Future directions for capacity design of welded beam-to-column connections in steel seismic moment resisting frames

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Synopsis

Beam-to-column connections adjoining plastic hinges in beams are required to be designed for the overstrength plastic moment of the beam and the associated equilibrium shear force. Three factors influence this overstrength plastic moment in the beam, namely uncertainty in yield stress of steel, effect of strain-hardening on the beam cross-section, and local instability of the beam flanges and web due to slenderness. The effect of strain-hardening is different in non-seismic and seismic moment frames due to different plastic rotation demands on them. This paper presents an explicit procedure for the capacity design of welded beam-to-column connections, which accounts for the expected strain hardening for any desired level of plastic rotation. An idealised shear-moment interaction boundary is used as a guide to estimate the maximum probable bending moment expected to be developed in the beam during strong earthquake shaking, and a step-wise procedure for design of a beam-to-column moment connection is presented through a specific example of a cover-plated and ribbed connection.

Notations

The following symbols are used in this paper:

b	Breadth or width
b_{bf}	Breadth of beam flange
b_{cp}	Breadth of cover plate
d_p	Depth of beam
d_c, d_{c1}, d_{c2}	Depth of column
f_u	Ultimate normal stress
f_y	Minimum specified normal yield stress of steel
f_y^*, f_{yact}	Actual yield stress of steel
h_{rp}	Height of the rib plate
l	Length
l_c	Length of cover plate
l_{ph}	Length of the plastic hinge
l_t	Distance of the plastic hinge from the end of connection reinforcement end
l_{whrp}	Length of fillet weld between cover plate and outer rib plates
l_{wpc}	Length of fillet weld between connection plates and beam flange
t_{bf}	Thickness of beam flange
t_{bw}	Thickness of beam web
t_{cp}	Thickness of cover plate
t_{rp}	Thickness of the rib plate
t_{wcp}	Thickness of fillet weld between connection plates and beam flange
t_{whrp}	Thickness of fillet weld between cover plate and outer rib plates
A_{rp}	Area of rib plate
A_{wcp}	Area of fillet weld between the cover plate and rib plate
A_{whrp}	Area of fillet weld between cover plate and outer rib plates
E	Young's modulus of steel
I	Moment of inertia of the section
I_b	Moment of inertia of the beam section
L	Distance between the centre-lines of the columns
L_o	Length of shear-link
M	Bending moment
M_0	Overstrength plastic moment in beam in strain-hardened state
M_{pl}	Beam plastic moment capacity using minimum specified yield strength
$M_{pl,Rd}$	Design plastic moment resistance of beam
M_{pr}	Connection design moment
M_{Rd}	Design moment resistance of beam
M_y	Yield moment
R	Overstrength factor
R_c	Overstrength factor due to compactness
R_s	Overstrength factor due to strain hardening
R_y	Overstrength factor due uncertainty in the estimation of yield strength
T	Thickness
T_{cp}	Yield capacity of cover plate
T_{cp}, T_{cp}^*	Yield tension capacity of cover plate
T_d	Design pull force for the top half of the connection
T_f	Yield tension capacity of beam flange
T_{rp}	Design pull force for the rib plate
T_{rp}^*	Yield tension capacity of rib plate
T_{wcp}	Horizontal force transferred by fillet welds between connection plates and beam flange
T_{whrp}	Tension force on fillet weld between cover plate and outer rib plates
V	Shear force
V_0	Equilibrium shear force in the beam when overstrength plastic moment hinges are formed at the beam-ends
V_d	Design shear for the top half of the connection
V_{pl}	Section plastic shear capacity using minimum specified yield strength
V_{pr}	Connection design shear
V_{rp}	Design shear force for the rib plate
V_{wcp}	Shear force transferred by fillet welds between connection plates and beam flange
V_{whrp}	Shear force on fillet weld between cover plate and outer rib plates
V_y	Yield shear
W_{pl}	Plastic section modulus of the beam
Y	Failure stress in Von Mises criterion
α	Shear-moment interaction factor
β	Beam overstrength factor
ε	Normal strain
ε_r	Rupture strain
ε_{sh}	Strain-hardening strain
ε_u	Ultimate strain
ε_y	Yield strain
φ	Curvature
φ_{max}	Maximum curvature
φ_y	Yield curvature
φ_y^*	Idealised yield curvature
γ_{M0}	Partial safety factor for resistance
γ_{gov}	Overstrength factor
λ_{pl}	Limiting slenderness parameter for compact section
λ_{ov}	Limiting slenderness parameter for non-compact section
μ	Curvature ductility of the section
μ_y	Yield curvature ductility
μ_{sh}	Strain-hardening curvature ductility
μ_u	Ultimate curvature ductility
σ	Stress
σ_{xx}	Normal stress
τ_{xz}	Shear stress
τ_y	Minimum specified shear yield stress of steel
θ_{pl}	Joint plastic rotation
θ_p	Plastic rotation in beam
$\theta_{pl,max}$	Maximum plastic rotation in beam

Introduction

Extensive connection failures in steel frame buildings during the 1985 Michoacan (Mexico), 1994 Northridge (USA) and 1995 Kobe (Japan) earthquakes led to a significantly different approach for the seismic design of beam-to-column moment connections. Beam-to-column connections are now required to (a) remain elastic and not undergo inelasticity or brittle-mode failure in the connection elements, and (b) resist the maximum probable beam moment and corresponding shear expected during strong seismic shaking when plastic hinges are formed in the beam. The former is achieved by reinforcing the connection region with additional elements like cover plates and rib plates. And, the latter is achieved by a realistic estimate of the upper-bound moment and shear imposed by the beam on the column. The above two requirements are in line with the strong-column weak-beam design philosophy, and use the capacity design concept.

In the seismic design of connections, first the maximum moment demand on the beam is estimated from elastic analysis of the structure subjected to the design loads amplified by the code-specified load factors. Then, a beam section is chosen to satisfy the moment demands from all the code-specified load combinations. This beam section is checked for safety under the amplified shear forces obtained from the various code-specified load combinations. Under strong seismic shaking, the beam is expected to undergo significant strain-hardening at the plastic hinges, i.e. the moment developed M_0 at a plastic hinge is higher than the nominal plastic moment capacity M_{pl} ($=f_y W_{pl}$, where f_y is the characteristic yield stress and W_{pl} is the plastic section modulus), which is based on idealised rectangular stress block. As per the capacity design concept, this beam is also checked for safety against the equilibrium shear forces V_0 generated when overstrength plastic hinges are formed at the beam ends (Fig 1). The connection adjoining the plastic hinge is now required to carry the effect of V_0 and M_0 without undergoing any inelastic action.

The first concern in the estimation of M_0 (and hence V_0) is that the characteristic yield stress f_y as assumed in the design of beams cannot be used in the design of connections, and the *actual* values of f_y as used in beam designs is a lower-bound estimate, which is intended to achieve conservative beam designs. However, for a safe design of beam-to-column connections, an upper-bound of the forces from the beam on to the connections is desirable. Mill data of coupon tests has shown that the *characteristic* and *actual* f_y values can be higher than the characteristic value up to 30% in A36 steel and up to 50% in A572 Grade 50 steel [Englehardt and Sabol, 1998; Malley and Frank, 2000]. The second concern in the estimation of M_0 is that strain-hardening of the beam section will depend on the extent of plastic rotation imposed on the beam during seismic shaking. During seismic shaking, as the rotation demand on the beam increases, the fibres in the beam cross-section strain-harden based on the actual stress-strain curve of structural steel; this is in contrast to the idealised elastic-perfectly plastic stress-strain curve of steel. Recent experimental studies [e.g., Engelhardt and Sabol, 1998] have confirmed that the beam capacities larger than M_{pl} are achievable. The designer needs to know the expected level of rotation (some codes specify these [e.g. UBC 1997; FEMA 1995]), and also have a procedure for relating this expected rotation to the level of strain-hardening in the beam causing an increase in the beam moment. The third concern is that if the beam is not suitable for sustaining large plastic rotations, its cross-section may sustain local buckling or the beam itself may undergo lateral torsional buckling. While the first two factors increase the beam moment from its nominal plastic moment value, the third causes a drop. However, it may be conservative to take the third factor as unity.

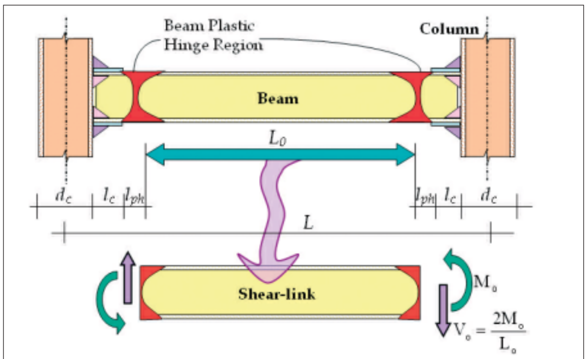

Fig 1. Overstrength moments at ends and associated equilibrium shear force in beam when plastic hinges are formed under strong seismic shaking

Thus, the main challenge in seismic design of welded steel beam-to-column connections (or for that matter, even of bolted connections) is in estimating shear and moment demands for the expected level of inelastic action. Understandably, when the equilibrium shear V_0 is being sought under the action of moment developed at the plastic hinges M_0 acting at beam ends, the beam cross-section will place a restriction on the relative values of V_0 and M_0 due to interaction between the normal and shear stresses acting on it. This paper presents a quantitative procedure for arriving at these demands V_0 and M_0, which considers all the above issues and which is consistent with most requirements imposed by the modern seismic codes of practice. This procedure is general enough to be valid even if the codes alter numerical limits on some parameters, like the expected plastic rotational demands on the connections.

Codes of practice

Design codes across the world use different strategies to address the seismic design of beam-to-column moment connections. Three modern codes of practice are discussed here, namely New Zealand Standard [NZS 3404, 1997], American Institute of Steel Construction code *Seismic Provisions for Structural Steel Buildings (SPSSB) 1997* [AISC, 1997] and the Eurocode 8 [preEN 1998 (Part 1.1), 2003]. These codes are in general agreement with each other and require that the connections must be robust enough to transfer the beam-end forces to the columns. However, they adopt different strategies for controlling the seismic behaviour of connections and estimating design demands on them. Also, neither of these codes makes an explicit choice *for* or *against* welded or bolted connections, even though they recognise that fully-restrained connections perform much better than partially-restrained ones; the designers are required to make this choice.

The New Zealand standard NZS3404 classifies structures into four categories depending on the level of overall ductility to be achieved in them, namely fully ductile structures as Category 1 ($\mu > 3.0$), limited ductile structures as Category 2 ($3.0 > \mu > 1.25$), nominally ductile as Category 3 ($\mu = 1.25$) and elastic structures as Category 4 ($\mu = 1.0$). For beams, increase in material strength f_y alone up to 30%, increase in section strength M_{pl} due to strain-hardening of the section alone up to 15% and an increase up to 35% due to the combined effects of the above two are recognised explicitly in the code. The limiting plastic rotations at the plastic hinges are listed in the code for different members. Members that carry larger axial load and sustaining seismic actions are required to have smaller plastic rotations. For instance, beams in Category 1 structures need to sustain plastic rotations not more than 0.045 radians under seismic actions and 0.065 radians under non-seismic actions.

The American Institute of Steel Construction code SPSSB 1997 states the design requirement in terms of the inelastic rotation at the joint undergoing plastic actions to be at least 0.03, 0.02 and 0.01 radians in special moment frames (SMFs), intermediate moment frames (IMFs) and ordinary moment frames (OMFs), respectively. SMFs, IMFs and OMFs are expected to sustain significant, moderate, and limited inelastic deformations. There is no restriction on the ratio (f_y^*/f_y) of actual and characteristic yield strengths. Further, in contrast to the New Zealand Standards, here no overstrength factors or beam stability factors are specified. The designer has to determine these three factors to estimate the connection design forces, i.e., V_0 and M_0.

The Eurocode 8 suggests that the connection be designed for the overstrength plastic moment and associated shear force. In estimating the overstrength moment of resistance, reference is made to Eurocode 3 [DD ENV 1993 (Part 1), 1992]. Eurocode 3 suggests that design moment resistance M_{Rd} of the connection

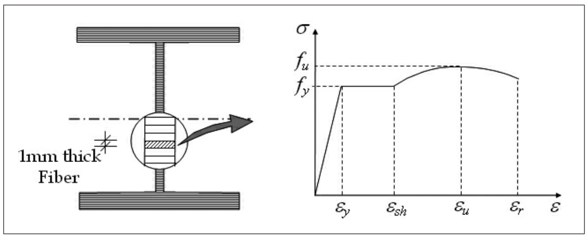

Fig 2.
Fibre model showing discretisation of a beam section and the explicit form of stress–strain relationship for steel [Murty and Hall, 1994] used in this study

be taken as at least 1.2 times the design plastic moment resistance $M_{pl,Rd}$ of the beam, to account for the overstrength (possibly meaning strain-hardening alone). However, in the estimation of $M_{pl,R}$d, it uses the partial safety factor for resistance γ_{M0} of 1.1; since upper bound strength of the beam is being sought, this strength reduction factor does not give an conservative estimate of the flexural demand from the beam. In addition, for connections adjoining dissipative zones, Eurocode 8 requires that either the actual yield strength f_{yact} be used in the calculations in place of the yield strength, or $1.1\gamma_{ov}f_y$ where the overstrength factor γ_{ov} to be used in design may be taken as 1.25 in the absence of national data. Interestingly, while all the quantities required for obtaining V_0 and M_0 are already provided, Eurocode 8 also requires that plastic rotation capacity of the plastic region is not less than 0.035 radians in ductility class *high* (DCH) structures and not less than 0.025 radians in ductility class *medium* (DCM) structures. This plastic hinge rotation requirement may be an over-specification for the connection design or it may even conflict with the overstrength factor stated previously.

Past studies on shear–moment interaction of beams

One important issue identified earlier in the paper was the interaction between the overstrength plastic moment M_0 and the associated shear V_0. In the early studies on shear-moment interaction for I-sections based on the maximum shear strength criterion for yielding [Hodge and Brooklyn, 1957], the yield strength f_y was assumed to be the limiting strength, and strain-hardening of steel was not considered. The shear-moment interaction was not found to be significant in shallow beams having depth-to-span ratios less than about 0.1. Approximate shear-moment (*V-M*) interaction curves were proposed for deep beams (plate girders) based on the tension-field action of the web [Basler, 1962]. The shear capacity of the web was reportedly not affected by the bending moment on the section so long as the flanges did not yield. The web shear capacity dropped quickly as yielding of the beam flanges increased. Even this study does not consider the effect of strain-hardening in steel. Based on another study [Cooper *et al*, 1978], the AISC-LRFD code [AISC, 1994] gave a linear interaction between shear and bending moment for plate girder design when the design shear was more than 60% of the factored nominal shear capacity and the bending moment was more than 75% of the factored nominal bending moment capacity of the section.

Present study

This paper presents a formal procedure that is uniformly valid for any configuration of moment connection suitable for seismic applications. However, this paper illustrates the procedure through the example of a *welded cover-plated and ribbed* steel moment resisting connection. A realistic stress-strain relationship for steel, the pre-determined location of plastic hinge formation in the beam, and a target plastic rotation at the plastic hinge, are used to assess the demand imposed by the beam on the connection elements. Shear-moment (*V-M*) strength envelopes are derived afresh for some parallel-flange hot-rolled sections (AISC W-sections [AISC, 1989]) and used as a guide to compare different connection design procedures in vogue with the proposed one.

Beam overstrength

In the rest of this paper, the ratio (f_y^*/f_y) of actual and characteristic (minimum specified) yield strengths is denoted as R_y. Also, the ratio of the beam bending moment achieved due to strain hardening (based on actual stress–strain curve) and the nominal plastic moment capacity (based on idealised elastic-perfectly plastic stress–strain curve) is denoted as R_s.

Shear–moment interaction with strain hardening

A fibre model is employed to obtain the *V-M* interaction curves for the AISC W-sections. The cross-section is sub-divided into fibres of 1mm thickness along the depth of the beam (Fig 2). The dimensions of the beam section are rounded-off to the nearest millimetre. A smooth and explicit form of stress–strain curve is used beyond the strain hardening strain [Murty and Hall, 1994] (Fig 2). The Von Mises yield criterion for steel represented by

$$\sigma_{xx}^2 + 3\tau_{xz}^2 = Y^2 \qquad \text{...(1)}$$

is used and Y is taken as the ultimate stress f_u of steel. The curvature is increased from zero to a value corresponding to the maximum strain ε_r at the extreme fibre, and at each of these curvatures, the shear and moment capacities are calculated.

The normalised *V-M* interaction curves for 13 AISC W-sections (W36×300, W33×240, W27×177, W21×142, W18×114, W16×96, W14×426, W14×84, W12×190, W12×58, W10×112 and W8×67) for R_y = 1.0, and the ratio f_u/f_y = 1.5 (corresponding to A36 steel) and f_u/f_y = 1.3 (corresponding to A572 Grade 50 steel) are shown in Fig 3. These values of f_u/f_y are based on the coupon tests reported in literature [Englehardt and Sabol, 1998; Malley and Frank, 2000]. The value of R_y is taken as unity while developing the *V-M* curves, and this factor is applied while calculating the final probable moment capacity of the beam (Eq. 7). Thus, the *V-M* curves are made independent of the uncertainty in the estimation of yield strength of steel. The uniaxial stress–strain curve of steel (Fig 2) has a drop in the stress beyond the ultimate strain ε_u. The limiting shear stress from Eq. (1) when $\sigma_{xx} = f_u$, is zero. For strains greater than ε_u, Eq. (1) suggests that the shear stress τ_{xz} in fibres is non-zero. However, in this study it is assumed that all fibres having strains beyond ε_u do not have shear capacity. While obtaining the limiting *V-M* boundary, it is assumed that as the beam develops ultimate strength, its flanges and webs do not buckle. The nominal shear strength ($V_{pl} = \tau_y t_{bw} d_b$) and the nominal bending moment capacity ($M_{pl} = f_y W_{pl}$) of the section are used to normalise the shear and moment capacities, respectively. The first yield shear stress $\tau_y = f_y/\sqrt{3}$ corresponding to a state of pure shear is used here.

The normalised *V-M* interaction curves so obtained for the 13 cross-sections are nearly identical (Fig 3). Furthermore, the interaction between V and M is weak for $M < M_y$ and for $V < V_y$. This is consistent with the tension-field action for shear transfer in the web as previously mentioned to be effective so long as the flanges remain elastic [Cooper *et al*, 1978]; yielding of flanges introduces additional shearing strain in the web, which rapidly reduces its shear capacity. Upper bounds of these normalised *V-M* interaction curves (Fig 3) can be represented by linear segments:

$$
\begin{aligned}
&\frac{V}{V_{pl}} = \beta && \text{for } 0 \le \frac{M}{M_{pl}} \le \frac{M_y}{M_{pl}} \\
&\frac{V}{V_{pl}} + \alpha\,\frac{M - M_y}{M_{pl}} = \beta && \text{for } \frac{M_y}{M_{pl}} \le \frac{M}{M_{pl}} \le \beta \qquad \text{...(2)} \\
&\frac{M}{M_{pl}} = \beta && \text{for } 0 \le \frac{V}{V_{pl}} \le \frac{V_y}{V_{pl}}
\end{aligned}
$$

where $\beta = R_y \dfrac{f_u}{f_y}$ is defined as the *beam overstrength factor* and

$$\alpha = \left(1 - \frac{V_y}{\beta V_{pl}}\right) \Big/ \left(1 - \frac{M_y}{\beta M_{pl}}\right)$$

as the *shear–moment interaction factor*. The point on the upper bound curve where the shear interaction becomes significant is taken as corresponding to $V = V_y$. This point is similar to M_y on the moment axis. However, unlike M_y, the value of V_y may not have any physical significance. Nevertheless, Eq. (2) is a good approximation for the upper bound of the *V-M* interaction boundary (Fig 3). For the section sizes considered in this study, the *V-M* interaction is very weak. The *V-M* surface proposed in this paper is useful for the design deep beams with larger shear capacities wherein *V-M* interaction is significant.

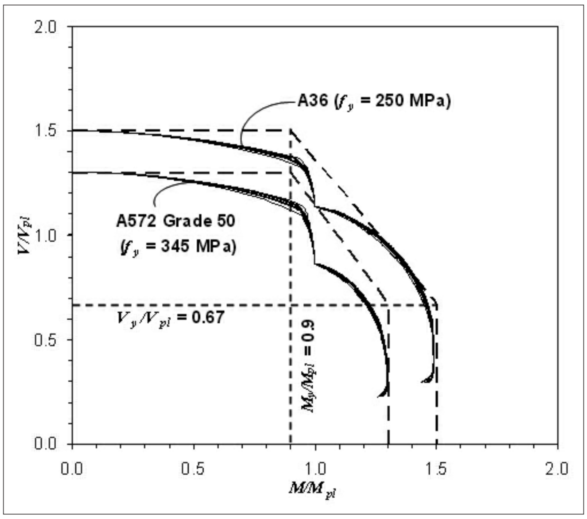

Fig 3.
Normalised V-M interaction surfaces for AISC W-sections with upper bound for minimum specified yield strength and f_u/f_y = 1.5 (f_y = 250MPa) and f_u/f_y = 1.3 (f_y=345MPa), and f^*_y/f_y = 1.0

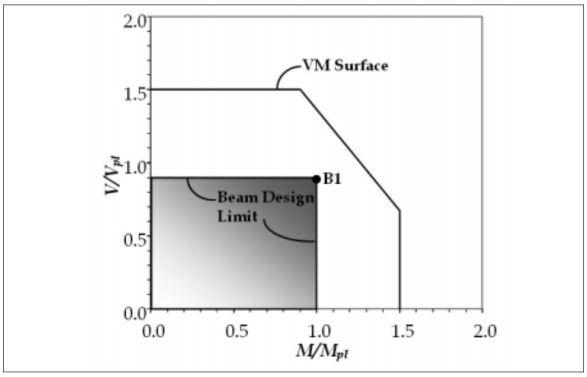

Fig 4.
***V-M* interaction surface used in design of beams with minimum specified yield strength f_y and f_u/f_y = 1.5 (for A36 steel)**

Curvature ductility

In the design of beams, a section whose M_{pl} is larger than the maximum bending moment demand M, is selected. Then, it is ensured that V_{pl} of this section is larger than the maximum shear demand V. On the *V-M* plane obtained with R_y=1.0 (Fig 4), point B1 corresponds to the upper bound of the code design procedure for beams, i.e., the design V and M can be anywhere below and to the left of point B1; this is indicated by the shaded region. Clearly, beams designed by this procedure possess large overstrength indicated by the area under the *V-M* curve beyond the shaded region.

One way of quantifying the post-yield strain hardening performance of a beam is through the curvature ductility μ at the section. μ can be estimated from the plastic rotation θ_p required to be developed at the beam end, using

$$\mu = \frac{\varphi_{max}}{\varphi^*_y} = \frac{EI\theta_{pl}}{M_{pl}\, l_{ph}} \qquad ...(3)$$

where φ_{max} is the curvature corresponding to maximum rotation of θ_{pl}; φ^*_y is the idealised curvature corresponding to M_{pl}; l_{ph} is the length of the plastic hinge in the beam. Fig 5 shows variation of normalised moment M/M_{pl} developed with curvature ductility μ (= φ/φ_y) imposed on the cross-section for the 13 AISC W-sections, with f_u/f_y = 1.5 (A36 steel) and R_y = 1.0. These curves imitate the stress–strain curve of steel (Fig 2). The M/M_{pl} versus μ curves for the different sections are close to each other and hence can be idealised by a single M/M_{pl} versus μ curve (Fig 5) having elastic, perfectly plastic and smooth strain hardening regions given by:

$$R_s = \frac{M}{M_{pl}} = \mu \qquad for\ 0 \le \mu \le \mu_u$$

$$R_s = \frac{M}{M_{pl}} = 1 \qquad for\ \mu_y < \mu \le \mu_{sh} \qquad ...(4)$$

$$R_s = \frac{M}{M_{pl}} = 0.83 + 0.02\mu - 2\times10^{-4}\mu^2 + 1\times10^{-6}\mu^3 - 2\times10^{-9}\mu^4 \qquad for\ \mu_{sh} < \mu \le \mu_u$$

where μ_y is the curvature ductility at idealised yield, μ_{sh} is the curvature ductility at which strain hardening begins, and μ_u is the ultimate curvature ductility. Here, the values of μ_y=1.0, μ_{sh}=11.2 and μ_u=128 are independent of the section size. Least square best fit method was used to obtain the best-fit curve for the range $\mu_{sh} < \mu \le \mu_u$. The ratio R_s of bending moment M developed due to strain hardening in the beam and the plastic moment M_{pl} based on minimum specified yield strength of steel, i.e. R_y = 1.0, is a measure of the strain hardening in the beam.

Most codes require a plastic rotation capacity of 0.03-0.04 radians to ensure good inelastic behaviour of the beam during strong ground shaking. For elasto-plastic idealisation of *M-φ* curve of the beam, the desired curvature ductility μ may be calculated using Eq.(3). The plastic hinge length l_{ph} can range between 0.5d_b and db. For l_{ph}=0.5d_b, μ is given by:

$$\mu = \frac{2EI\theta_{pl}}{M_{pl}\, d_b} \qquad ...(5)$$

For A36 (f_y = 250MPa) steel and the beams used in this study, μ ranges from 2-30 for θ_{pl} varying between 0.01 and 0.04 radians. Thus, R_s is in the range 1.0-1.28. The value of 1.2 recommended by Eurocode 3 for R_s is reasonable. R_s given by Eq.(4) is obtained using f_y=250MPa and f_u/f_y=1.5. Further investigations are required to study the influence of f_y and f_u/f_y on R_s. The above procedure has a distinct advantage that the designer can determine the curvature ductility of the beam from the plastic rotation demand on the beam-to-column connection.

Compactness of the section

Premature local buckling of the flanges and web can adversely affect the maximum strength of the beam. For this reason, AISC-SPSSB provisions [AISC, 1997] allow the use of lower beam capacity (0.8R_yM_{pl}) for the design of a connection when the beam section is expected to buckle before reaching its maximum capacity. But, whether the beam buckles prematurely or not needs to be verified through experiments. However, an analytical method is required to encourage designers to implement this provision. In this regard, the AISC-LRFD provisions [AISC, 1994] on the compactness of the section based on b/t ratio can be used to decide the local stability of the beam. According to those provisions, the section is *compact* if b/t is less than λ_{pl}, *slender* if b/t ratio is more than λ_r, and *non-compact* if b/t ratio is between λ_{pl} and λ_r, where λ_{pl} and λ_r are the slenderness parameters as defined in the AISC code [AISC, 1994]. Thus, if the b/t ratio classifies the section to be slender, then the beam buckles locally prior to reaching its maximum capacity. Since the beam capacities M/M_{pl} *versus* curvature ductility μ shown in Fig 5 are based on the assumption that local bucking will not occur, a compactness factor R_c is introduced to account for the reduction in the maximum achievable beam capacity owing to premature buckling such that,

$$R_c = \begin{cases} 1.0 & for\ \frac{b}{t} < \lambda_{pl} \\ 0.8 + \frac{(b/t) - \lambda_{pl}}{\lambda_r - \lambda_{pl}} & for\ \lambda_{pl} \le \frac{b}{t} < \lambda_r \\ 0.8 & for\ \frac{b}{t} \ge \lambda_r \end{cases} \qquad ...(6)$$

The compactness parameters of AISC are intended for design of beams and therefore underestimate the actual capacity of sections, to keep the design conservative. However, for the design of beam-to-column connection, these cannot be used as such. For the design of connections, the upper bound of the maximum

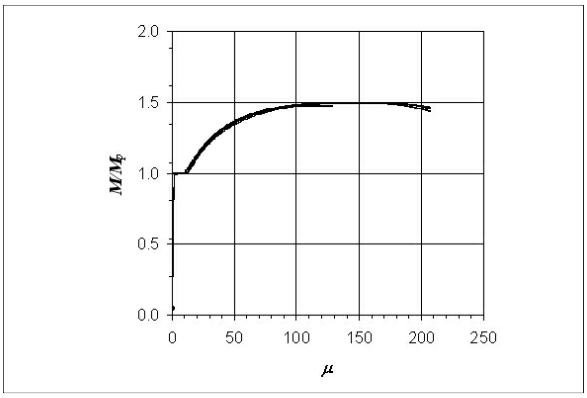

Fig 5.
Beam moment developed for f_u/f_y = 1.5 at different levels of curvature ductility imposed on the W-sections considered

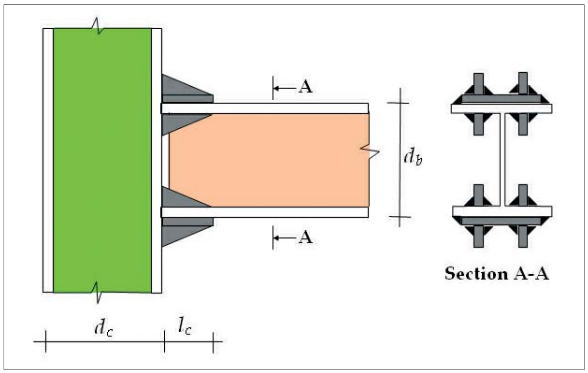

Fig 6. Details of the connection configuration used showing the locations of flange cover plates and vertical rib plates

probable section capacity is required. Detailed study is required to develop the slenderness limits for b/t, such that they give the upper bound of section capacities. Until such time, the value of compactness reduction factor R_c may be taken as 1.0.

Maximum moment demand on connection

The maximum achievable beam moment M_{pr} considering the effect of uncertainty in the estimation of yield strength (R_y), strain hardening due to post-elastic deformations (R_s) and compactness of the section (R_c) is now expressed as

$$\frac{M_{pr}}{M_{pl}} = R = R_y R_s R_c \qquad ...(7)$$

Capacity design procedure for welded beam-to column-connection

Detailed finite element studies of beam-to-column joint regions showed that Euler–Bernoulli hypothesis is not applicable near the joint regions [Goel *et al*, 1996; Arlekar and Murty, 2000]; near the joint, the beam shear is diverted towards the beam flanges resulting in stress concentration near the junction of the beam flanges and column face. Based on these results, a connection configuration consisting of outer flange cover plates and vertical rib plates (inner and outer) is used in this study (Fig 6). The presence of vertical rib plate also reduces the potential of crack initiation at the re-entrant cover-plate to column flange junction, as observed in preliminary laboratory experiments [Moitra, 2000]. Thus, the step-wise procedure for design of connections is illustrated hereunder for cover-plated and ribbed connection.

Part A: Moment demand on connections

1. Estimate the material yield strength uncertainty factor R_y from field data.
2. Based on the level of inelasticity expected in the frame, identify a target maximum plastic rotation $\theta_{pl,max}$ at the beam plastic hinge.
3. Estimate the expected curvature ductility μ in the section using Eq (5), and then the strain-hardening factor R_s from Eq (4).
4. Determine the compactness factor R_c from Eq (6).
5. Calculate maximum probable moment M_{pr} to be carried by the connection from Eq (7).
6. Assume the length l_c of the cover plate reinforcing the beam to be one-half the depth of the beam, i.e. $l_c = d_b/2$. Take the width b_{cp} of the cover plate to be such that the beam flange and the cover plate can be connected by a fillet weld, i.e.

$$b_{cp} = b_{bf} - 4t_{bf} \qquad ...(8)$$

The fillet weld between beam flange and cover plate is a critical one, and as the cover plate is expected to transfer moment corresponding to the overstrength capacity of the beam, its thickness is expected to be equal to or larger than the beam flange thickness. Thus, b_{cp} is calculated such that a fillet weld of maximum size equal to twice the thickness of beam flange can be deposited along each edge of the cover plate to connect it to the beam flange in the direction of the beam span.

7. Calculate the length L_o of the shear-link in the beam from Eq (9), assuming that plastic hinges are located at either end of the beam, at a distance of $d_b/2$ from the ends of the connection reinforcement regions on the ends of the beam (Fig 1).

$$L_0 = L - \left(\frac{d_{c1}}{2} + l_c + \frac{d_b}{2} + \frac{d_b}{2} + l_c + \frac{d_{c2}}{2}\right) \qquad ...(9)$$

8. Calculate the probable maximum shear force V_{pr} on the connection, from

$$V_{pr} = 2M_{pr}/L_0 \qquad ...(10)$$

9. Calculate the vertical force V_d and the horizontal pull force T_d on the top half of the connection, from

$$V_d = \frac{V_{pr}}{2}, \text{ and} \qquad ...(11)$$

$$T_d = \frac{M_{pr}}{d_b} + \frac{V_{pr}}{2} \qquad ...(12)$$

The second term in Eq (12) is due to the moment amplification, which is the consequence of shifting the plastic hinge away from the column face into the beam span beyond the end of connection reinforcement region. These forces are calculated using the *Improved Truss Model* [Arlekar and Murty, 2004].

Part B: Design of cover plates

10. Calculate tension T_{wcp} and shear V_{wcp} in fillet weld between beam flange and cover plate from

$$T_{wcp} = V_d, \text{ and} \qquad ...(13)$$

$$V_{wcp} = T_d - T_f \qquad ...(14)$$

where $T_f = f_y b_{bf} t_{bf}$ is the capacity of beam flange using minimum specified strength of the beam steel. Here, it is assumed that the minimum specified strength of welds is higher than that of the steel in the connection elements (beam flange, cover plates and rib plates).

11. Using data from step 6, calculate length l_{wcp} of weld between beam flange and cover plate, from

$$l_{wcp} = 2l_c + b_{cp} \qquad ...(15)$$

12. Calculate area A_{wcp} of fillet weld required to transfer combined effect of T_{wcp} and V_{wcp} from beam flange to cover plate, from

$$A_{wcp} = \sqrt{\frac{T_{wcp}^2 + 3V_{wcp}^2}{f_y^2}} \qquad ...(16)$$

and from it, the thickness t_{wcp} of this fillet weld, from

$$t_{wcp} = \frac{A_{wcp}}{l_{wcp}/\sqrt{2}} \qquad ...(17)$$

The thickness of cover plate t_{cp} is same as the thickness t_{wcp} of the fillet weld.

Part C: Design of rib plates

13. Assuming that the vertical force is transferred to the column only by two outer rib plates, calculate shear V_{rp} in each rib plate, from

$$V_{rp} = \frac{V_d}{2} \qquad ...(18)$$

14. The fillet weld between outer rib plates and cover plate is also under combined action of tension T_{whrp} and shear V_{whrp}. Calculate these forces, from

$$T_{whrp} = V_{rp}, \text{ and} \qquad ...(19)$$

$$V_{whrp} = \frac{T_d - T_f - T_{cp}}{2} \qquad ...(20)$$

where $T_{cp} = f_y b_{cp} t_{cp}$ is the capacity of cover plate using minimum specified strength, T_d is from Eq (12), and $T_f = f_y b_{bf} t_{bf}$ is the beam flange yield strength.

15. Calculate area A_{whrp} of fillet weld between cover plate and one rib plate, from

$$A_{whrp} = \sqrt{\frac{T_{whrp}^2 + 3V_{whrp}^2}{f_y^2}} \qquad ...(21)$$

16. Assume thickness t_{rp} of rib plate. Size t_{wrp} of fillet weld between rib plate and cover plate is taken to be equal t_{rp}.

Calculate the length l_{whrp} of this fillet weld, from

$$l_{whrp} = \frac{A_{whrp}}{t_{whrp} / \sqrt{2}} \quad ...(22)$$

and from this, the length l_{rp} of rib plate, from

$$l_{rp} = \frac{l_{whrp}}{2} \quad ...(23)$$

17. Calculate horizontal tension T_{rp} for each rib plate, from

$$T_{rp} = \frac{T_d - T_f - T_{cp}}{2} \quad ...(24)$$

18. Calculate area A_{rp} of each rib plate required to transfer combined tension and shear to column, from

$$A_{rp} = \sqrt{\frac{T_{rp}^2 + 3V_{rp}^2}{f_y^2}} \quad ...(25)$$

19. Assume a thickness t_{rp} of rib plate, and calculate its height h_{rp}, from

$$h_{rp} = \frac{A_{rp}}{t_{rp}} \quad ...(26)$$

Part E: Check for moment amplification and shear

20. For the actual dimensions of the cover plate and vertical rib plates provided, calculate the tensile and shear capacities, from

$$T^*_{cp} = f_y \, b_{cp} \, t_{cp} \quad ...(27)$$

$$T^*_{rp} = f_y \, b_{rp} \, t_{rp} \text{ and} \quad ...(28)$$

$$V^*_{rp} = \frac{f_y}{\sqrt{3}} b_{rp} \, t_{rp} \quad ...(29)$$

21. Ensure that the connection moment capacity (based on the minimum yield specified strength f_y of steel) to resist the external moment is more than the moment demand on it (based on actual yield strength $R_y f_y$ of steel), including the effect of moment amplification, using

$$\left(2T^*_{rp} + T^*_{cp} + T_f\right)\left(d_b + t_{cp}\right) \le M_{pr} + V_{pr}\left(l_c + l_t\right) \quad ...(30)$$

22. Ensure that the connection shear capacity (based on the minimum yield specified strength f_y of steel) to resist the external shear is more than the shear demand on it (based on actual yield strength $R_y f_y$ of steel), using

$$4V^*_{rp} \ge V_{pr} \quad ...(31)$$

In Eqs (30) and (31), the inner rib plates are not considered in estimating the moment and shear capacity of the connection. Using the inner rib plates will increase and thus, this is a conservative approach. Furthermore, in Eq (31), the shear capacity of cover plates and beam flanges is not considered, this also results a conservative estimate of the shear resistance of the connection.

Connections are designed for forces resulting from the *maximum probable* beam capacity. Here, the strength of connection elements is calculated using only the *expected minimum specified* yield strength f_y of steel.

Concluding remarks

This paper reviews three modern codes for seismic design of connections, namely the New Zealand, American and Euro codes. All three code procedures philosophically agree with each other; each provides a different method for estimating the maximum probable bending moment. One area where all codes are silent is in converting the plastic rotation requirement at the connection into quantitative shear and moment demands on the connection elements. This paper proposes a formal quantitative procedure for estimating the design forces on the connections. This procedure allows the designer to fix the desired level of inelastic deformation at the beam-to-column joint based on the rotational demand on the beam. The overstrength capacity of beam is included and the associated shear-moment interaction curves are developed for the beam. These shear-moment interaction curves are idealised with linear segments. This idealised curve is used to determine the shear demand for which the beam-to-column connection is to be designed. Thus, the maximum probable beam bending moment M_{pr} for which the beam-to-column connection is designed, is obtained by applying the uncertainty factor (R_y), strain-hardening factor (R_s), and compactness factor (R_c) to the idealised plastic moment capacity M_{pl}. A step-wise procedure for proportioning the connection elements of various connection configurations is also included in the paper.

The inclusion of compactness factor (R_c) opens avenues for further research on improvement of the calculation of maximum probable moment on the connection. In its current form, the compactness factor (R_c), based on slenderness limits for local buckling of beam flange (originally derived for design of beams), needs to be improved to account for upper bound strength of the beam to make the connection design conservative, in addition to accounting for the lateral torsional buckling of the beam and local buckling of beam web.

REFERENCES

- AISC, (1989), *Manual of steel construction allowable stress design– ninth edition*, American Institute of Steel Construction, Inc., USA, 1989
- AISC, (1994), *Metric load and resistant factor design specification for structural steel buildings*, American Institute of Steel Construction, Inc., Illinois, USA, 1994
- AISC, (1997), *Seismic provisions for structural steel buildings*, American Institute of Steel Construction, Inc., Illinois, USA, 1997
- Arlekar, J. N., and Murty, C. V. R.: (2004), 'Improved truss model for design of welded steel MRF connections', *J. Struct. Engng*, ASCE, **130**/. 3, p 498-510, March 2004
- Basler, K.: (1962), 'Strength of plate girders under combined bending and shear,' *J. Struct. Div., Proc. Am. Soc. .Civ. Eng.*, **87**/ST7, p 181-197, 1962
- Cooper,P. B., Galambos,T. V. and Ravindra,M. K.: (1978), ''LRFD criteria for plate girders', *J. Struct. Div., Proc. Am. Soc. Civ. Eng.*, **104**/ST9, p 1389-1407, 1978
- DD ENV 1993 (Part 1), (1992), *Eurocode 3: Design of Steel Structures, Part 1.1 General rules and rules for buildings*, DD ENV 1993-1-1:1992, British Standards Institution, London, UK
- Englehardt,M. D. and Sabol,T. A.: (1998), 'Reinforcing of steel moment connections with cover plates: Benefits and limitations', *Eng. Struct.*, **20**/4-6, p 510-520, 1998
- FEMA 267, (1995), 'Interim guidelines: Evaluation, repair, modification and design of welded steel moment frame structures', *Report No.SAC-95-02,* SAC Joint Venture, CA, USA, 1995
- FEMA 267B, (1999), 'Interim guidelines advisory N. 2, Supplement to FEMA 267,' *Report No. SAC-99-01*, SAC Joint Venture, CA, USA, 1999
- FEMA 350, (2000), 'Recommended seismic design criteria for new steel moment-frame buildings', SAC Joint Venture, CA, USA, 2000
- FEMA XXX, (1999), 'Seismic design criteria for new moment-resisting steel frame construction - 50% draft', *Report No.SAC-XX-XX-XX,* SAC Joint Venture, CA, USA, 1999
- Goel, S. C., Lee,H. K. and Stojadinovic,B.: (2000), 'Design of welded steel moment connections using truss analogy', *AISC Engng J.*, First Quarter, p 31-40
- Goel, S. C., Stojadinovic, B. and Lee,H. K.: (1996), 'A New Look at Steel Moment Connections', *Report No. UMCEE 96-19*, Department of Civil and Environmental Engineering, The University of Michigan College of Engineering, 1996
- Hodge, P. G. Jr., and Brooklyn, I. N.: (1957), 'Interaction curves for shear and bending of plastic beams', *J. App. Mech., Am. Soc. Mech. Eng.*, Paper No.57, APM-19., p 453-456, 1957
- Malley, J. O. and Frank, K.: (2000), 'Materials and fracture investigations in the FEMA/SAC PHASE 2 steel project', *12th World Conf. Earthquake Engineering, Paper ID. 2544*, New Zealand, 2000
- Miller, D. K.: (1998), 'Lessons learnt from the Northridge earthquake,' *Engng Struct.*, **20**/4-6, p 249-260, Elsevier Science Ltd, 1998
- Moitra, A.: (2000), 'Experimental investigation of cyclic response of reinforced connections for earthquake resistant steel MRFs', *Master of Technology Thesis,* Department of Civil Engineering, Indian Institute of Technology Kanpur, India, 2000
- Murty, C. V. R. and Hall, J. F.: (1994), 'Earthquake collapse analysis of steel frames', *Earthquake Engineering and Structural Dynamics*, **23**, p 1199-1218, 1994
- NZS 3404, (1997), *Steel Structures Standard - Parts 1 and 2*, Standards New Zealand, Wellington, NZ
- preEN 1998 (Part 1), (2003), Eurocode 8: *Design of Structures for Earthquake Resistance – Part 1: General rules, seismic actions, and rules for buildings,* prENV 1998-1:2003, European Committee for Standardization, Brussels, December 2003
- UBC, (1994), Uniform Building Code, 1994 Edition, *Int. Conf. Building Officials*, CA, USA, 1994