

Moment Connection for force BEYOND Material Overstrength



- Ribs are the triangular protrusions, joined in a C where adjacent to webs.
- Ribs and Coverplates are welded to face of column by complete penetration welds (in green at left).
- No part of the beam section proper is welded to the face of column, except through connection ribs and coverplates.

Background and critical comments



- It is my contempt that the dynamic loading coming from wind or earthquake loads may very well produce forces at the connection well beyond of those surmised limited by the mere material overstrength factor due to inertial performance of the structure, particularly the ramlike IMPACT behavior of the still unbuckled propelled flange (in alike manner to an arrow that doesn't buckle while impacting a target; a thin wire 30 cm long would go through your hand if thrown as a bullet irrespective of its easy buckling when you press its tips closer statically). Typical design for forces downscaled even to the 5th part from those coming from elastic response warrants that, for the initial only moderately damped and plastified structure, solicitation at over material overstrength levels -source demand at over mere material overstrength level- may very well warranted from such reservoir. Our beam dies killing.
- These peak forces need to be acknowledged for a design where no damage nor relaxation is expected at the assumed rigid joints under strong dynamic forces.
- Additional Stress Concentration Factors seem not required, since the stress raisers are present for both static and dynamic analysis situations. Contrarily, it is the pretension of designs that attempt to keep hinges in the beams to ensure that the beam remains in elastic state from the hinge to the column, connection included, and it would be being inconsistent with such analysis to use for the evaluation of the forces the inelastic response spectrum charts.
- Although it may be argued with reason that the prevailing understanding of the use of inelastic response spectrum is that it produces reasonable forces to ensure structural safety anywhere when complying with the design standards, it is my contention that the Northridge's failures show precisely the contrary, not anywhere the forces so determined are safe, and the highest forces at the connections will be more consistent with some to me unknown nonlinear aerospace level type of analysis performed upon the real structure able to reckon the existence of impact forces above the (input data and curtailing of forces through pastification of hinges) material strengths.
- In sum, for the first higher wind or earthquake thrust I understand some impact factor may be needed (when loading rate and inertial solicitation goes as high) to properly portrait the forces at the connection, and is introduced here for use in the simple strut and tie approach.
- Strut and Tie forces and those at welds are calculated here as in the Goel, Lee, Stojadinovic article in AISC EJ Vol. 37 1 without any critical judgement, then I going more explicit in specific determination of welds. As in the article, it is accepted the analysis shows plastic hinges appearing on the beams between 0 and 1 beam depths from the face of the column and that this causes the procedure being conservative.
- Weld backing bars are renowned stress raisers which permanence after welding is contrary to good practice and MUST ALL be removed.
- It may very well turn out from developments from my reasoning here that at the present status of knowledge of material behavior, analysis and construction practice, the intact survivability as a class property of this and another kinds of rigid moment connections during strong earthquakes can't be economically -if not architecturally or technically- warranted without further developments.
- Exemplifying, even if the structure was cast of a mold, it would inertially be subjecting itself (locally, these are no planets being suck by a black hole) to bigger forces than its natural material strength, so it may turn not to have nothing to do even with details within the class or their calculus. That these rupturing levels of forces are being attained at the joints at strong earthquakes should be subject to its proof in realistic full scale simulation, but the the widescale joint failures in strong earthquakes suggest this can be a very likely possibility.
- We use to shake brusquely a thin dry wooden stick to break it by one whiplike stroke; that the forces have gone at the rupture point instantaneously but

efficiently higher than material overstrength is factually proven by the rupture at the point; AND if some glued aluminium foil sleeve or pipe was there to help to stand the thrust, it would have (if useful to such purpose) had to support forces over the strength of the (otherwise broken) force imparting part (the wood stick), while at the time the included stick would be being reinforced by the sleeve and only in the ongoing process of closing to get broken. The resisting part (conservatively the aluminium sleeve only) may be analyzed for such over_material_strength force if known.

- So here you have the structural calculus equivalent of the existence of transluminal particles. Are there forces beyond material strength in real structures subject to strong earthquake forces? My bet is yes.
- Taken otherwise, the assumption of inelastic behavior -as presently portraited first in codes and then complying programs- for such strong earthquakes not only seems to be meaning some degree of generic structural and non-structural damage in the building, but specifically the high likelihood of maybe unacceptable nonrecoverable plastic structural damage or rupture at these most stressed points, the rigid joints.
- This also conveys the interest of for economy using the lesser number of the expensive rigid joints (i.e, use a hinge-fuses approach), but this also may not be possible at the level of forces and displacements imposed by a strong earthquake, if not by percentual imposition of lateral force for the rigid frame in the relevant code. Reliability against lateral forces also would ask for a high number of the expensive rigid joints.
- Purportedly, the technology should be available in the areospace industry, from which we may be in need of some not forthcoming technology transfer, lest we be like gods.



F_y := 360·MPa

specified yield strength

F_{ym} := 520·MPa

average (from mill data) yield strength

ϕ := 0.9

for yield

b_f := 30·cm

t_f := 2.5·cm

d := 80·cm

t_w := 1·cm

double tee beam dimensions

L_b := 8·m

α_h := 0.5

- fraction of flange force directly passed to column by coverplate, assumed (the rest is passed by ribs)
- between 0.3 and 0.7 give reasonable values, it seems

Impact_Factor := 1

must be always >=1 and should be > 1 if forces at connection from analysis with inelastic response spectra or charts

You only will be beyond safety warranted by interim guidelines if you enter an Impact Factor bigger than 1



$$\beta := 0.95 \cdot \left(1.1 \cdot \frac{F_{ym}}{F_y} \right)$$

β = 1.51

$$Z_b := b_f \cdot \frac{d^2}{4} - (b_f - t_w) \cdot \frac{(d - 2 \cdot t_f)^2}{4}$$

Z_b = 7218.75 cm³

M_{pri} := Impact_Factor · β · Z_b · F_y

M_{pri} = 399.99 m·ton

$$V_{ci} := \frac{2 \cdot M_{pri}}{L_b - 2 \cdot d}$$

V_{ci} = 125 ton

$$T := \frac{M_{\text{pri}}}{d - t_f} \quad C := T$$

$$T = 516.12 \text{ ton} \quad C = 516.12 \text{ ton}$$

Note that the dynamic push and pull at flanges imparted by the floors through beams can get as big as shown, being so unlikely that the general analysis (position of hinges assumed resulting not at columns, for example) needs not to be modified seeing such high if momentaneous at flange connection point forces.

$$A_h := \frac{\alpha_h \cdot (T + V_{ci})}{\phi \cdot F_y}$$

$$A_{v1} := 124 \cdot \text{cm}^2 \quad \text{unwarranted guess}$$

Given

$$\sqrt{\left[\frac{(1 - \alpha_h) \cdot (T + V_{ci})}{A_{v1}} \right]^2 + 3 \cdot \left(\frac{0.5 \cdot V_{ci}}{A_{v1}} \right)^2} = \phi \cdot F_y$$

once this condition is met, **IF** weld strength is higher than that of base metal **AND** complete penetration welds are used for both coverplate to column and rib to column, no further check of the strength at the column interface is required

$$A_v := \text{Find}(A_{v1})$$

$$A_h = 97.03 \text{ cm}^2 \quad \text{required section of every top and bottom coverplate}$$

$$A_v = 102.41 \text{ cm}^2 \quad A_v \text{ is total for upper and bottom ribs in every flange}$$

$$N_f := \frac{1}{2} \cdot (1 + \alpha_h) \cdot (T + V_{ci}) \quad V_f := \frac{V_{ci}}{4}$$

$$N_f = 480.84 \text{ ton}$$

along flange

forces to be taken by welds at the coverplate-flange interface

$$V_f = 31.25 \text{ ton}$$

transversally

(Half) Rib to coverplate required weld strength

$$\frac{A_v}{2} \cdot F_y = 187.97 \text{ ton}$$

(Half) Rib to column required weld strength

$$\frac{A_v}{2} \cdot F_y = 187.97 \text{ ton}$$

preferably complete penetration, is so won't require sizing the fillet weld

$$N_c := T + V_{ci}$$

$$V_c := \frac{V_{ci}}{2}$$

$$N_c = 641.12 \text{ ton}$$

normal to face of column

$$V_c = 62.5 \text{ ton}$$

vertically

Forces to be taken by welds at column interface for every flange plus respective top and bottom rib connection



Required Plate and Ribs area

$$A_h = 97.03 \text{ cm}^2$$

required section of every top and bottom coverplate

$$A_v = 102.41 \text{ cm}^2$$

A_v is total for upper and bottom ribs in every flange

$$\frac{A_v}{2} = 51.2 \text{ cm}^2$$

each top OR bottom rib

Some forces for weld design

$N_f = 480.84 \text{ ton}$

force along flange

forces to be taken by welds at the coverplate-flange interface

$V_f = 31.25 \text{ ton}$

vertical force

$\frac{A_v}{2} \cdot F_y = 187.97 \text{ ton}$

(Half) Rib to coverplate required weld strength, to size fillet weld

$N_c = 641.12 \text{ ton}$

normal to face of column

- **IF** weld strength is higher than that of base metal **AND** complete penetration welds are used for both coverplate to column and rib to column, no further check of the strength at the column interface is required
- otherwise dimension the roughly cross shaped weld at each flange-column interface with fillet welds for the forces at left
- it will be assumed that ribs and coverplates are all welded with complete penetration welds at its interface with the column, so as per above we don't require this check by already implied

$V_c = 62.5 \text{ ton}$

vertically

Section Dimensions for Coverplates and Ribs

In the detail, beams are not welded at all to the column. Connection depends exclusively upon coverplates and gusset ribs. The inner ribs may get substituted by 1 or 2 properly shaped shearplates, at least one prewelded to the column. Bottom Coverplate usually will be wider than the flange, while top coverplate will be less wide than the flange, in both cases to allow to weld downwards. Since Ribs for our case would be too thick, we put pairs of them, atop and below each flange.

Ribs

$N_{ribs} := 2$

1 or 2

$t_{rib} := 25 \text{ mm}$

$$h_{rib} := \frac{\frac{A_v}{2 \cdot N_{ribs}}}{t_{rib}}$$

$h_{rib} = 10.24 \text{ cm}$

Dimension of Ribs

$N_{ribs} = 2$

A pair of ribs can still be beveled for a complete penetration weld of the rib against the column

atop and under every flange

Coverplates

We will make top coverplate 2 flange thickness less wide than the flange

$b_{top_plate} := b_f - 2 \cdot t_f$

$b_{top_plate} = 25 \text{ cm}$

$$t_{\text{top_plate}} := \frac{A_h}{b_{\text{top_plate}}}$$

$$t_{\text{top_plate}} = 38.81 \text{ mm}$$

quite thick for 1 single plate
and our taste

We will make top coverplate 2 flange thickness more wide than the flange

$$b_{\text{bottom_plate}} := b_f + 2 \cdot t_f$$

$$b_{\text{bottom_plate}} = 35 \text{ cm}$$

$$t_{\text{bottom_plate}} := \frac{A_h}{b_{\text{bottom_plate}}}$$

$$t_{\text{bottom_plate}} = 27.72 \text{ mm}$$

We have drawn in the sketch the two plates like those at bottom

Dimension Fillet Welds

$$F_{\text{EXX}} := 70 \cdot \text{ksi}$$

$$\phi_{\text{weld}} := 0.75$$



General Theory for Fillet Welds Strength at Throat (particularized for 90° between joined planes)

From the tensors transformation between the throat plane and stresses at one of the joined faces...

$$\sigma(n, t_I) := \frac{t_I + n}{\sqrt{2}}$$

$$\tau_I(n, t_I) := \frac{t_I - n}{\sqrt{2}}$$

$$\tau_{II}(t_{II}) := t_{II}$$

σ normal to the throat plane
 n normal to one joined face

perpendicular to the edge
in respective plane

parallel to the edge
in respective plane

greek letters represent stresses referred to (at) the throat plane and english alphabet letters stresses at the faces. Factored stresses at the faces n , t_I y t_{II} are determined according to the

case and the theory of strength of materials but **considering at the joined face only one surface of width equal to throat's width**, and from such, stresses at the throat plane are calculated.. Then for the weld to be safe at the throat it must be satisfied the experimental relationship...

$$\sigma_{\text{co}}(n, t_I, t_{II}) := \sqrt{\sigma(n, t_I)^2 + 1.8 \cdot (\tau_I(n, t_I)^2 + \tau_{II}(t_{II})^2)} \quad \sigma_{\text{co}}(n, t_I, t_{II}) \leq \sigma_u$$

With this theory can be solved every strength at throat case for 90 deg fillet welds; in some cases some reductions to efficiency are practiced with experimental basis. Furthermore, if the strength of the weld metal exceeds that of the base metal, the strength at the faces is automatically met and only this check at the throat plane needs to be checked.

In the spanish code, once the forces are properly factored (not to a level much dissimilar for steel than in the US) no weld strength reducing factor is used, so the US codes seem to go more conservative for welds, as much as the weld strength reducing factor ratio indicates.

Limit strength at the frontal (fillet) welds between coverplate and flange

We have brought about the general theory of 90 deg fillet welds because nor the US or Spanish codes treat this case explicitly

$\theta := \operatorname{atan}\left(\frac{V_f}{N_f}\right)$

$\theta = 3.72 \text{ deg}$

$n := 310 \cdot \text{MPa}$

unwarranted guess for maximum stress at joined face (but counting only throat width)

$t_I(n) := n \cdot \tan(\theta)$

t_{II} is zero for the frontal weld

$\sigma(n) := \frac{t_I(n) + n}{\sqrt{2}}$

$\tau_I(n) := \frac{t_I(n) - n}{\sqrt{2}}$

$n_I(n) := n$

Given

$\sqrt{\sigma(n)^2 + 1.8 \cdot \left(\tau_I(n)\right)^2} \leq F_y$

$n := \operatorname{Maximize}\left(n_I, n\right)$

$n = 309.39 \text{ MPa}$

$t_I(n) = 20.11 \text{ MPa}$

$\phi_{\text{weld}} \cdot n = 232.04 \text{ MPa}$

Now we can judge the load the frontal weld can bear comparing its **main** component with $\phi_{\text{weld}} \cdot n$

For example, one weld of throat

$$a := \frac{t_f - 2 \cdot \text{mm}}{\sqrt{2}} \quad a = 16.26 \text{ mm}$$

or choose (lesser) yours $a := 15 \cdot \text{mm}$ (we do)

$$b_f - 2 \cdot t_f - 2 \cdot a = 22 \text{ cm} \quad \text{typical length}$$

would be able to take vertical component factored final load of

$$N_{\text{frontal_weld_capacity}} := a \cdot (b_f - 2 \cdot t_f - 2 \cdot a) \cdot (\phi_{\text{weld}} \cdot n)$$

$$N_{\text{frontal_weld_capacity}} = 78.08 \text{ ton}$$

Since the main component is

$$N_f = 480.84 \text{ ton} \quad \text{PerOne}_{\text{forSides}} := \frac{N_f - N_{\text{frontal_weld_capacity}}}{N_f}$$

$$\text{PerOne}_{\text{forSides}} = 0.84$$

Strength according to Appendix J of LRFD for lateral fillet welds with loads at an angle

$$F_w(\theta) := 0.6 \cdot F_{\text{EXX}} \cdot (1 + 0.5 \cdot \sin(\theta)^{1.5})$$

on throats of welds where force forms an angle θ to weld alignment

Lateral fillet welds flange to coverplate

We will accept rigid body transmission of the forces and so shear forces acting through shear center enanywhere. Only then the effect of (the minor) vertical force causes some "frontalization" of the attack angle to the weld, and from LRFD appendix J...

$$\theta := \text{atan}\left(\frac{V_f}{N_f}\right) \quad \theta = 3.72 \text{ deg}$$

$$\text{Force} := \text{PerOne}_{\text{forSides}} \cdot \sqrt{V_f^2 + N_f^2} \quad F_w(\theta) = 291.98 \text{ MPa}$$

$$\text{Required_throat_area} := \frac{\text{Force}}{\phi_{\text{weld}} \cdot F_w(\theta)} \quad \text{Required_plan_area_of_weld} := \text{Required_throat_area} \cdot \sqrt{2}$$

$$\text{Length}_{\text{per_side}} := \frac{\frac{\text{Required_throat_area}}{a}}{2} + a \qquad \text{Length}_{\text{per_side}} = 0.62 \text{ m} \qquad \text{Length}_{\text{per_side}} \leq 0.8 \cdot d = 1 \qquad \text{must be 1 for OK}$$

$$Z := \text{ceil}\left(\frac{a \cdot \sqrt{2}}{\frac{\text{in}}{16}}\right) \cdot \frac{\text{in}}{16} \qquad \text{Frontal_weld}_{\text{length}} := b_f - 2 \cdot t_f - 2 \cdot a$$



Flange to coverplate fillet weld

$a = 15 \text{ mm}$	throat	$\text{Length}_{\text{per_side}} = 61.75 \text{ cm}$	$\text{Frontal_weld}_{\text{length}} = 22 \text{ cm}$
$Z = 22.22 \text{ mm}$	$Z = 14 \frac{\text{in}}{16}$ size	$\text{Length}_{\text{per_side}} \leq 0.8 \cdot d = 1$	must be 1 for OK



For the (Half) Rib to coverplate weld

$$\theta := \text{atan}\left(\frac{\frac{V_{ci}}{4}}{\frac{A_v}{2} \cdot F_y}\right) \qquad \theta = 9.44 \text{ deg}$$

$$\text{Force} := \frac{\sqrt{\left(\frac{V_{ci}}{4}\right)^2 + \left(\frac{A_v}{2} \cdot F_y\right)^2}}{N_{\text{ribs}}} \qquad \text{here we account for the fact of the ribs being possibly 2 atop or under every flange}$$

$$F_w(\theta) = 299.2 \text{ MPa}$$

$$\text{Required_throat_area} := \frac{\text{Force}}{\phi_{\text{weld}} \cdot F_w(\theta)}$$

$$\text{Required_plan_area_of_weld} := \text{Required_throat_area} \cdot \sqrt{2}$$

$$a_{\text{rib}} := \frac{t_{\text{rib}} - 2 \cdot \text{mm}}{\sqrt{2}}$$

substitute t of coverplate where t_{rib} if lesser

$$a_{\text{rib}} = 16.26 \text{ mm}$$

or enter yours throat

$$a_{\text{rib}} := 8 \cdot \text{mm}$$

$$L_{\text{rib}} := \frac{\text{Required_throat_area}}{2 \cdot a_{\text{rib}}}$$

$$L_{\text{rib}} = 26.02 \text{ cm}$$

$$Z_{\text{rib}} := \text{ceil} \left(\frac{a_{\text{rib}} \cdot \sqrt{2}}{\frac{\text{in}}{16}} \right) \cdot \frac{\text{in}}{16}$$



Rib to coverplate fillet weld

2 x

L_{rib} = 26.02 cm

sides, we will weld the tip as well

N_{ribs} = 2

a_{rib} = 8 mm

throat

Z_{rib} = 12.7 mm

Z_{rib} = 8 $\frac{\text{in}}{16}$

size

Final Comments

- Joel et al. use a single C tab and prefer not to join it to the column at its central parts.
- Even with impact factor 1 ribs can be really thick and a symmetrical weld can be preferable, even if one of them needs to be made from a single bevel.
- Any backing bar would be then interstitial and only accessible from its top side for removal (even if theoretically unlockable by downwards hammering), so it is understandable the single tab preference. This may lead to abnormal C thickness even with forces limited to mere static overstrength.
- I also have reserves about the equidistributive behavior assumed for the weld sizing be of need safe.
- From my viewpoint it is very good (more expensive) practice to weld the central part, this making the structure more reliable against dynamic forces.
- More important, it is seen that connection can go as big that other alternatives such dogbone unions may turn out to be more factible or more recommendable, even to the expense of less efficient use of material. Any dogbone alternative is also more tolerant from the architectural standpoint. Anyway the force over material overstrength remain a possibility but somewhat downscaled by the lesser plastic moment at hinges.
- The whole subject of the rigid unions in rigid frames having to support strong earthquakes remains open, as proven by the caution (more than 5 years) in giving binding formal recommendations by the conversant researching teams of the most technologically advanced countries in the world. This also may be good for able responsible designers, since they won't be tied to limiting prequalified solutions unsuitable to their particular design concept. The caution is understandable since it seems not enough number of full size subassemblies seems to be being tested at the loading rates and force levels imparted by strong earthquakes. So theorizations can't be put to proper test.
- The legal body should also be advised that no closed form technical solution to the rigid frame joint problem has been available, nor it is today. If citizens are worried enough by this problem they must understand that a betterment of the design of joints to some degree of reliability can only come between other things but through insistent testing of fully real size structures under the difficult or at least costly process of mimicking realistic strong earthquake forces, this requiring important investments in research. The number of test should be as high as to be possible to extract statistical meaningful response from the deterministic applied forces.