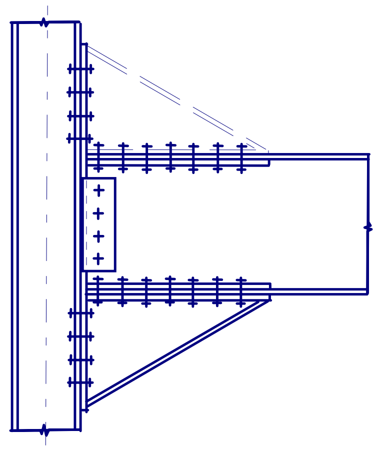
c) *Welded Column Tree with Bolted Beam*



d) *Bolted End Plate Connection*

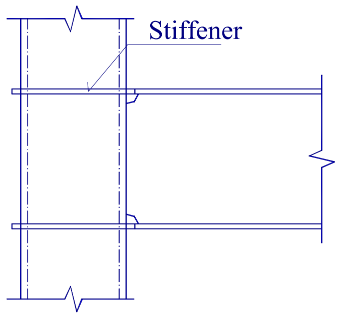


e) *Welded Flange Plates with Bolted Beam*



f) *Bolted Haunch (Double Haunch)*

Figure 8.1 Specific joints in USA



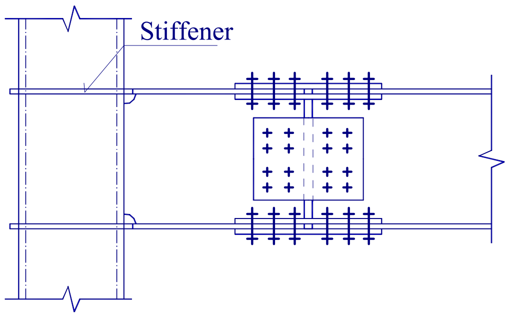
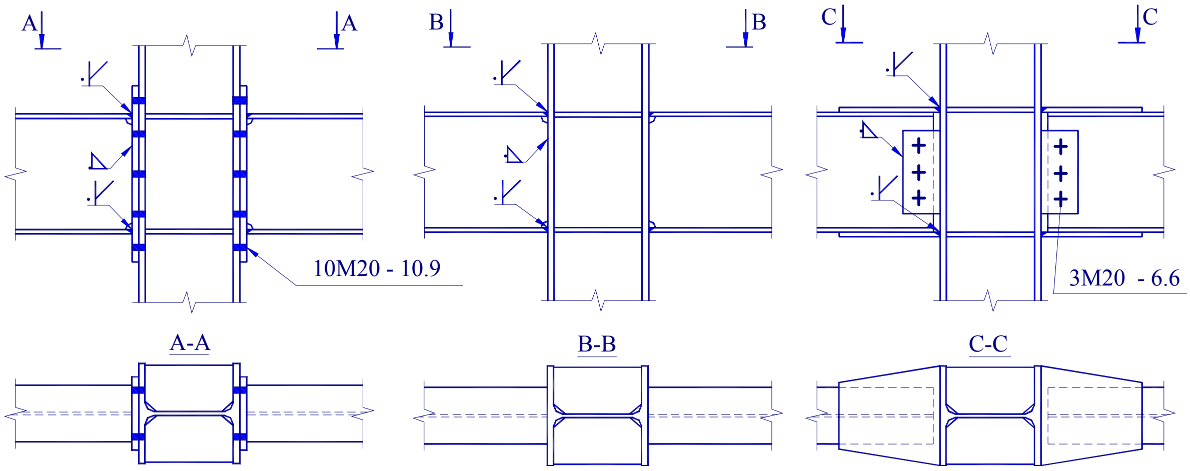


Figure 8.2 Specific joints in Japan



Extended End Plate Joint *Welded Joint* *Welded Flange Plate Joint*

Figure 8.3 Specific joints in Europe

8.4 Design and fabrication recommendations

prEN 1998-1 [prEN 1998-1, 2003] provides the following general rules for steel connections in dissipative structures:

- The adequacy of design should prevent localisation of plastic strains, high residual stresses and fabrication defects. The adequacy of design should be supported by experimental evidence.
- Non dissipative connections of dissipative members made by means of full penetration butt welds are deemed to satisfy the overstrength criterion.
- For fillet weld or bolted non dissipative connections, the following requirements should be met:

$$R_d \geq 1{,}35\, R_{fy} \tag{8.2}$$

- Only categories B and C of bolted joints in shear and category E of bolted joints in tension should be used, with controlled tightening of the bolts. Shear joints with fitted bolts are also allowed.
- For bolted shear connection, the shear resistance of the bolts should be higher than 1,2 times the bearing resistance.
- The strength and ductility of members and their connections under cyclic loading should be supported by experimental evidence, in order to comply with specific requirements defined for each structural type and structural ductility classes. This applies to all types of connections in dissipative zones. The requirements on ductility are expressed for various structural types in Clauses 6.6 and 6.9. When expressed in term of available plastic rotation

$$\phi_p = \delta/(0{,}5\, L) \tag{8.3}$$

The same standard prEN 1998-1 provides the following requirements for MRF (Moment Resistant Frame) beam-to-column connections:

- If the structure is designed to dissipate energy in the beams, the connections between the beams and the columns should be designed for the required degree of overstrength, taking into account the moment resistance $M_{pl.Rd}$ and the shear force ($V_{G,Ed}+V_{M,Ed}$) evaluated in 6.6.2 of standard prEN 1998-1.
- Dissipative semi-rigid and/or partial strength connections are permitted provided that all of the following conditions are satisfied: a) the connections have a rotation capacity consistent with global deformations; b) members framing into the connections are demonstrated to be

stable at the ultimate limit state (ULS); c) the effect of connections deformation on global drift is taken into account.

- The connection design should be such that the plastic rotation capacity ϕ_{Cd} in the plastic hinge, is not less than *35 mrad* for structures of ductility class H and *25 mrad* for structures of ductility class M with $q>2$. These values should be obtained under cyclic loading without degradation of strength and stiffness greater than 20%; they should be supported by experimental evidence. This requirement is valid independently of the intended location of the dissipative zones.
- When partial strength connections are used, the column capacity design should be derived from the plastic capacity of the connections.

The influence of local geometric details and material properties on the inelastic behaviour of fully restrained steel connections has been investigated in different countries in the last years. Some of the conclusions are presented bellow [El-Tawil et al, 2000], [Mao et al, 2001]:

Material properties – Yield-to-Ultimate Stress Ratio (YUSR)

Connections with YUSR (f_y/f_u) equal to *0,65* or *0,80* behaved in a similar manner for a plastic rotation capacity up to *0,030 rad*. Compared with these two specimens, connection with YUSR = *0,95* had a significantly reduced plastic hinge length at a plastic rotation capacity of *0,030 rad*. The plastified length of the beam with YUSR = *0,95* was about half the corresponding length in YUSR = *0,80*. The smaller plastified length increased the local strains, which in turn promoted earlier local buckling. These elevated strains also increase susceptibility to low cycle fatigue failure.

Local details – Access Hole Size and Geometry

Increasing the size of the web cope would permit easier welding on the beam bottom flange, and possibly promote better weld quality [Chi et al, 2000]. However, the analyses suggest that it is important to use a small access hole to minimise the potential for ductile fracture at the root of the hole. The analysis confirms that the access hole in which the web terminates perpendicular to the flange is inferior to the semicircular detail from the ductile fracture point of view.

Local details – Continuity Plates

FEMA-267 recommendations require the use of continuity plates in all connections. However, the analyses suggest that the provisions may be relaxed with regard to continuity plate thickness for one-sided connections.

Q&A 8.1 Connections Subject to Dynamic Load

Is prEN 1993-1-8 applicable to connections subject to dynamic load, particularly wind load, in addition to static loads?

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prEN 1993-1-8 is partially applicable to connections subject to dynamic load. Concerning the moment resistance $M_{j,Rd}$ and the initial rotational stiffness $S_{j,ini}$ of the joint, the relations given by prEN 1993-1-8, can be utilised for dynamic loads.

Concerning the rotation capacity of steel joints, provisions given in prEN 1993-1-8 do not cover the typology of steel joints. However, an experimental research programme, has observed that the rotation capacity of joints subject to dynamic load is approximately *0,5* times the rotation capacity determined under a static load.

The collapse mechanism could also be modified in dynamic behaviour, compared with the collapse mechanism determined from prEN 1993-1-8 for static loading, according to the weakest component of the joint, see [Grecea, 2001].

Q&A 8.2 Influence of Unsymmetrical Loading

Is prEN 1993-1-8 applicable to connections subject to seismic load, particularly unsymmetrical seismic load, in addition to static loads?

The conclusions of a research program developed in European countries [Mazzolani, 2000] are that the values of the characteristics given by the prEN 1993-1-8 cannot be utilised in these conditions of loading.

A comparison between the values given in prEN 1993-1-8 and the characteristics determined on joints subject to cyclic symmetrical loading was given in the previous answer. A project comparing joints subject to cyclic symmetrical loading and joints subject to cyclic unsymmetrical loading was carried out with some interesting conclusions. Concerning the moment resistance, the values obtained with the second group are smaller by 20-40%, depending of the joint configuration. The rotation capacity of the second group was 150-200% times the rotation capacity of the first group, depending on the joint configuration.

Q&A 8.3 Influence of Strain-Rate Loading

What is the influence of strain-rate loading on the behaviour of steel joints?

An experimental research programme [Mazzolani, 2000] concluded that the strain-rate loading has an important influence on the behaviour of joints.

A strain rate in the range of $0,03\text{-}0,06\ s^{-1}$ (typical for steel members yielding under seismic action) increases the yield strength and, to a lower extent, the ultimate strength of welded connections; for structural steel this phenomenon is well known. Ductility is reduced by up to 27%. However, a decrease of ductility due to high strain rates is not straightforward for cyclic loading, where the results are rather scattered.

Q&A 8.4 Welding Technology

What is the influence of welding technology and detailing on the behaviour of steel joints?

Experimental tests performed under the COPERNICUS RECOS [Mazzolani, 2000] project showed the importance of the quality of welding. Three types of welds were studied (double bevel, fillet and single bevel), and the "ideal" behaviour (rupture in the base metal) was observed for the double bevel welds. This fact is attributed to the lack of defects for this welding procedure, in comparison to the other two procedures [Dubina, Stratan, 2002]. Fillet welded specimens were characterised by an intermediary behaviour, the main cause of failures in welds being the undersized welds. This fact demonstrates one of the disadvantages of this type of welding, which is the difficulty to control and verify the weld size, particularly on a building site. Single bevel weld specimens had, in general, an unsatisfactory behaviour, due to incomplete penetration at the root of the weld, which initiated cracks in the weld region.

The behaviour of the three weld types have to be analysed in conjunction with the technological and economical aspects of welding. Single bevel and double bevel welds require supplementary mechanical operations (edge preparation), while double bevel and fillet welds require overhead position in the case of site welding. The following recommendations are proposed for the particular case of beam-to column joints in moment-resisting frames:

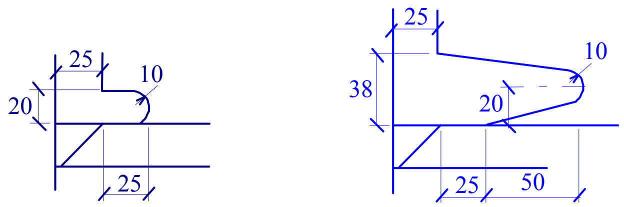
- Double bevel and fillet welds are recommended for shop welding of the subassemblies, with the condition of strict verification of weld size in the case of fillet welds.

- Single bevel welds are adequate for site welding, but re-welding of the root is compulsory in order to eliminate defects at the root of the weld.

Cyclic loading increases the probability of weld fracture for partial joints (undersized fillet welds), and for welds with defects (single bevel welds). It is to be stressed that weld defects such as undersized fillet welds and incomplete penetration single bevel welds were observed both for beam-to-column joints tests, as well as for welded connection tests. This fact shows the need for the strict control of welding quality.

Specific tests performed in USA and Japan showed that US construction uses self-shielded flux core metal arc welding (FCAW), while Japanese construction uses gas shielded metal arc welding (GMAW) with carbon dioxide (CO_2) shielding gas. The comparison suggests that GMAW is more costly, but it may provide greater toughness.

In numerous inelastic cyclic connection tests with notch tough beam flange welds, fracture initiating at the toe of the weld access holes was observed. An effort was made to study the influence of access hole geometry and size on the potential of ductile fracture initiation near the holes. The results indicate the importance of selecting a proper weld access hole configuration. The modified weld access hole geometry shown as configuration b) in Figure 8.4 is recommended for seismic restraint design in comparison with the standard configuration a) [Mao et al, 2001].



a) standard *b) modified*

Figure 8.4 Configurations of weld access hole

The web attachment detail was found to have a significant effect on the fracture potential of the beam flanges near the interface of the weld metal and base metal. Based on the analysis results, a detail consisting of a complete penetration groove welded beam web in conjunction with supplemental fillet welds on the shear tab is recommended for seismic design. Weld procedures using a notch tough electrode for the beam web groove weld must be carefully followed to minimise weld defects. In addition, a stronger panel zone is recommended. The bolted shear tab detail is recommended for use only in ordinary moment resisting frames.

Different analyses provide evidence that with higher toughness materials and significant detailing improvements, such as backing bar removal, slight overmatching of weld strengths, and limited panel shear deformations, connections can achieve the target inelastic rotation of *0,030 rad* prior to weld root fractures.

Q&A 8.5 High Strength Bolts in Seismic Joints

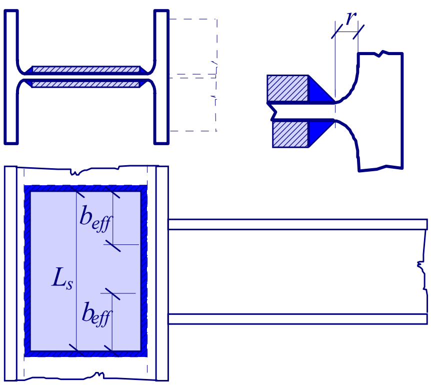
Is it possible to use High Strength bolts as ordinary bolts in seismic joints?

High strength bolts (in US HSFG, High Strength Friction Grip bolts) can be used as ordinary bolts in seismic joints. It is recommended that they are tightened at a level of 50% of their preloading force. In this case the surfaces of the plates do not have to be prepared for working as a slip-resistant connection, see Q&A 6.8.

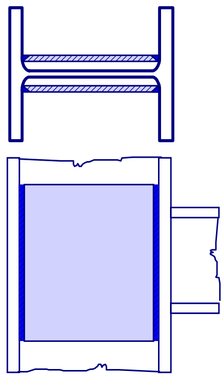
Q&A 8.6 Column Web Panel

What is the influence of the column web panel in a joint subject to dynamic loads?

The behaviour of the column web panel is described prEN 1993-1-8, but only for static loads. In the case of T joints or double T joints with unsymmetrical loads, the web column panel has a strong influence on the behaviour of the joint. For specific dynamic and seismic unsymmetrical loads, it was demonstrated, that the resistance of the joint is reduced by between 20 - 40% and the ductility is increased by 150 - 200%, due to the web panel. Adding supplementary web plates on the column web panel, as shown in Figure 8.5, can increase the resistance of the joint. The solutions have advantages and disadvantages. The first solution, see Figure 8.5a, is the optimum for static loads (the effective with formulas for b_{eff} see in prEN 1993-1-8) and requires supplementary plates to be welded to the toe of the root radius. This introduces additional stresses into the joint. In the second solution, see Figure 8.5b, welding the supplementary plate to the flanges avoid the additional stresses in the flange/web, see [Dubina et al, 2000]. This solution reduces the length of the flange and may cause problem for minor axes beams.



a) welded to the toe of the root radius



b) welded on the flanges

Figure 8.5 Supplementary web plates

Concerning the subject of seismic behaviour of steel frames with deformable panel zones, an interesting conclusion was made by [Schneider, Amidi, 1998] after a numerical study, saying that current provisions governing the minimum strength design of panel zones might result in joints with high shear distortions, which increases the potential for flange fracture of the girder framing into the joint. These results suggest that the 1991 NEHRP provision requiring the panel zone strength to be greater than 90% of the sum of the beam strengths at the joint appears to be a minimum.