DRAFT TECHNICAL COMMENTARY

IS 18168:2023

Steel Structures for Seismic Forces

Eccentrically Braced Frames (EBFs)

Clause-wise Technical Review and Recommendations

This document is a preliminary technical commentary on the provisions for Eccentrically Braced Frames (EBFs) in IS 18168:2023, prepared to facilitate technical discussion, critical review, and contributions from researchers, academicians, and practising structural engineers.The commentary draws upon IS 18168:2023, relevant international standards, published research, and established literature. Certain technical provisions, equations, and terminology are reproduced or summarized for the purpose of explaining the engineering rationale and identifying potential areas for clarification or improvement. Relevant sources are acknowledged wherever applicable. This document is a working draft and is subject to revision following technical review and contributions from all of you structural steel enthusiast.

C11 Shear Links

Shear links are a deformation-controlled element (DCE) in an eccentrically braced frame. They are specifically designed to yield in shear or flexure before any other structural element in the intended capacity design hierarchy of EBF system.

Depending on its length, a link may yield primarily in shear (short links) or in flexure (long links) or both in shear and flexure combined, and the current standard's (IS 18168:2023) provisions are oriented towards promoting shear yielding (provision of short links) wherever possible. Shear yielding engages the link's web uniformly along its length, producing stable, repeatable hysteretic behaviour with large inelastic rotation capacity, without the strength and stiffness degradation that flexural hinges tend to show (flange local buckling becomes the limiting criteria) after a few cycles of large rotation which can be seen Fig. 13a. Flexural yielding, by contrast, concentrates plastic strain in a narrow region at the ends of the link, making the response more sensitive to local flange

buckling (Refer to Fig. 13c) and sensitive to the detailing of the plastic hinge region. Consequently, flexural links generally exhibit lower inelastic deformation capacity.

It is suggested that Clause 11 be retitled "Links of Eccentrically Braced Frames" instead of "Shear Links" to avoid implying that EBF links are exclusively shear-dominated.

C11.1 Sections

Only I-shaped cross sections (standard rolled wide flange sections or built-up sections) or built-up box sections are allowed as Link sections. Hollow Structural Sections (HSS) are generally not recommended for use as EBF links due to concerns regarding their low-cycle fatigue resistance under cyclic inelastic deformations. Experimental studies have shown that HSS links are susceptible to local buckling and subsequent fracture, with cracks often initiating at the rounded corners due to stress concentrations, cold-forming-induced plastic strains, and residual stresses. In some cases, fracture has been observed to propagate rapidly following the onset of local buckling (Bruneau et al., 2026). Although built-up box sections are also susceptible to stress concentrations, residual stresses, and potential cracking at welded corners, the absence of cold-forming-induced corner strains, together with appropriate weld detailing, fabrication quality control, and material toughness, can help mitigate these concerns and improve their cyclic deformation capacity. Although HSS sections have been prohibited to be used as link, more research on this cross section can help its adoption similar to built-up box sections.

Reinforcement of link webs using doubler plates is not permitted, as reliable composite action and uniform participation of the plates during cyclic inelastic deformation cannot be assured. Additionally, beam web penetrations within the link are not permitted because they may adversely affect the link's inelastic behaviour (AISC 341-22).

Extremely tall and narrow boxes can experience lateral-torsional buckling, and hence provision of Links of built-up box sections shall have a moment of inertia, I_y, about an axis in the plane of the EBF greater than $0.67I_x$, where I_x is the moment of inertia of the link about an axis perpendicular to the plane of the EBF.

C11.2 Link Shear Strength

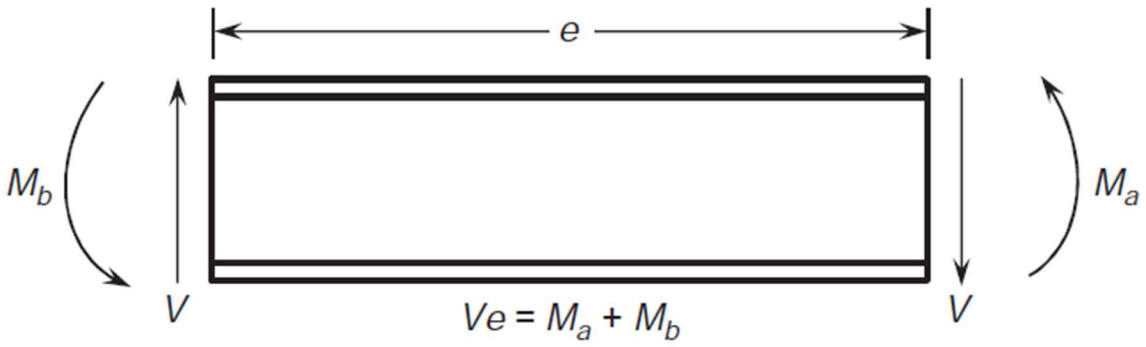


Fig 1: Free body diagram of Link (Bruneau et al., 2026)

Figure 1 shows the free-body diagram of the link. For a balanced yielding condition, i.e., for yielding in shear and flexure to occur simultaneously, the link length can be expressed as $e_b = 2M_p/V_p$. When $e_o < e_b$, theoretically a shear yielding is expected to be governed, and when it is the converse, a flexural yielding will govern.

The effects of axial load on the link can be ignored if the factored axial load demand on the link does not exceed 15% of its axial yield strength. However, when the framing arrangement (EBF configuration) allows larger axial forces to develop in the link, the available shear strength and the link length are to be reduced.

C11.3 Length of link

Even though the choice exists to provide shear governed or flexure governed link, the code currently mandates the use of shear links. Shear yielding results in uniform web yielding which produces stable, fat hysteresis loops with large ductility, whereas flexural yielding concentrates strain at the link ends, making it more prone to local buckling and strength degradation.

Test results (not related to Indian steel) showed that a properly stiffened short link can strain harden and develop a shear strength equal to $1.5Vp$. The end moments of a link that has yielded in shear can continue to increase due to this strain hardening. To avoid high bending strains that may lead to severe flange local buckling or to failure of connection welds, these end moments are limited to $1.2Mp$, and the maximum length, e, for a shear link is modified as shown in equation (1).

$$e = \frac{2(1.2)M_p}{1.5V_p} = 1.6M_p/V_p$$

Now, for a flexural hinge, i.e., when the moment in the link reaches $1.2M_p$, and when $e = 2.6M_p/V_p$, the corresponding shear is $V = 0.92V_p$. Therefore, when $e \geq 2.6M_p/V_p$, flexural hinge is formed. Finally, when $\frac{1.6M_p}{V_p} \leq e < 2.6M_p/V_p$, simultaneous flexural and shear yielding occurs. These limits are depicted in Fig. 2.

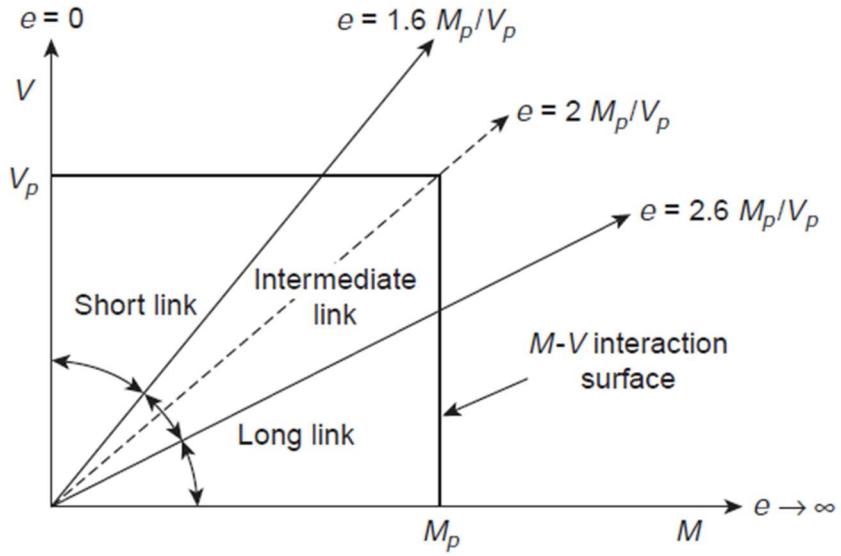


Fig 2: Classification of Links (Bruneau et al., 2026)

The rotations achievable in links subjected to flexural yielding under high axial forces have not been adequately studied. Consequently, where high axial forces can develop in the link, its length is limited to ensure that shear yielding, rather than flexural yielding, governs to ensure stable inelastic behaviour (AISC-341:22).

Figure 3 shows the failure of short, intermediate, and long links. In short links (Fig. 3(a)) the yielding of the web is observed through the flaking of the paint/whitewash during the experimental test. In the intermediate link (Fig. 3(b)), both the yielding of the web and the local buckling of the flange at the ends of the link is also visible indicating the combined action. Finally, for the long links (Fig. 3(c)), failure occurs due to flange local buckling.

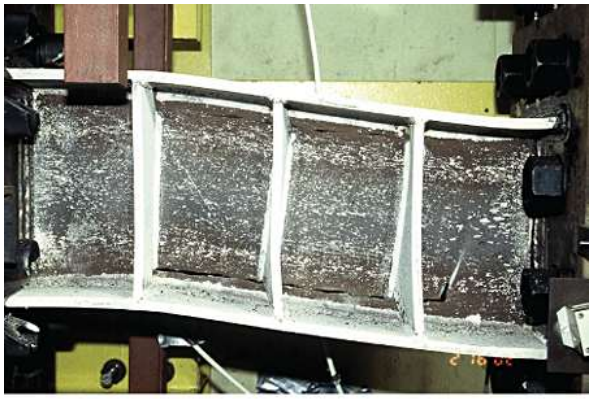


Fig. 13(a): Link failure - Short Link (Bruneau et al., 2026)

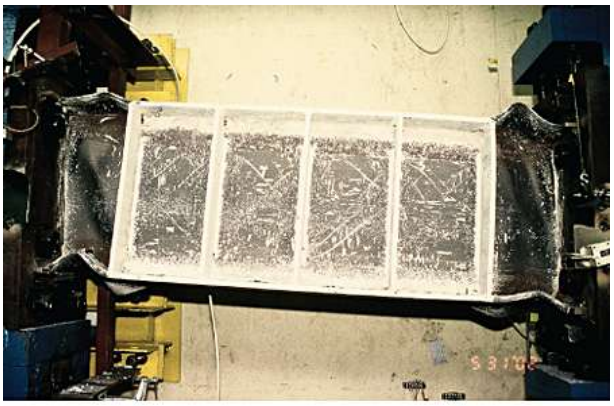


Fig. 13(b): Link failure - Intermediate Link (Bruneau et al., 2026)

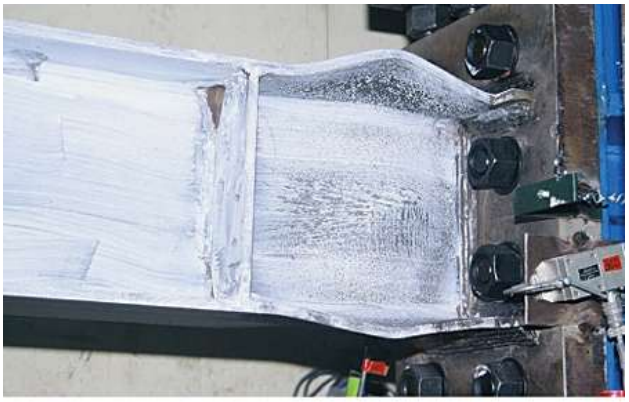


Fig. 13(c): Link failure - Long Link (Bruneau et al., 2026)

C11.4 Stiffeners for I-shaped Link Sections

Figure 4 shows two I-shaped links tested cyclically (Malley & Popov, 1983). For the specimen without stiffeners, web local buckling (limit state for shear links) due to shear would occur early. Such local buckling could be delayed by adding transverse stiffeners (Bruneau et al., 2026). Also, as seen in Figure 5 below, the energy dissipation capacity of the stiffened link is better than that of the unstiffened ones.

A properly detailed and restrained link web can provide stable, ductile, and predictable behaviour under severe cyclic loading. The design of the link requires close attention to the detailing of the link web thickness and stiffeners. Full-depth stiffeners are required at the ends of all links and serve to transfer link shear forces to the reacting elements and to restrain the link web against buckling. (AISC:341-22)

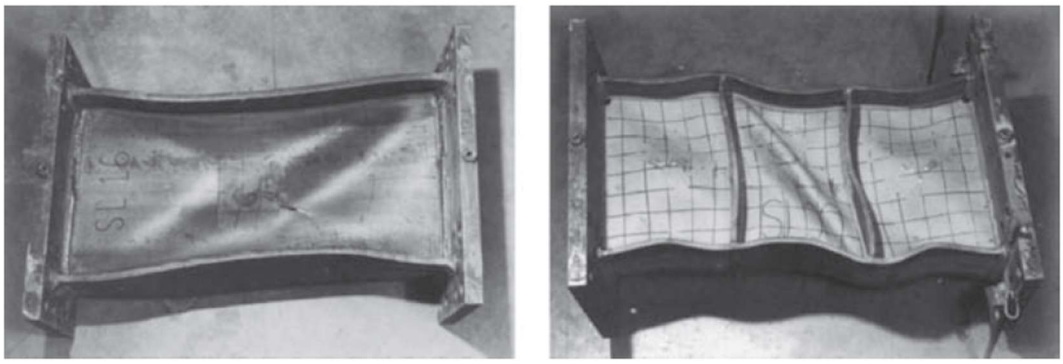


Fig. 4: Unstiffened versus stiffened link (Malley et al., 1983)

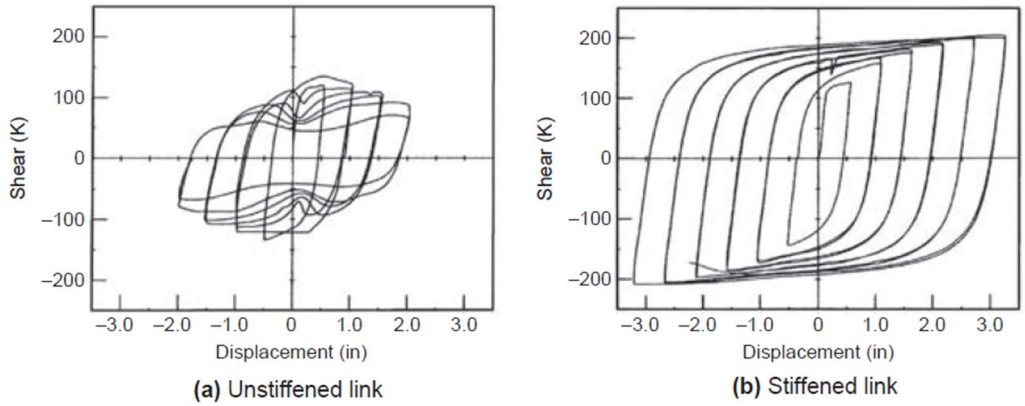


Fig. 5: Cyclic response of unstiffened and stiffened link (Hjelmstad et al., 1983)

C11.4.1 End Web Stiffeners

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than ($b_f - 2t_w$) and a thickness not less than the larger of $0.75t_w$ or 10 mm, where b_f and t_w are the link flange width and link web thickness, respectively.

C11.4.2 Intermediate Web Stiffeners

Links are subjected to inelastic rotation during seismic events. The link rotation angle is the plastic rotation of the link relative to the portion of the beam outside the link. The links need sufficient deformation capacity to accommodate this deformation demand. Testing shows that a link's inelastic rotation capacity is dependent on the link's length—the shorter the length, the larger the rotation capacity (Kasai et al.,1986a). To develop a large rotation capacity, closely spaced intermediate stiffeners are needed. Allowable link deformation capacity adopted by AISC-341:22 is shown in Figure 6 below.

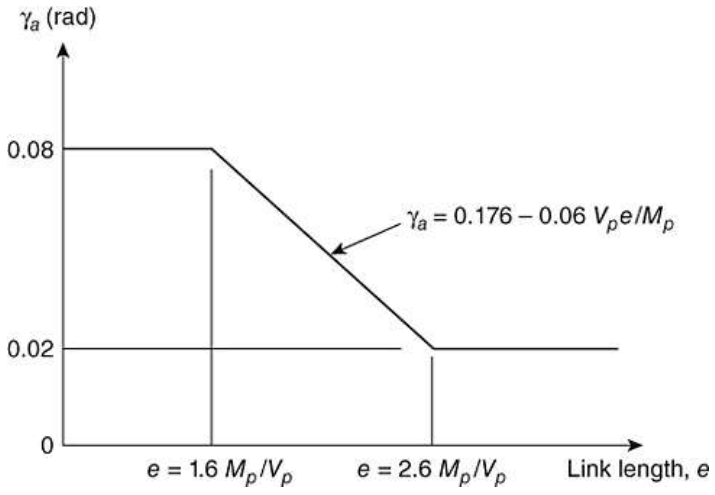


Fig 6: Allowable link rotation angles adopted by AISC341-22 (Bruneau et al., 2026)

Accordingly, AISC-341:22 states that,

- Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding ($30t_w - d/5$) for a link rotation angle of 0.08 rad or ($52t_w - d/5$) for link rotation angles of 0.02 rad or less. Linear interpolation shall be used for values between 0.08 and 0.02 rad.

- Links of length greater than or equal to $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at 1.5 times b_f from each end of the link.
- Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements as specified in the preceding two provisions.
- Intermediate web stiffeners shall not be required in links of length greater than $5M_p/V_p$. (AISC:341-22)

The Indian code has adopted the conservative version of this provision and has limited the maximum Intermediate stiffener spacing to $30t_w - d/5$, irrespective of the link length. Further tests on Indian sections can help in relaxing these provisions.

C11.5 Stiffeners for Box Link Sections

C11.5.1 End Web Stiffeners

The clause here is adopted from AISC 341-22. Full-depth web stiffeners shall be provided on each side of each link web at the diagonal brace connection. These stiffeners may be welded to the outside or inside face of the link webs. These stiffeners shall each have a width not less than $b/2$, where b is the inside width of the box section. Each stiffener shall have a thickness not less than the larger of $0.75t_w$ or 10 mm.

C11.5.2 Intermediate Web Stiffeners

This clause provision is also similar to those provided in AISC 341:22. AISC clause states that, box links shall be provided with intermediate web stiffeners as follows:

- For links of length $1.6M_p/V_p$ or less, and with a web depth-to-thickness ratio, h/t_w, greater than or equal to $0.67\left(\frac{E}{RyFy}\right)^{0.5}$, full-depth web stiffeners shall be provided on one side of each link web, spaced at intervals not exceeding $20t_w - (d - 2t_f)/8$.
- For links of length $1.6M_p/V_p$ or less and with a web depth-to-thickness ratio, h/t_w, less than $0.67\left(\frac{E}{RyFy}\right)^{0.5}$, no intermediate web stiffeners are required.

- For links of length greater than $1.6M_p/V_p$, no intermediate web stiffeners are required.

Though the Indian clause has a limit to the web depth-to-thickness ratio as $19\frac{\epsilon}{\sqrt{f_y}}$*, It should have been* $19\frac{\epsilon}{\sqrt{R_y}}$*, which is like the AISC clause above. The coefficient in the AISC clause, after substituting* $E = 210000\ N/mm^2$*, comes to around 300. After substituting for* ϵ *in the Indian provision, the provision simplifies to* $\frac{300.41}{\sqrt{R_y f_y}}$*. So this typographical error needs to be corrected.*

The equation for maximum spacing needed for the links to develop a link rotation angle of 0.08 rad, specified as $20tw - (d - 2tf)/8$, is derived in Berman and Bruneau (2005a). The use of intermediate web stiffeners was shown (Berman and Bruneau, 2006, 2008b, 2008c) to have a significant effect on the shear yielding strength in built-up box links with h/t_w greater than $0.64\left(\frac{E}{RyFy}\right)^{0.5}$ and less than or equal to $1.67\left(\frac{E}{RyFy}\right)^{0.5}$. For h/t_w less than $0.64\left(\frac{E}{RyFy}\right)^{0.5}$ flange buckling was the controlling limit state, and intermediate web stiffeners had no effect; hence, there is no requirement for Intermediate stiffeners for such links. (Berman and Bruneau, 2006, 2008b, 2008c) as in (AISC 341-22). As the presence of stiffeners didn't impact flange buckling, welding the stiffeners to the flange may be unnecessary, though in the testing done by Berman and Bruneau (2006, 2008b, 2008c), these stiffeners were welded to both the webs and the flanges. (AISC-341:22) and hence, in the Indian Code, it has been stated to weld to the outside or inside face of link webs.

C12.3 Eccentrically Braced Frame

Eccentrically braced frames (EBF) of structural steel shall be designed to satisfy the requirements given hereunder.

C12.3.1 Basis of Design

Moment resisting frames (MRFs) are ductile but don't have enough stiffness, and their design, in most cases, is governed by code-specified drift limits. Concentrically braced frames, known for their high lateral stiffness, have lower ductility than MRFs. Eccentrically Braced Frames (EBFs) are said to have the advantages of both lateral load-resisting systems: high stiffness comparable to SCBFs and high ductility comparable to MRFs. It is obvious in the Response Reduction factors given in IS 1893 Part-1, with the highest R given for MRFs and EBFs. EBF dissipates seismic

energy imparted by the shear yielding of links. So the design should ensure the desired capacity hierarchy such that the link yields in shear first. Shear has been mentioned here as IS18168:2023 mandates for short shear links. Additionally, the current code promotes the use of link to beam connection as shown in Fig. 4 of the code, and avoids the use of link-to-column connection.

For the connection of the brace away from the link, where it meets beam-column joint, the code mentions that eccentricities less than the beam depth are permitted in the brace connection if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.

The provisions below ensure that cyclic yielding in the links can occur in a stable manner while the diagonal braces, columns, and portions of the beam outside of the link remain essentially elastic under the forces that can be developed by fully yielded and strain-hardened links (AISC-341:22).

C12.3.2 Analysis

The requirements specified hereunder shall be satisfied in the analysis of EBFs.

C12.3.2.1

The shear demand on the links shall be determined based on the analysis required by IS 1893 (Part 1), i.e., the force capacity of the link shall be greater than the internal forces obtained through analysis of the structure for the load combinations mentioned in IS 1893 Part 1: 2016, as well as other relevant codes like IS 800:2007.

The seismic design philosophy of an Eccentrically Braced Frame (EBF) is based on capacity design, wherein the inelastic response is intentionally concentrated within a short segment of the beam known as the link, while the braces, columns and the remaining portions of the beams are proportioned to remain essentially elastic. By localizing damage in a replaceable link element, the frame combines the high stiffness and strength of concentrically braced frames with the superior ductility and energy dissipation characteristics of moment-resisting frames.

The link acts as a structural fuse, dissipating earthquake energy through stable cyclic yielding while protecting the remaining structural members from significant inelastic deformation. Consequently, the braces, beam segments outside the link, beam-column connections and columns

are designed following the principles of capacity design so that yielding occurs preferentially within the link.

C12.3.2.2

The required strength of the elements other than the link, i.e., beams, columns, braces, and connections, shall be determined based on the expected over-strength capacity of the link for the reasons mentioned in C12.3.2.1. This can be done by following the column tree method analysis, where a section is cut in the middle of the links of the frame, and the expected over-strength of the link (V_{mh}) shall be applied at the cuts and analysed for the load combinations in which Dead load and live load shall have the usual load combination multipliers (1.2 or 1.5), and 1 as load multiplier for expected over-strength capacity of the link**.** The force-controlled elements, i.e., the beams, braces, and columns, shall also be designed for the forces obtained from the analysis of the Load Combinations mentioned in Clause 5.5 of the code and shall be designed for the largest forces obtained from both these methods.

Limited yielding outside of the links, particularly in the beams, is sometimes unavoidable in an EBF. Such yielding is unlikely to be detrimental to the performance of the EBF if the beam and brace have sufficient strength to develop the full inelastic strength and deformation capacity of the link. The relaxation in link forces for beam design is done with the view that their strength shall be increased substantially by composite action with the floor slab, and experimental evidence suggests that limited yielding of the beam doesn't affect the performance of the EBF, if the stability of the beam is ensured (Engelhardt & Popov, 1989). Since the inelastic deformation in the beam outside the link is expected, the expected material yield stress is allowed to be used to calculate the strength of the beam outside link.

In most EBF configurations, the diagonal brace and the beam are subjected to large axial loads combined with significant bending moments. Consequently, both the diagonal brace and the beam should be designed as beam-columns (AISC:341-22).

Analyses shall be performed for each direction of frame loading. For systems that include columns that form part of two intersecting frames in orthogonal or multi-axial directions, the analysis shall consider the potential for link yielding in both directions simultaneously (AISC:341-22).

C12.3.3 System requirements

The requirements specified hereunder shall be satisfied in the analysis of EBFs.

C12.3.3.1 Link rotation angle

The limits of plastic rotation angle are like those in AISC:341-22 and have been decided based on past experimental evidence.

It has been demonstrated experimentally (Whittaker et al., 1987; Foutch, 1989) and analytically (Popov et al., 1989) that links on the first floor usually undergo the largest inelastic deformation. In extreme cases, this may result in a tendency to develop a soft story. The plastic link rotations tend to attenuate at higher floors and decrease with the increasing frame periods. However, in high rise buildings the upper floors are also expected to result in hinging in the links. Regardless, the code mandates the rotation of the link to be limited to 0.08 radians, which is again the limit for a short shear link.

C12.3.3.2 Bracing of link

Bracing shall be provided at both the top and bottom flanges of the link at the ends of the link for I-shaped sections. Such bracings shall have stiffness and strength as specified in 6.3.3.

Lateral restraint against out-of-plane displacement and twist is required at the ends of the link to ensure stable inelastic behaviour. A link with a built-up box cross-section, tested without lateral bracing in a full EBF configuration, exhibited no lateral-torsional buckling (Berman & Bruneau, 2007), and as a result, there is no requirement for lateral bracing for box link sections.

C12.3.4 Member Requirements

The requirements specified hereunder shall be satisfied.

C12.3.4.1 Basic requirements

Brace members, beams outside the links, and columns shall satisfy the width-to-thickness limitations specified in **5.3**. Apart from columns, the beams and braces in EBFs may be subjected to significant axial and bending forces; hence, their design capacities shall be determined as for beam-column members as per IS 800.

The link member and column is required to satisfy the section classification requirements for plastic sections. However, Clause 5.3 of IS 18168:2023 also requires the beam outside the link to satisfy the requirements for plastic sections, which may be unnecessarily conservative since these beam segments are intended to remain essentially elastic. In comparison, AISC 341-22 permits the beam outside the link, when it is a different section from the link, and the diagonal braces to satisfy the width-to-thickness limits for moderately ductile members. Furthermore, for short shear links with ($e \leq 1.6M_p/V_p$), AISC 341-22 permits the flanges of I-shaped links and the webs of built-up box links to satisfy moderately ductile width-to-thickness limits. These relaxations may be considered in the Indian provisions.

C12.3.4.2 Links

The link length *e* shall be considered to extend from brace connection to brace connection for centre links. In addition, requirements specified in 11 shall apply.

C12.3.4.3 Protected zones

The protected zones of EBFs are the links. Therefore, no welding or cutting or changes should be made in this location. Use of shear studs on the links is also prohibited.

C12.3.4.4 Beam-to-column connections

In an EBF, inelastic deformation is intended to occur primarily in the links, while the beams outside the links, braces, columns, and connections are expected to remain essentially elastic. However, limited secondary yielding of beams and columns may occur under severe earthquake loading. Where the brace or gusset plate connects to both the beam and column at a joint, significant moment transfer may develop through the connection assembly. Accordingly, IS 18168:2023 requires the connection assembly to be designed for the amplified expected plastic moment capacity of the beam, ($1.1R_y f_{yb} Z_{pb}$), to prevent premature connection failure. Additionally, the sum of the expected column flexural strengths is required to exceed this moment, thereby reducing the likelihood of column yielding and the formation of an undesirable storey mechanism. This connection moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the over-strength earthquake load.

In comparison, AISC 341-22 permits alternative connection arrangements, including a simple connection capable of accommodating a rotation of 0.025 rad, or a moment-resisting connection designed for the lesser of the amplified expected flexural strengths of the beam and columns, subject to the applicable requirements. Thus, the Indian provision is more restrictive, as it mandates both the amplified beam moment resistance of the connection assembly and a column-over-beam strength hierarchy, even though the primary energy dissipation mechanism in an EBF is intended to be confined to the links. Similar connection design alternatives may therefore be considered in the Indian provisions.

C12.3.4.5 Braces

This clause is in conflict with the Clause 12.3.2.2 of IS18168:2023, where in the force controlled members have been mandated to be designed for the expected over-strength capacity of the link defined by 1.1 $R_y S_h$ times the design strength of the links. So this clause needs to be combined with Cl. 12.3.2.2 for more clarity.

C12.3.4.6 Brace connections

The provision permits limited slip in bolted brace connections with oversized holes, primarily to accommodate erection tolerances and avoid excessively stringent slip-resistance requirements. However, it should be clarified that this relaxation applies only to the slip limit state, while the connection must continue to satisfy bolt shear, bearing and other applicable strength limit states under the amplified forces corresponding to the expected link strength as directed through Clause 5.5. *The reference to Clause 12.3.2.2 based force demand also needs to be included.*

If it is intended to resist the link end moments using the braces, then a fully restrained connection needs to be provided. This arrangement helps in reducing the flexural demand on beam outside the link. However, the brace needs to be then designed as a beam-column element.

C12.3.4.7 Column splices

In addition to the requirement of Clause 7.5, the clause 12.3.4.7 provides the force demands in flexural and shear with which the splice needs to be designed.

C12.3.4.8 Demand critical welds

Demand-critical welds are welds at locations susceptible to significant inelastic cyclic demands during earthquakes, for which enhanced toughness and quality requirements are specified to minimize the risk of brittle fracture and preserve the intended ductile seismic behaviour of the structure. The locations coming under demand critical weld are provided in this clause.

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