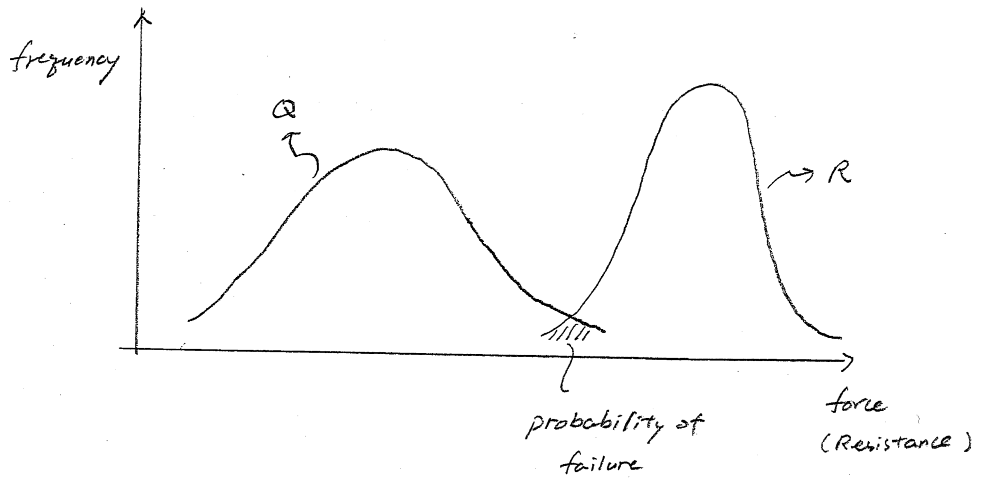
Steel Design

<u>Design codes</u> : KBC 2016 (load resistance factor design)
AISC- LRFD 2005

Design concept for safety

$LQ \leq \phi R$



Q = load
L = load factor r
ϕ = strength reduction factor
R = Resistance

Load Combinations

$1.4D$
$1.2D + 1.6L + 0.5(L_r\ or\ S)$
$1.2D + 1.0L \pm 1.3W + 0.5(L_r\ or\ S)$
$1.2D + 1.0L \pm 1.0E$
$0.9D \pm (1.0E\ or\ 1.3W)$

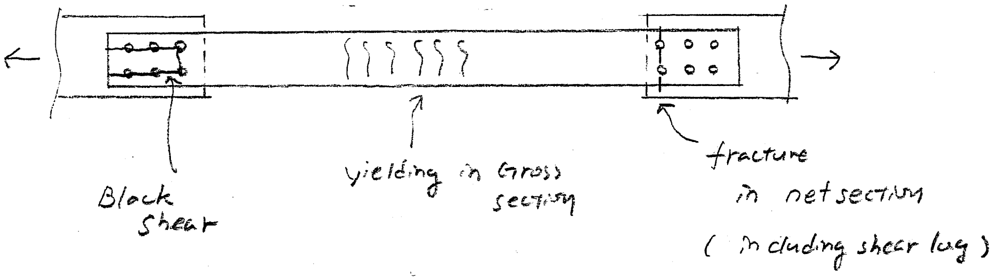
* $1.0L \rightarrow 0.5L$ for $w < 5.0\,kN/m^2$

Strength Reduction Factor

ϕ = 0.90 for flexure, Tension
0.90 for shear
0.90 for compression
0.75 for connection design

General Requirements for Design of Steel Members

1. Tension Member



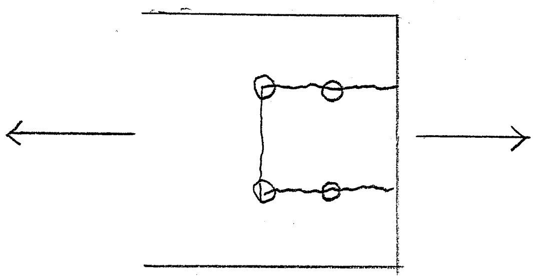
Limit States

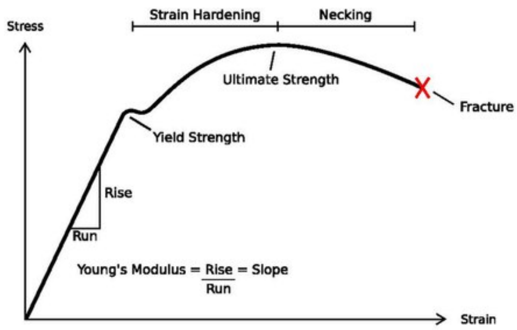
a. yielding in gross section : $\phi P_n = \phi A_g F_y$

b. Fracture in Net section : $\phi P_n = \phi A_e F_u$

c. Block shear : $\phi P_n =$ yielding in tension + fracture in shear

or fracture in tension + yielding in shear





<u>2. Compression member (H section or wide flange section)</u>

<u>Limit States</u>

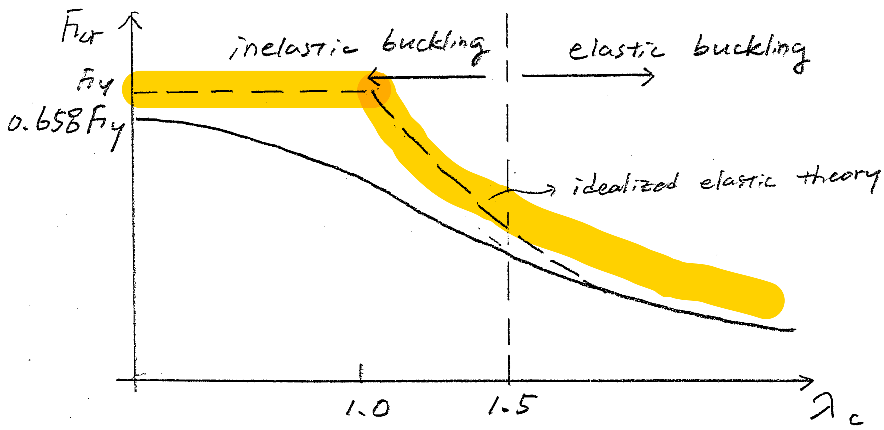
a. yielding

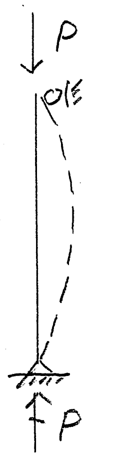
b. buckling (elastic and inelastic)

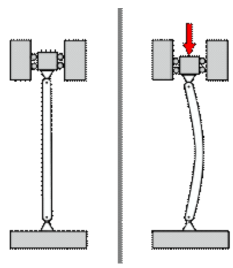
$$\lambda_c < 1.5 \qquad F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y$$

$$\lambda_c > 1.5 \qquad F_{cr} = \left(0.877/\lambda_c^2\right) F_y \;=\; = 0.877 F_e \qquad F_e = \frac{\pi^2 E}{\left(KL/r\right)^2}$$: elastic buckling load

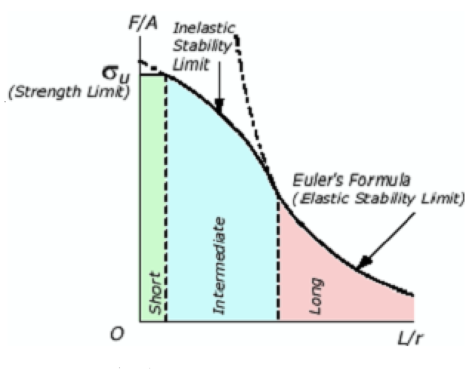
$$\phi P_n = \phi A_g F_{cr}, \quad \lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}}, \quad \lambda_c^{\,2} = \frac{F_y}{F_e}$$







$\lambda_c = 1.5$ considering the effect of residual stress



c. local buckling : for slender sections, the effective yield strength is reduced :

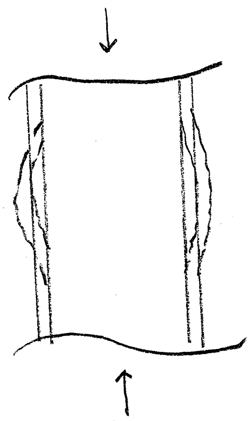
F_y is replaced with QF_y where $Q = Q_s\, Q_a$

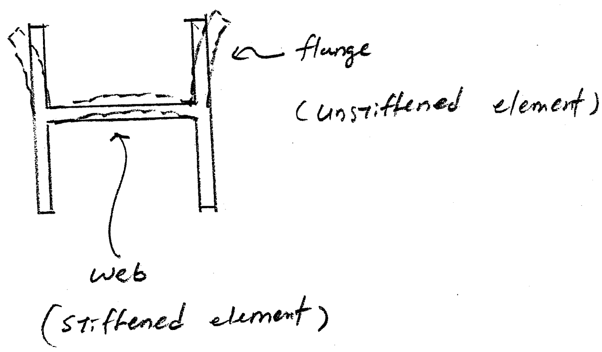
Q_s = reduction factor for unstiffened plates,

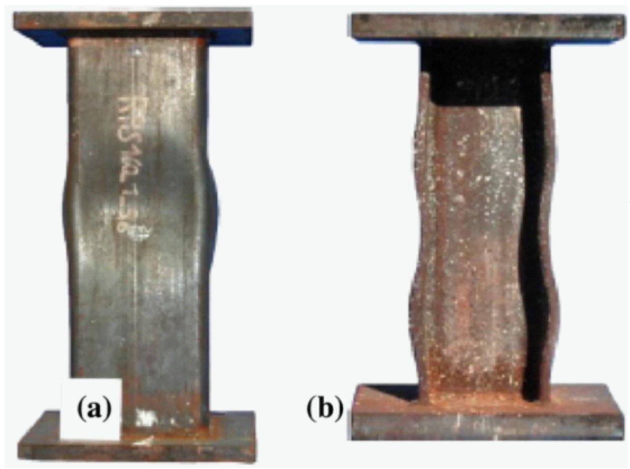
Q_a = factor for stiffened plates

Q_s , Q_a are function of b/t ratio.

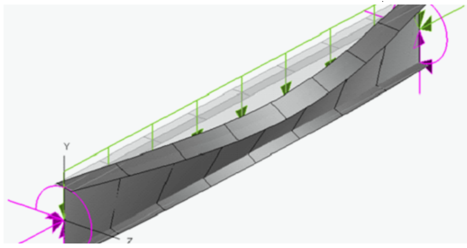
However, for most rolled sections, local buckling can be neglected.







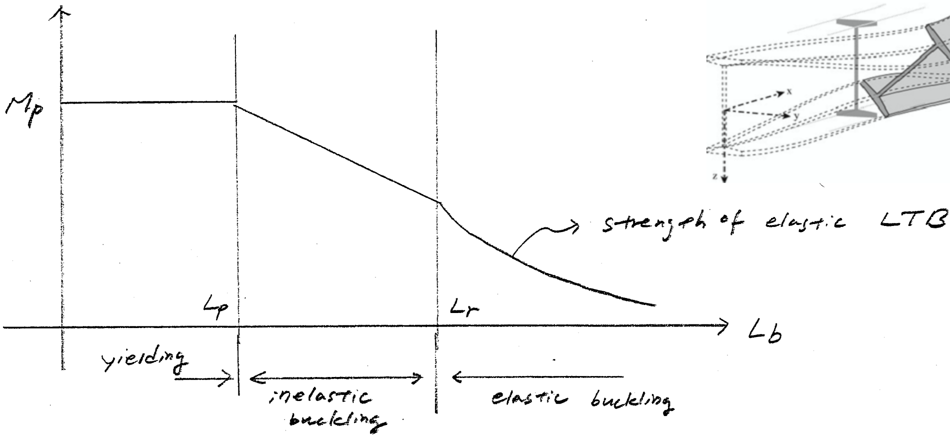
3. Flexure member **(H section or wide flange section)**

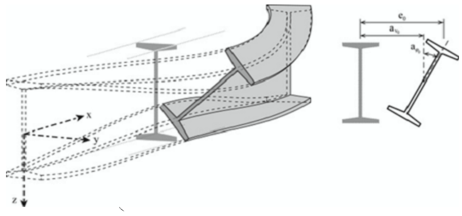


Limit States : $\phi M_n \geq M_u$ $\quad$ $M_n = \min(M_{n1}, M_{n2}, M_{n3})$

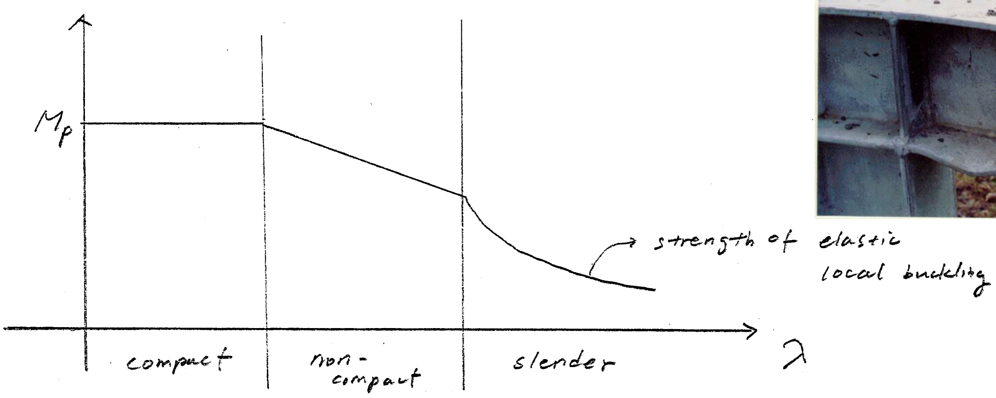
a. Flexural yielding $\quad M_{n1} = ZF_y$

b. Lateral Torsional Buckling (LTB) M_{n2} $\quad$ unsupported length = L_b





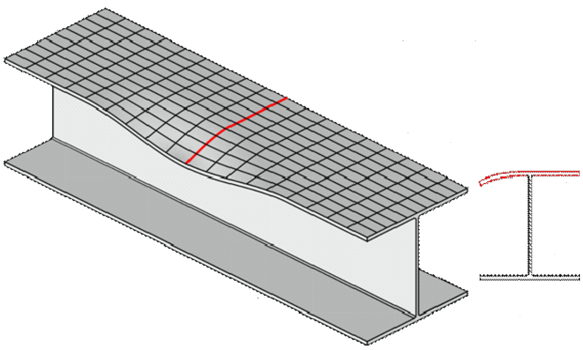
c. Flange Local Buckling M_{n3}, $\quad$ flange slenderness ratio λ = $b/2t_f$



d. web Local Buckling : M_{n1}, M_{n2}, M_{n3}, are reduced by web slenderness ratio of λ = h_c/t_w

e. Shear $\quad V_n = 0.6 F_y A_w C_v$ $\quad$ C_v = factor of web local buckling

f. Deflection

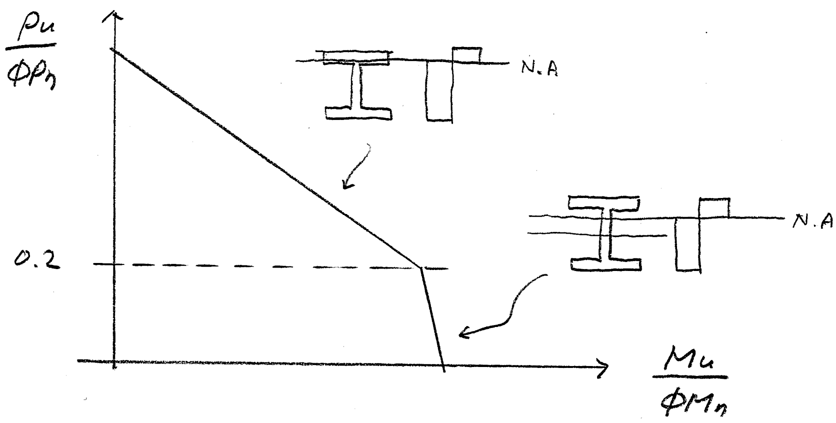


4. Beam – column (flexure + axial action)

Interaction

$$\frac{P_u}{\phi_c P_n} \geq 0.2 \qquad \frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

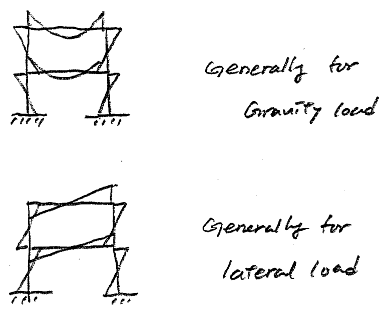
$$\frac{P_u}{\phi_c P_n} < 0.2 \qquad \frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$



$$M_u = B_1 M_{ns} + B_2 M_s$$

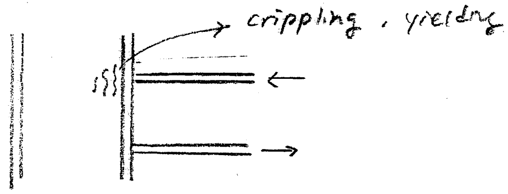
B_1 : Moment magnifier for non-sway mode

B_2 : Moment magnifier for sway mode

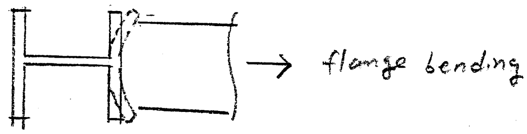


5. Details

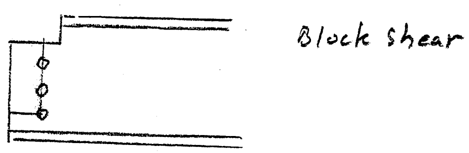
a.



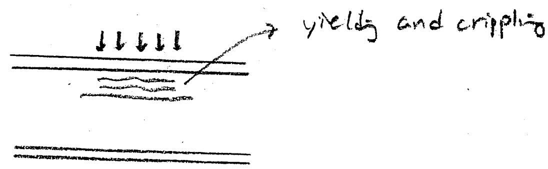
b.

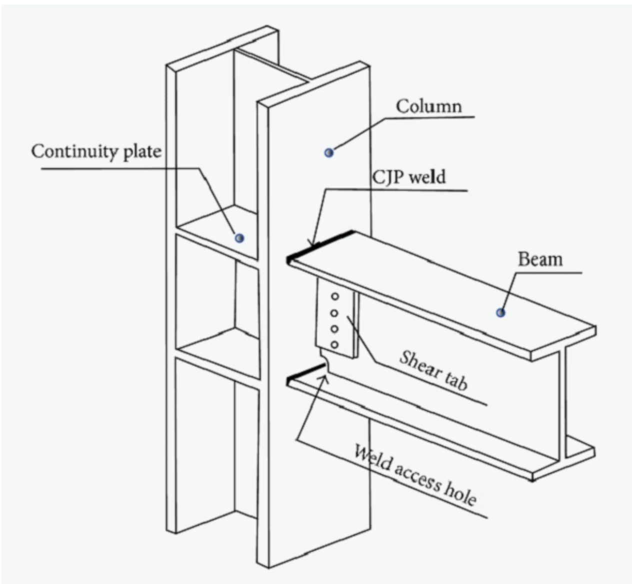


c.



d.





Earthquake Design of Steel Frame

Structural types according to the ductility capacity

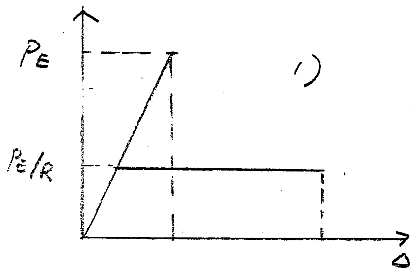
* Ordinary moment resisting frame (use lower R factor) not meeting special detailing requirements for ductile behavior

* Special moment resisting frame (use higher R factor) specially detailed to provide ductile behavior

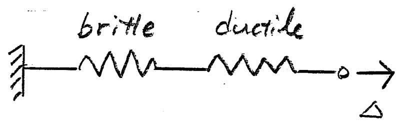
Principle of EQ Design of Steel Structure

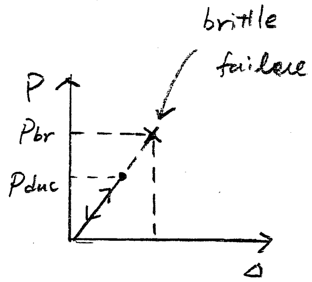
1) Ductile members such as beams and tension braces are designed for general EQ load combination under the assumption of sufficient ductility of the members.

2) Brittle members such as columns and compression braces are designed for special EQ load(using Ω_o) with increased EQ load, to prevent the possibility of brittle failure before yielding of ductile member. Yielding of ductile members governs the required strength of the structure.

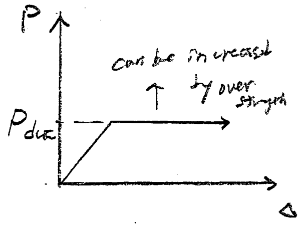


2)

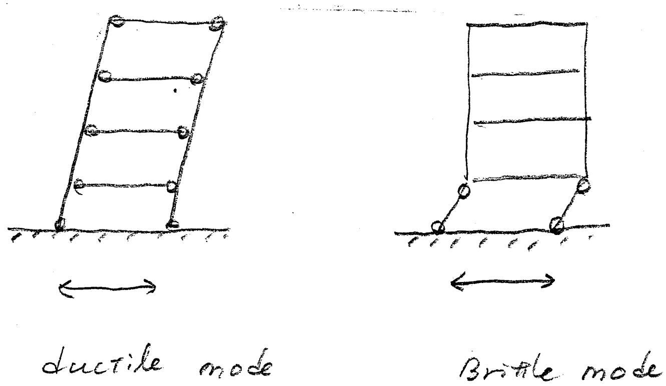




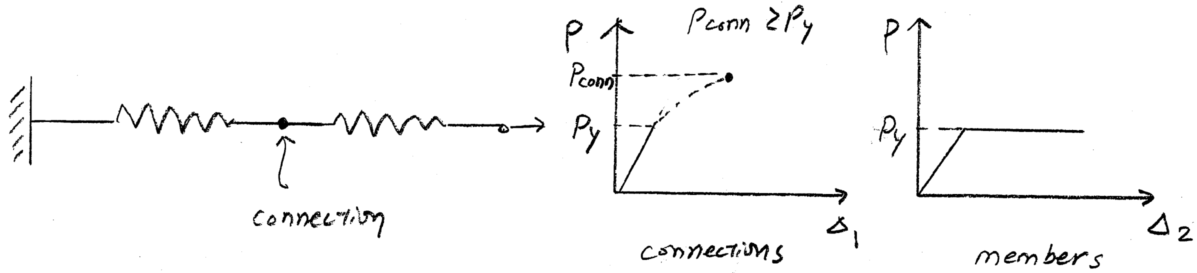
brittle member



ductile member



3) Connections and other details are designed for the connection forces required for yielding of the members to be connected. Generally, connections (welding and bolting) are not ductile.



Seismic Design of Steel Structure

KBC 2016 0713 earthquake design of steel structure (**structures with *R* >3.0**)
0714
ANSI/AISC 341-05 Seismic Provisions for Structural Steel Building

0713.1.1 scope

Structures with response modification factor R greater than 3.0

0713.4 Earthquake load combinations

$1.2D + 1.0E + f_1 L + f_2 S$ $\quad f_1 = 1.0$ for $w_2 \geq 5.0\text{kN/m}^2$

$0.9D + 1.0E$ $\quad 0.5$ otherwise

E = General EQ loading
$\Omega_0 E \pm 0.2 S_{DS} D$ = special EQ loading

0713.6 Material properties for determination of required strength

0713.6.1 Materials

SN, SHN, TMC steel (Korean Industrial Standard) should be used for intermediate frame and special frame.
SS, SM steel = general used of steel materials

<Table 0701.4.8.> Design strength of major structural steel, MPa

strength	Steel type / thickness	SS400 SM400 SN400 SMA400	SHN 400	SS4901)	SM490 SMA490	SN490	SHN490	SM520 (SM490Y)	SS540	SM570 SMA570
Fy	40mm or less	235	235	275	315	325	325	355	390	450
	40mm more 75mm less	215	235	255	295	295	325	335	-	430
	75mm more 100mm less	215	-	-	295	295	-	325	-	420
Fu	100mm less	400	400	490	490	490	490	520 (490)2)	540	570

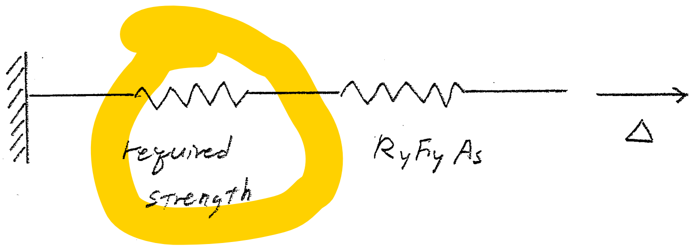
Strengh	Steelgrades / Thickness	SM490TMC	SM520TMC	SM570TMC	HSA800
Fy	≤ 80mm	315	355	450	650
Fu	≤ 80mm	490	520	570	800

note 1) SS490 : thickness ≤60mm 2) Fu of SM490 : 490MPa.

0713.6.2

The required strength of a connection and a brittle member shall be determined from the expected yield strength of the members connected : $R_y F_y$

R_y = overstrength factor of material > 1.0 (1.1~1.5)



Usually, the expected yield strength is greater than the design yield strength F_y

R_y increases for low strength steel

In the calculation of the resistance R_n, for safe design, F_y or F_u should be used without using the overstrength factor.

But, within a member, $R_Y F_y$ or $R_t F_u$ can be used.

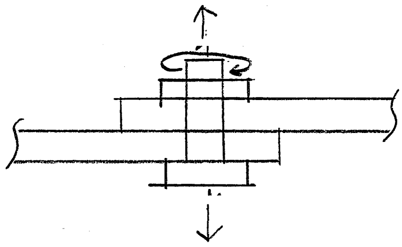
TABLE 0713.6.1 *Ry* and *Rt* Values for Different Member Types

Application		R_y	R_t
Rolled structural shapes and cold-formed sections	KS D 3503 SS400 KS D 3530 SSC400 KS D 3558 SWH400 KS D 3566 STK400, STK490 KS D 3568 SPSR400, SPSR490 KS D 3632 STKN400, STKN490	1.3	1.2
	KS D 3515 SM400, SM490, SM520 KS D 3864 SPAR295, SPAP235, SPAP325 KS D 4108 SCW 490-CF	1.2	1.2
	KS D 3861 SN400, SN490 KS D 3866 SHN400, SHN490	1.1	1.1
Plates	KS D 3503 SS400	1.3	1.2
	KS D 3515 SM490, SM490TMC, SM520 SM520TMC, SM570, SM570TMC, SMA400,SMA490,SMA570	1.2	1.2
	KS D 3861 SN400, SN490 KS D 5994 HSA800	1.1	1.1

0713.7.2 Bolted Joints

For general use, three types of bolt connection can be used :
bearing bolt, pretensionsed bolt, and slip-critical bolt connections

For all bolts for seismic design, at least pretensioned high-strength bolts should be used.



In all faying surfaces → Class A or better Slip-Critical Joints
Friction coefficient > 0.35

Design connection strength is permitted to be designed as that of bearing-type connection

In the case of bearing type connection, only standard holes or short-slotted holes should be used.

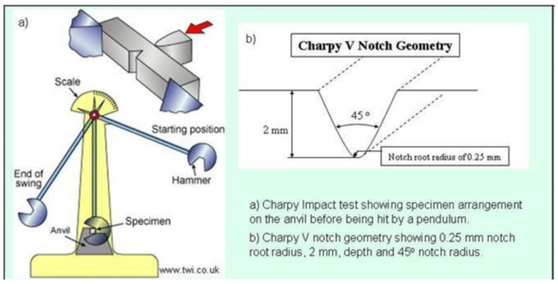
A ductile limit state should controls the design

$P_{ductile} < P_{brittle}$

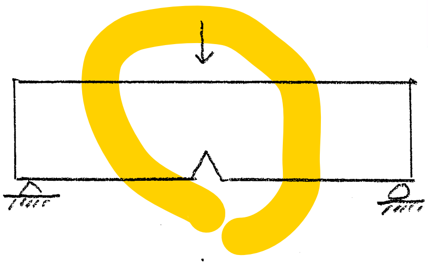
0713.7.3 Welded Joints

Welding for major connections should have sufficient Charpy-V-notch toughness in order to dissipate impact energy.

Requirement : 27 J at -30 C

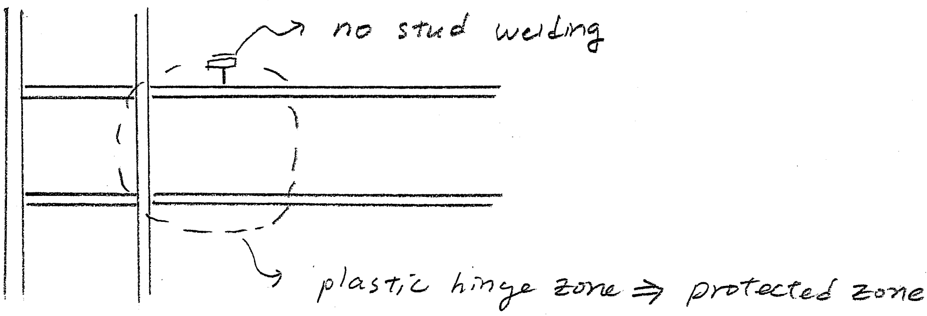


Brittleness depends on temperature



0713.7.4 Protected Zone

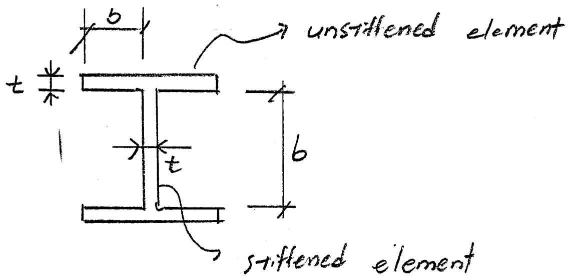
No weld, hole, or bolting even for miscellaneous parts is not permitted within the expected plastic hinge region (protected zone)



0713.8.2 Local Buckling

λ_p , λ_{ps} (seismically compact)

To insure large plastic hysteretic loading , b/t should be sufficiently small.



0713.8.3 Column Strength

When $P_u / \phi P_n \geq 0.4$

The required axial compressive and tensile strength shall be determined by special EQ load using Ω_o (overstrength factor)

But does not necessarily exceed

1) The maximum load transferred by 1.1 R_y X nominal strength of beams and braces

2) Maximum resistance of the foundation
(in tension)

0713.8.4 Column splices

The column splices should resist the column strength specified in 0713.8.3
Splices should be located 1.2m away from the beam-column connection.

14

<Table 0713.8.1.>

Limiting Width-Thickness Ratios for Compression Elements

Description of Element		Width-Thickness Ratio	Limiting Width-Thickness Ratios λ_{ps}(seismically compact)
Unstiffened Elements	Flexure in flanges of rolled or built-up H-shaped sections [a], [c], [e], [f]	b/t	$0.30\sqrt{E/F_y}$
	Uniform compression in flanges of rolled or built-up H-shaped sections [b]	b/t	$0.30\sqrt{E/F_y}$
	Uniform compression in flanges of rolled or built-up H-shaped sections [d]	b/t	$0.38\sqrt{E/F_y}$
	Uniform compression in flanges of channels, outstanding legs of pairs of angles in continuous contact, and braces [c]	b/t	$0.30\sqrt{E/F_y}$
	Uniform compression in flanges of H-pile sections	b/t	$0.45\sqrt{E/F_y}$
	Flat bars	b/t	2.5
	Uniform compression in legs of single angles, legs of double angle members with separators, or flanges of tees	b/t	$0.30\sqrt{E/F_y}$
	Uniform compression in stems of tees	b/t	$0.30\sqrt{E/F_y}$

14-1

Description of Element		Width-Thickness Ratio	Limiting Width-Thickness Ratios λ_{ps}(seismically compact)
Stiffened Elements	Webs in flexural compression in beams in SMF, unless noted otherwise	h/t_w	$2.45\sqrt{E/F_y}$
	Webs in flexural compression or combined flexure and axial compression [a], [c], [f], [g], [h]	h/t_w	$C_a \leq 0.125$ [i] $3.14\sqrt{\frac{E}{F_y}}(1 - 1.54C_a)$ $C_a > 0.125$ [i] $1.12\sqrt{\frac{E}{F_y}}(2.33 - C_a) \geq 1.49\sqrt{\frac{E}{F_y}}$
	Round Tube Section in axial and/or flexural compression [c]	D/t	$0.044E/F_y$
	Rectangular Tube Section in axial and/or flexural compression [c]	b/t 또는 h/t_w	$0.64\sqrt{E/F_y}$
	Webs of H-Pile sections	h/t_w	$0.94\sqrt{E/F_y}$

[a] Required for beams in SMF, Section 0713.9 and SPSW, Section 0713.16.

[b] Required for columns in SMF, Section 0713.9, unless the ratios from Equation (0713.9.3.) are greater than 2.0 where it is permitted to use λ_p in 2.4.1.

[c] Required for braces and columns in SCBF, Section 0713.12 and braces in OCBF, Section 0713.13.

[d] It is permitted to use λ_p in <Table 0702.4.1> for columns in EBF, 0713.13.

[e] Required for link in EBF, Section 0713.13, except it is permitted to use λp in <Table 0702.4.1> for flanges of links of length $1.6M_p/V_p$ or less

[f] Required for beams and columns in BRBF, Section 0713.15.

[g] Required for columns in SPSW, Section 0713.16.

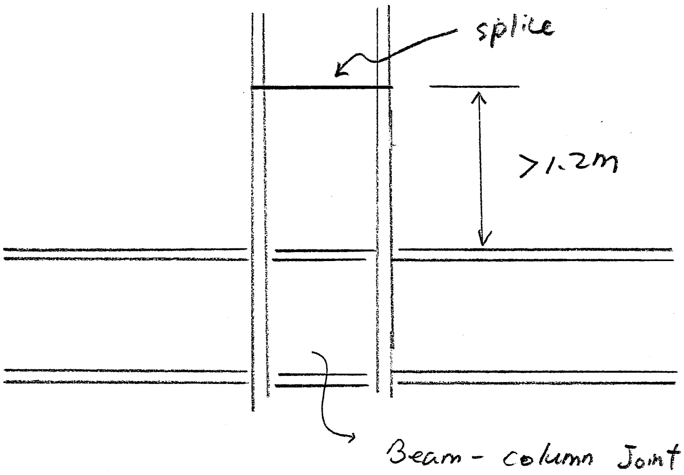
[h] For columns in SMF, Section 0713.9, if the ratios from Equation (0713.9.3) are greater than 2.0; For columns in EBF, Section 0713.13 or EBF webs of links of length $1.6M_p/V_p$ or less, it is permitted to use the following for λ_p:

$$C_a \leq 0.125,\ \lambda_p = 3.76\sqrt{\frac{E}{F_y}}(1-2.75C_a)$$

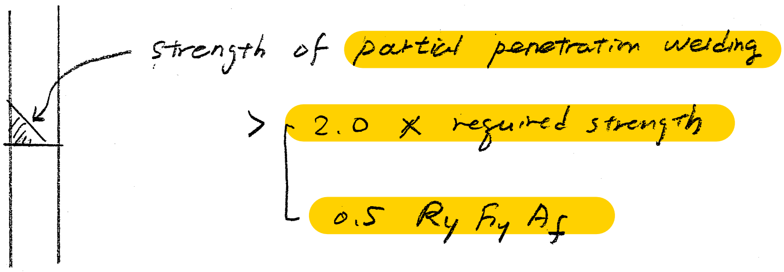
$$C_a > 0.125,\ \lambda_p = 1.12\sqrt{\frac{E}{F_y}}(2.33-C_a) \geq 1.49\sqrt{\frac{E}{F_y}}$$

[i] For LFRD, $C_a = \dfrac{P_u}{\phi_b P_y}$

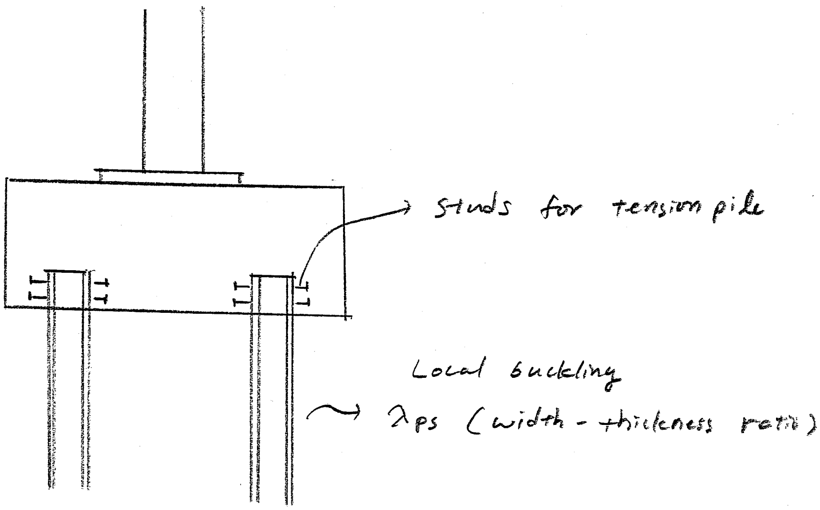
where, P_u : required compressive strength (N), P_y : axial yield strength (N), $\phi_b = 0.90$



When tensile strength is required, the strength of partial penetration welding should be greater than 2.0 x required strength and a half the strength of the column.
In this case, the net tensile force should be calculated using the overstrength factor,



0713.8.6 H-piles



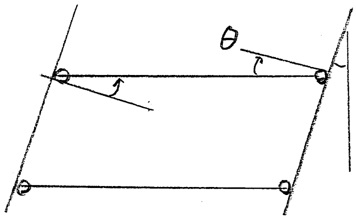
0713.9 Special moment frames (SMF)

A large plastic deformation capacity is required.

0713.9.2 Beam to Column Joints and Connections

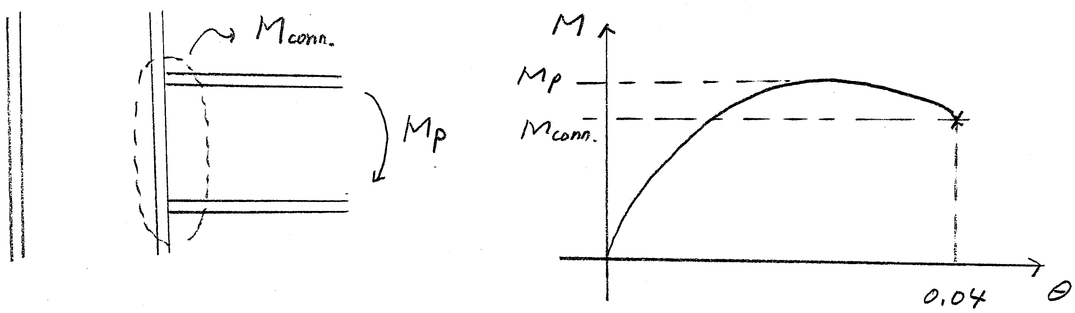
0713.9.2.1 Requirements

(1) θ capacity $\geq$ 0.04 radians (interstory Drift)



(2) $M_{connection}$ at $\theta = 0.04$ $\geq$ $0.8M_P$

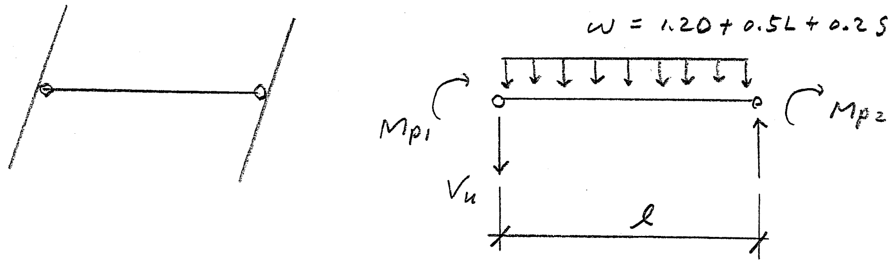
M_p = plastic moment of member connected



(3) Design shear force

Design shear force should be calculated based on the flexural capacity = capacity design

$V_u = \omega l/2 + (M_{p1} + M_{p2})/l$, $M_{p1} = 1.1R_yF_yZ = M_{p2}$



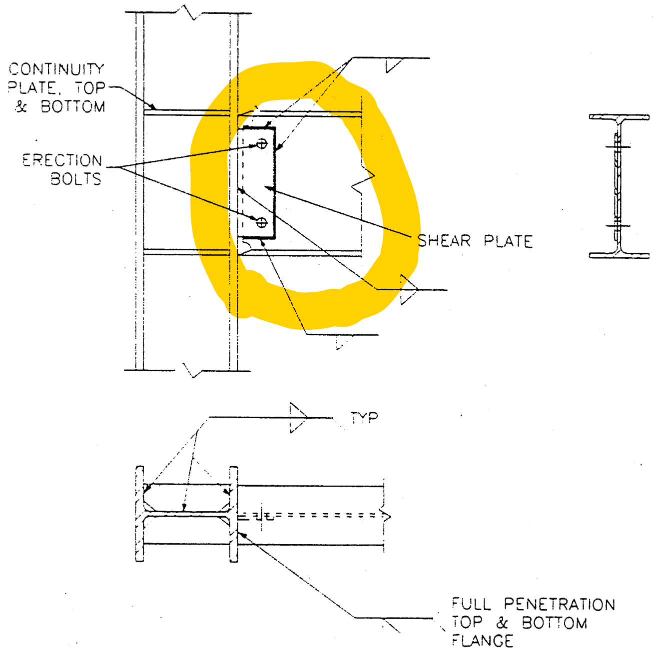
0713.9.2.2 Connection type and details (conformance demonstration)

Connections should be prequalified according to the design code or tested by qualifying cyclic experiment. At least, two specimens should be used.

test methods are specified in 0722.

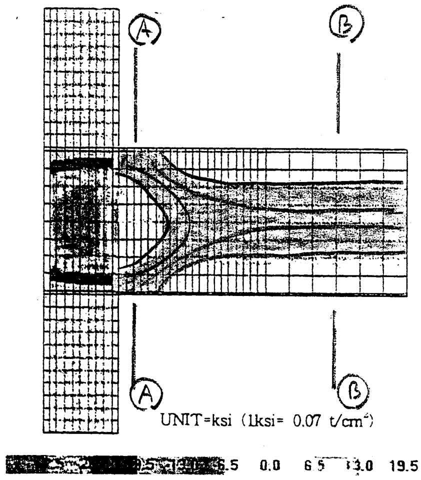
General guidelines for robustness of beam-column connections

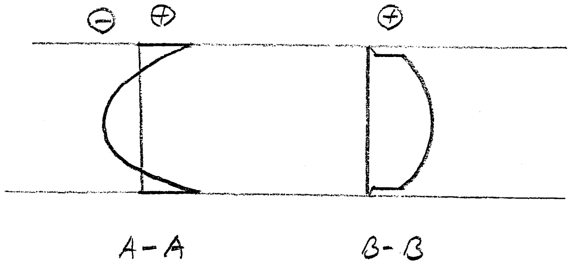
If $M_{fp} < 0.7M_p$, connection of beam web to column should be designed for bending moment resisted by the web as well as shear: the flexural contribution of web should not be neglected.



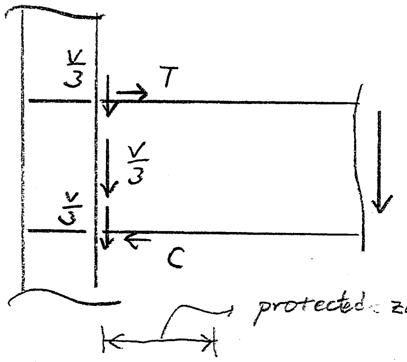
Brittleness of weld connection

1) Presence of residual stress
2) Shear distribution due to the end effect (disturbed region)



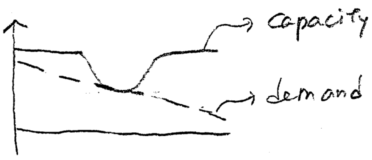


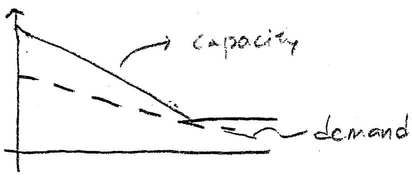
shear stress distribution

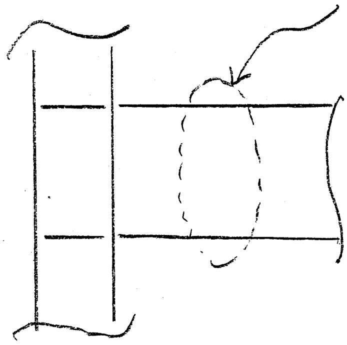


concentration of shear Force
on the Top & Bottom Flanges
cause brittle failure

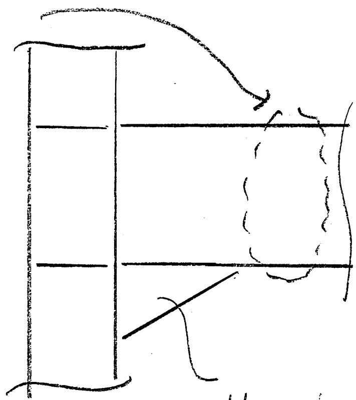
Solutions

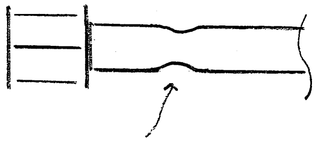






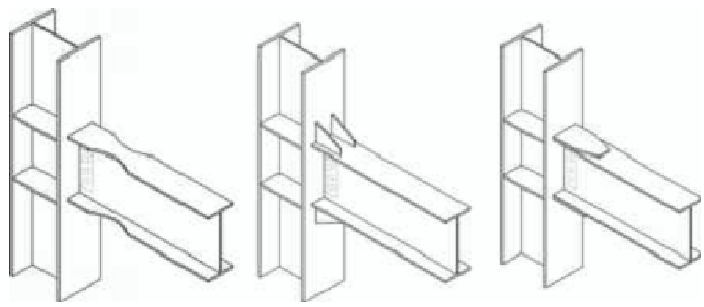
hinge regions
away from
beam-column
joint





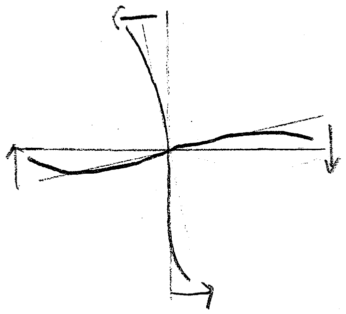
shallow flange width
(Dog bone)

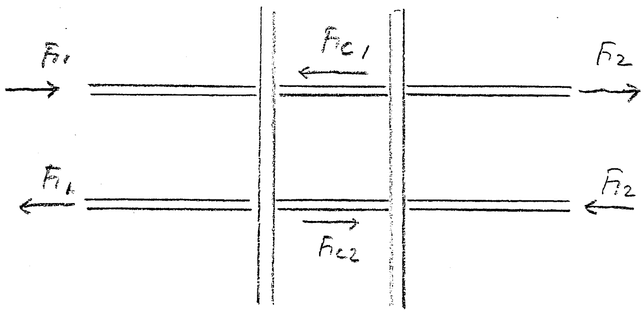
Weaking strategy



0713.9.3 Panel Zone

Shear demand





$$R_u = F_1 + F_2 - F_{c1} = \frac{M_{p1}}{\left(d_{b1} - t_{bf1}\right)} + \frac{M_{p2}}{\left(d_{b2} - t_{bf2}\right)} - \frac{M_{pc1}}{\left(h/2\right)}$$

Design Shear Strength $P_u \le 0.75 P_y$

$$\phi_v R_v = 0.6\phi_v F_y d_c t_p \left[1 + \left(\frac{3 b_{cf} t_{cf}^{\;2}}{d_b d_c t_p}\right)\right]$$ contribution of flange bending action

ϕ_v =1.0

t_p = total thickness of panel zone including doubler plate

d_c = overall column section depth

b_{cf} = width of the column flange

t_{cf} = thickness of the column flange

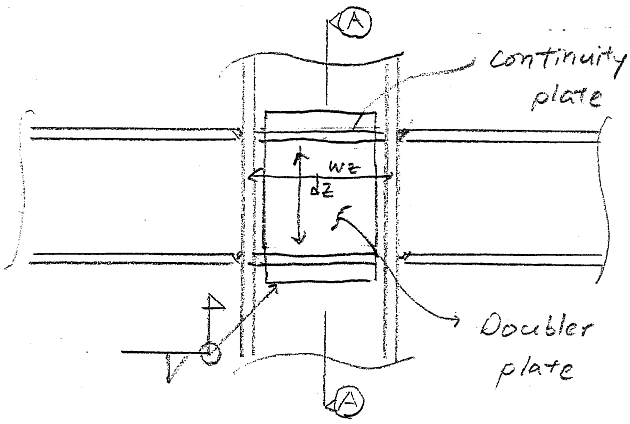
d_b = overall beam depth

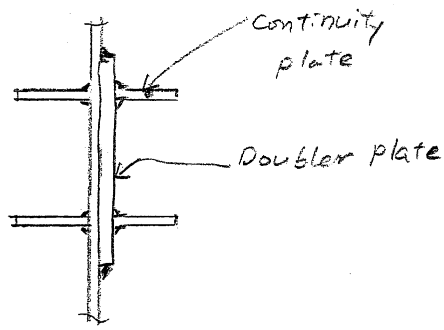
F_y = specified yield strength of the panel zone steel

Panel zone thickness (web) $\geq$ $(d_z + w_z)/90(mm)$

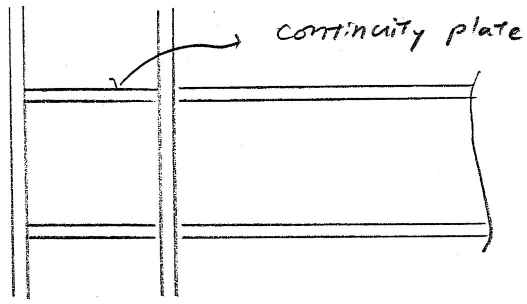
to prevent local buckling of web

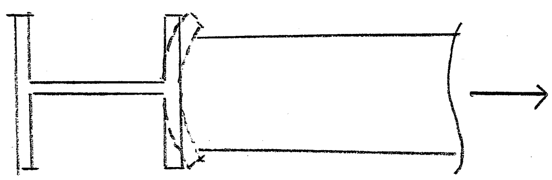
Doubler plate





0713.9.5 Continuity plate





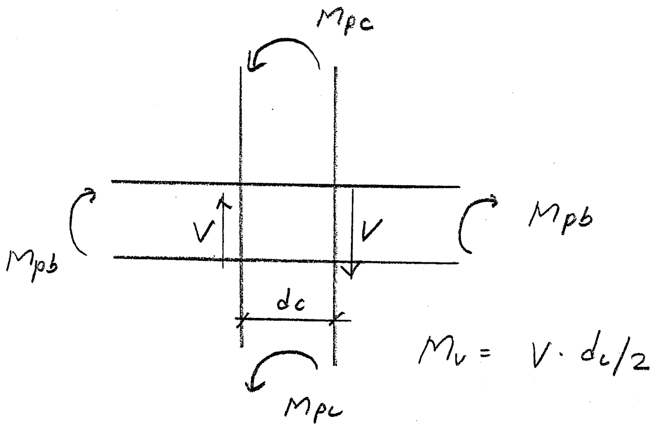
flange bending w/o continuity plate

0713.9.6 Column-beam moment ratio (strong column-weak beam)

column to beam strength ratio

$$\frac{\Sigma Z_c\left(F_{yc}-P_{uc}/A_g\right)}{\Sigma\left(1.1R_yF_{yb}Z_b+M_v\right)}=\left(\frac{\Sigma M_{pc}}{\Sigma M_{pb}}\right)\geq 1.0$$

)



A_g = gross area

F_{yb}, F_{yc} = yield strength

H = average story Height above and below the joint

P_{uc} = column axial force (≥ 0)

V_n = nominal strength of panel zone

Z_b, Z_c = plastic section modulus

d_b = depth of beam

These formulas require that the initial potential for yielding at a beam-to-column connection be in the beam or panel zone rather than in the column

$\Rightarrow$ "Strong column – weak beam"

0713.9.8Lateral bracing of Beams

Both flanges of a beam shall be laterally braced, with a maximum spacing of $L_b = 0.086 r_y E / F_y$.

at the locations where a plastic hinge will form during inelastic behavior of SMF.

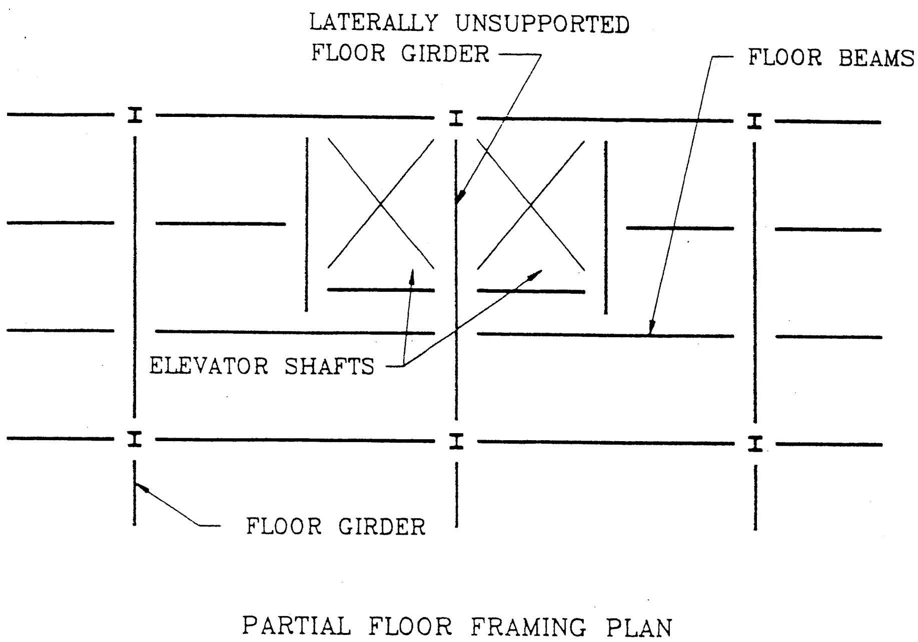


Figure 8-2 Example of laterally unsupported girder in elevator shaft framing.

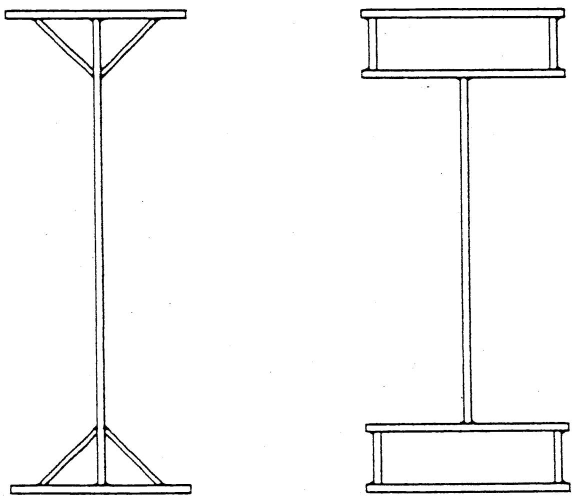


Figure 8-3 Girder sections with boxed flanges.

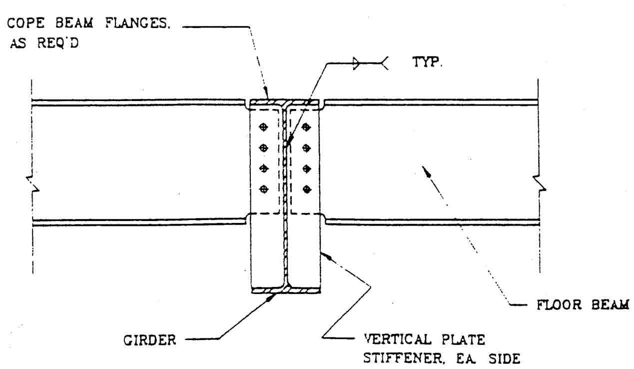


Figure 8-4 Floor beam connection to provide lateral support for lower flange of girder.

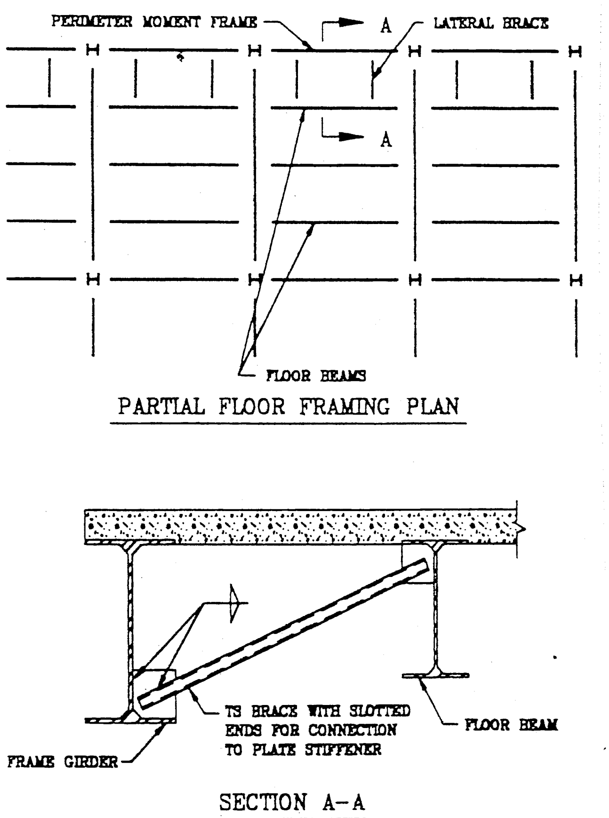
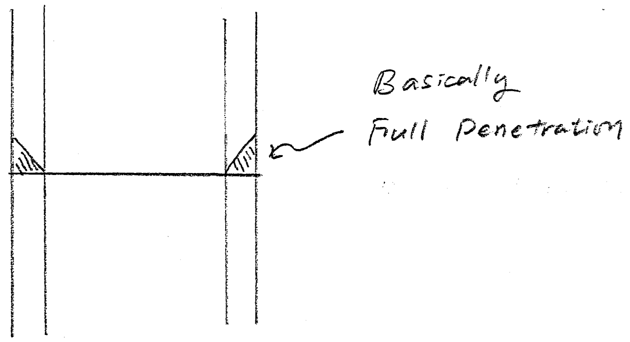


Figure 8-5 Typical detail for lateral bracing of compression flange of perimeter frame girders.

0713.9.9column splices

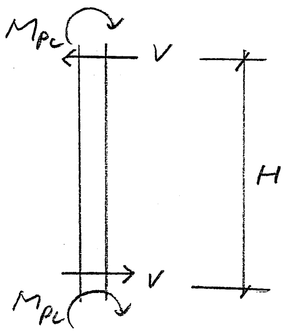
Basically, full penetration should be used for weld connection.



When partial penetration welding is used,

flexural strength of column splice > $R_y F_y Z$ (column strength)

Design shear of column splice $\geq 2M_{pc}/H$



$V_u = 2M_{pc}/H$

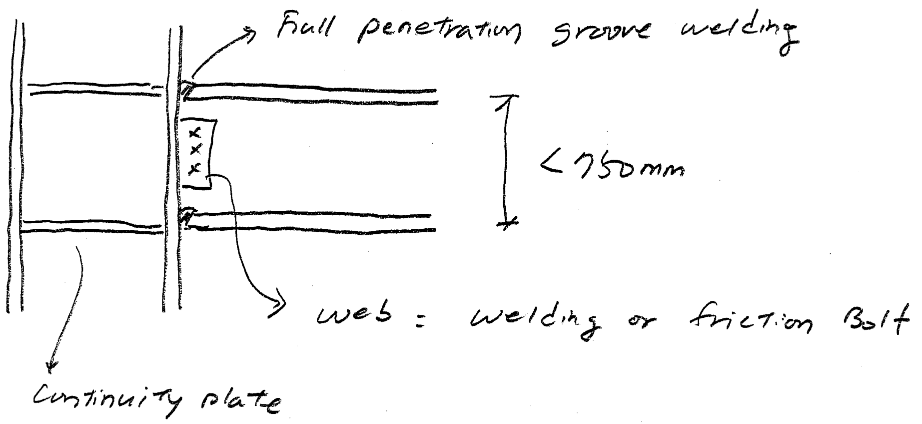
0713.10 Intermediate moment frame (IMF)

less ductility requirements than SMF

Minimum required story drift ratio = 0.02 rad.
(vs 0.04 rad. for SMF)

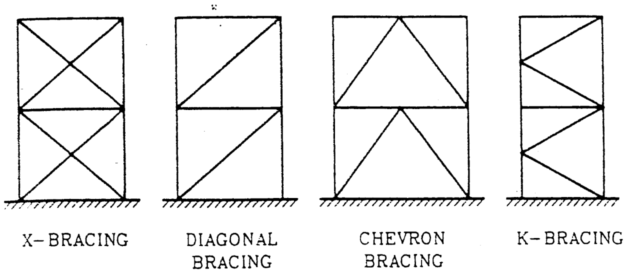
0713.10.2.2 beam to column connection

acceptable for the following details



0713.11 Ordinary moment frame (OMF)

0713.12 Special Concentrically braced Frame



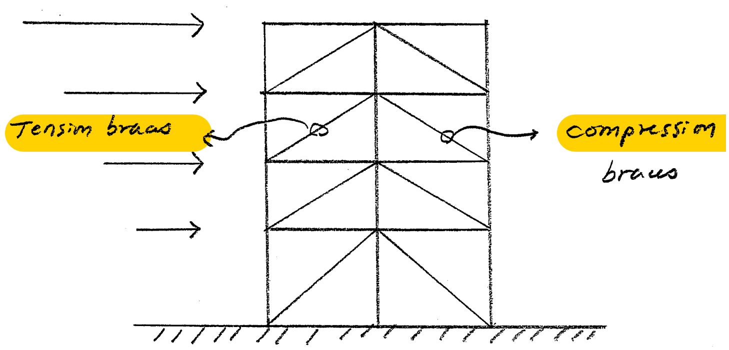
Bracing Member

a) Slenderness ratio : $KL / r \leq 4\sqrt{E_s / F_y}$

b) $F_u A_n \geq R_y F_y A_g$ (to prevent tensile fracture)

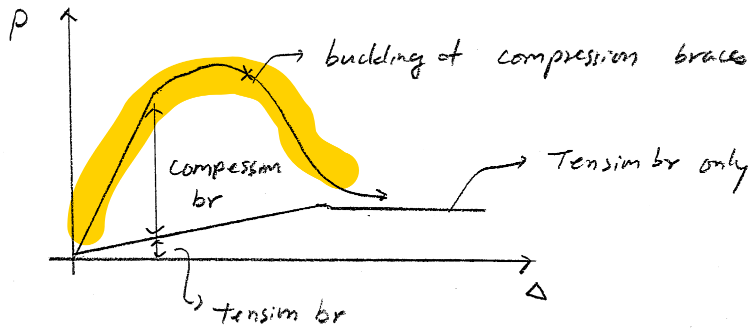
c) Tension bracings should resist at least 30% but no more than 70% of the total horizontal force

unless ϕP_n (brace in compression) $\geq$ special EQ load (Ω_o factor)



The forces of braces changes from compression to tension, or vice-versa under reversed cyclic loading.

Under compression, the braces are susceptible to buckling, which reduce the overall strength of the structure.



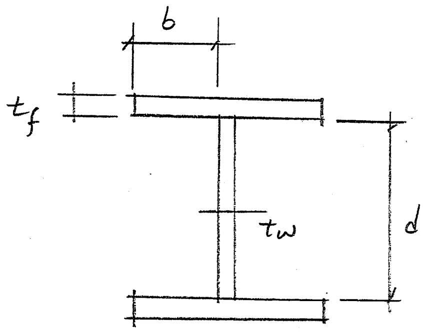
d) Braces should be compact section

$\lambda < \lambda_p$

Compact section : no-local buckling

Non-comport section: inelastic local buckling

Slender section: elastic local buckling



stiffened element (web) : slenderness ratio = d/t_w

unstiffened element (flange) : slenderness ratio = b/t_f

connections of brace members

a) connection tensile forces = design axial tension strength of the member $R_y F_y A_g$

Design Limit State : yield in gross area

Fracture in net Area

Block shear

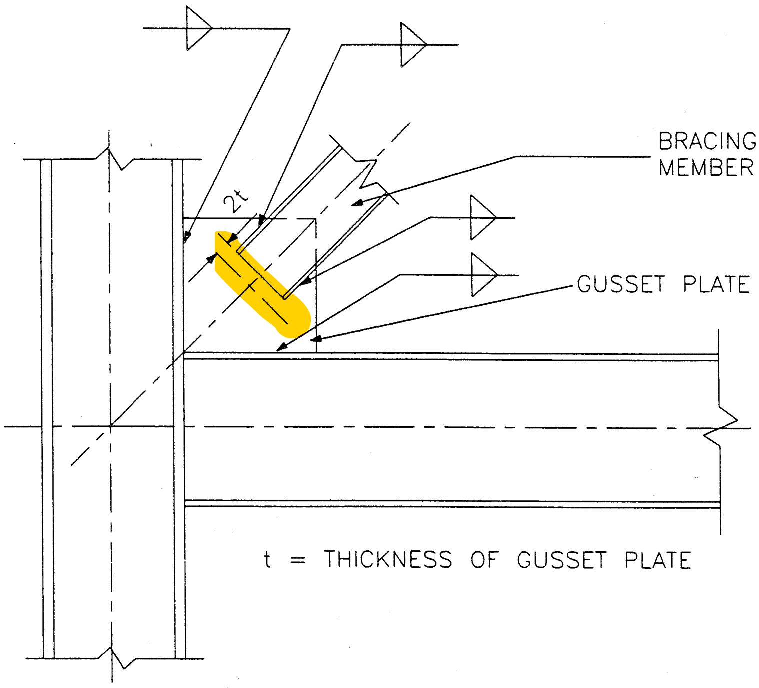
b) flexural strength of connection $\geq 1.1 R_y M_p$ of brace member in the direction that the brace will buckle.

c) connection compressive force = $1.1\ R_y P_n$

Gusset Plate

Sufficient strength to resist in-plane bucking mode

Detailed to permit the formation of a hinge line in a gusset



t = THICKNESS OF GUSSET PLATE

0713.12.4.1V and Inverted V Type Bracing

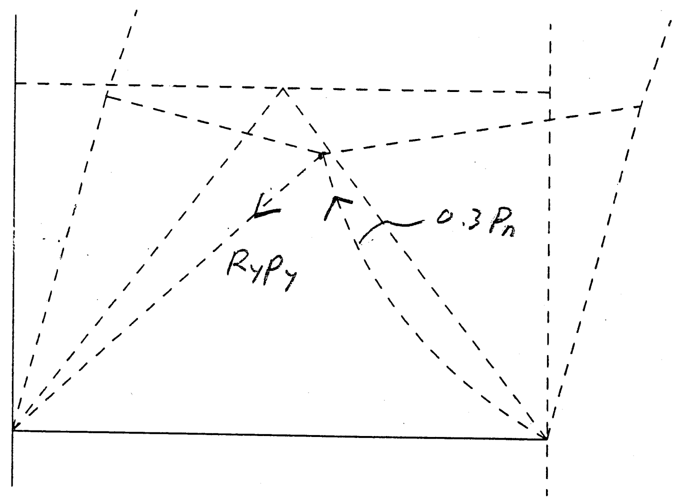
a) Beam should resist maximum unbalanced force + gravity loading

maximum unbalanced force $= (R_y P_y - 0.3\cancel{\phi_c} P_n)\cos\theta$

b) Beam should be continuous

c) Beam shall be capable of supporting all tributary dead and live loads assuming the bracing is not present

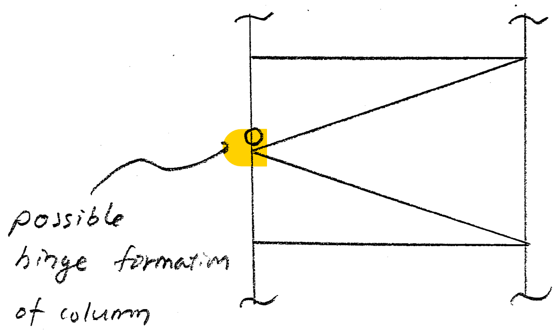
d) Beam shall be laterally supported



0713.12.4.2

K bracing system is not permitted in high seismic zone

High formation in column is not desirable.

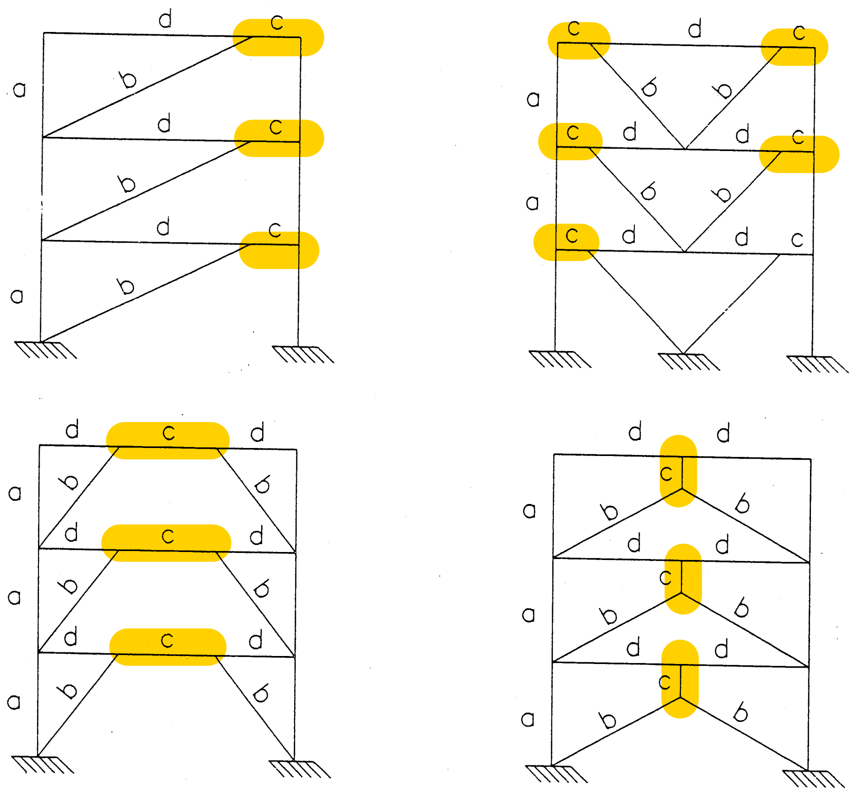


0713.14 Eccentrically Braced Frame (EBF)

Design Concept of EBF

c: designed as a ductile member (fuse) under high compression and bending moments

a,b,d : remain elastic even during large EQ



a = column
b = brace
c = link
d = portion of beam outside of link

Link

a) Link should meet local buckling requirements of columns (seismic compact section). Link is a compression member in high compression, which has large ductility capacity.

b) No opening and no double plate in the web. The web of the link is subjected to high shear.

c) Shear strength

For $P_u < 0.15P_y$

$\phi V_n = 2\phi M_p / e$: governed by flexural strength

or $= \phi 0.6F_y(d-2t_y)t_w$: governed by shear strength

For $P_u \geq 0.15P_y$ (interaction with compression should be addressed)

$$\phi V_n = 2\phi\left\{1.18M_p\left[1-\left(P_u / P_y\right)\right]\right\} / e$$

or = $\phi 0.6F_y(d-2t_y)t_w\sqrt{1-(P_u / P_y)^2}$

d) Link rotation angle (rotational capacity) γ_p

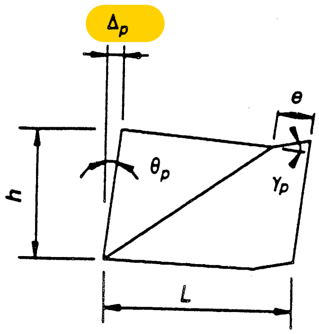
$\gamma_p \leq 0.08$ radians for $e < 1.6M_p / V_p$ short link

$\gamma_p \leq 0.02$ radians for $e > 2.6M_p / V_p$ slender link

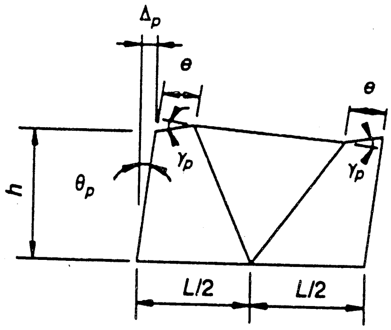
Linear interpolation can be used in between.

Link Stiffeners

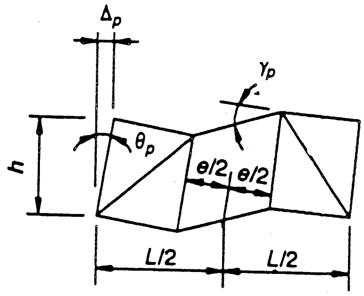
a)As the length of link decreases, and as the required link rotation angle increases, Spacing of the web stiffener of the link should be decreased.



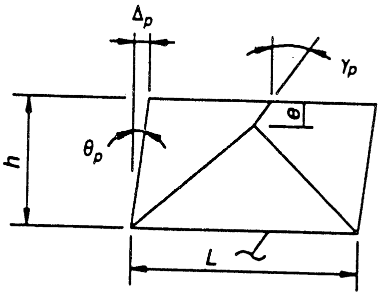
$$\gamma_p = \frac{L}{e}\theta_p$$



$$\gamma_p = \frac{L}{2e}\theta_p$$

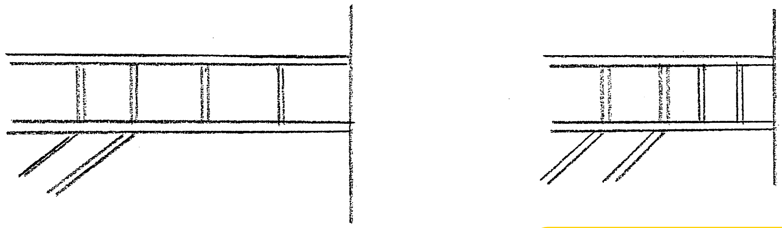


$$\gamma_p = \frac{L}{e}\theta_p$$



$$\gamma_p = \frac{h}{e}\theta_p$$

Δ_v = Story drift determined using base shear v, inches.
Δ_t = Total story drift, inches = $\Delta_v \times e'/e$.
Δ_e = Elastic story drift, inches = Δ_v times the earthquake load factor.
Δ_p = Plastic story drift, inches = $\Delta_t - \Delta_e$ (conservatively, $\Delta_e = 0$).
e = Link length, inches.
h = Story height, inches.
L = Column to column distance, inches.
θ_p = Plastic story drift angle, radians = Δ_p/h.
γ_p = Link rotation angle, radians.

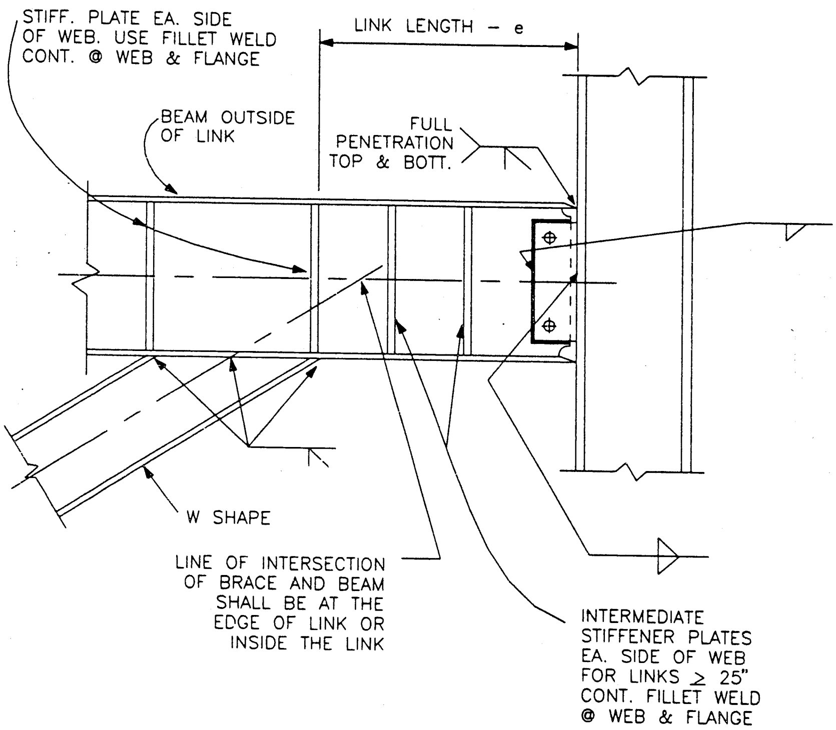


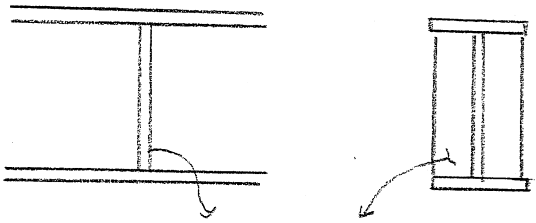
Greater rotational demand

b) If the length of link (e) $\geq 5M_p / V_p$, no stiffener is required

c) Lateral supports should be provided at the Link.

d) Link to column connection must be able to sustain the maximum Link rotation angle.



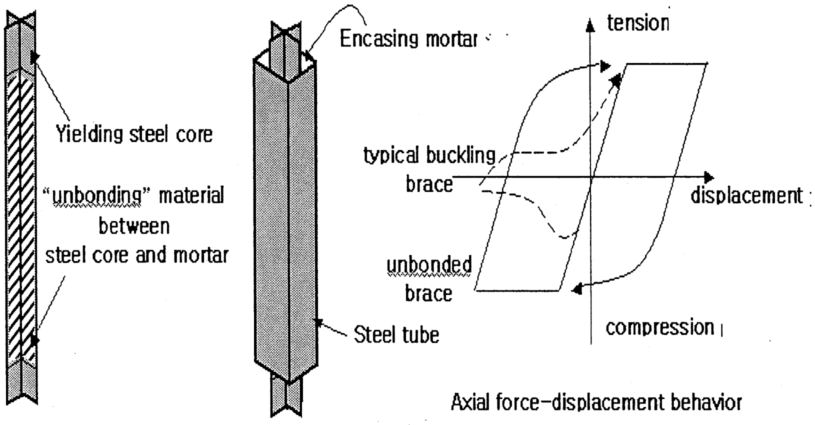


Combined width $\geq (b_f - 2t_w)$

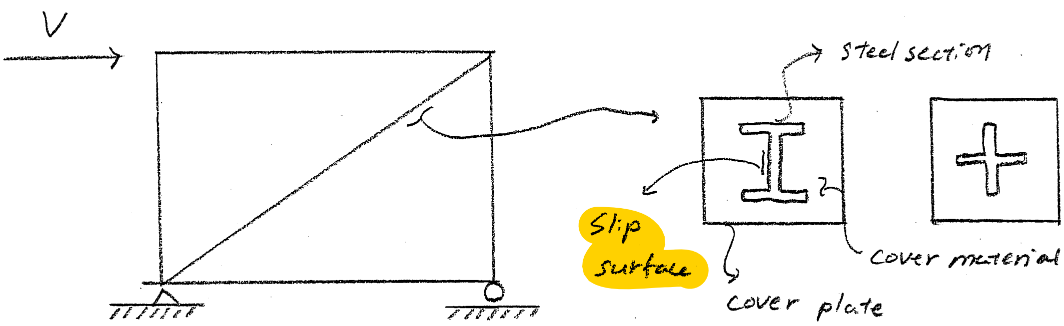
Stiffner thickness $\geq 0.75\, t_w$

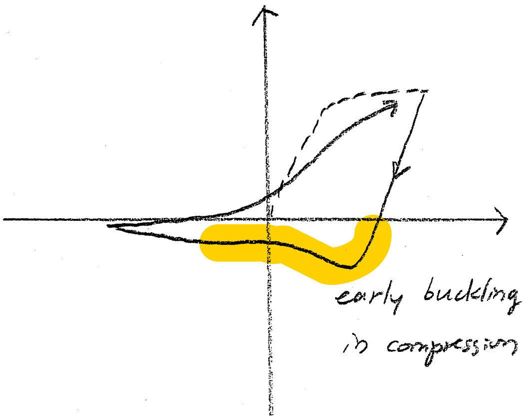
10 mm

0713.15 Buckling-Restrained Braced Frames (BRBF)

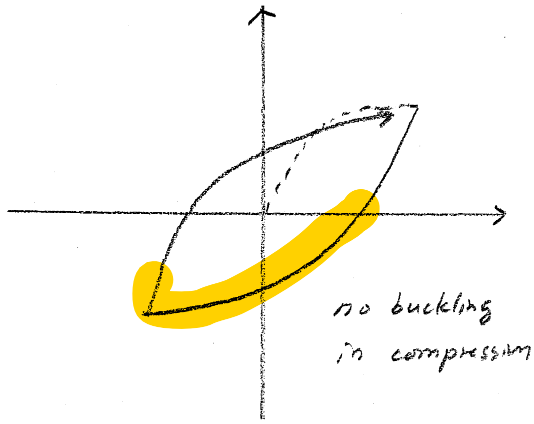


Axial force–displacement behavior

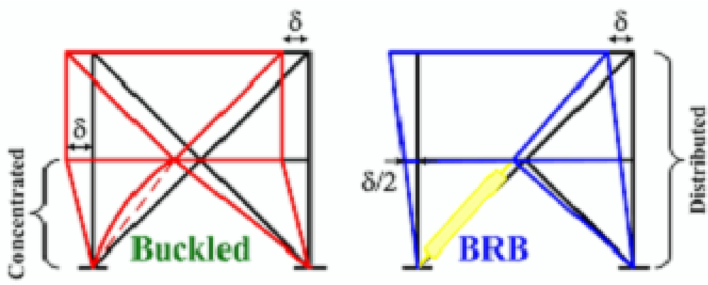




ordinary braces



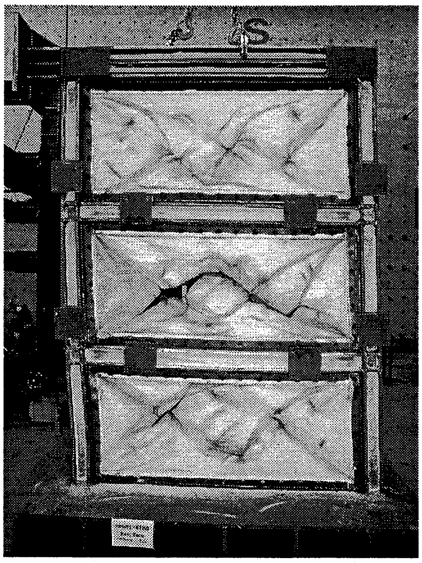
unbonded braces

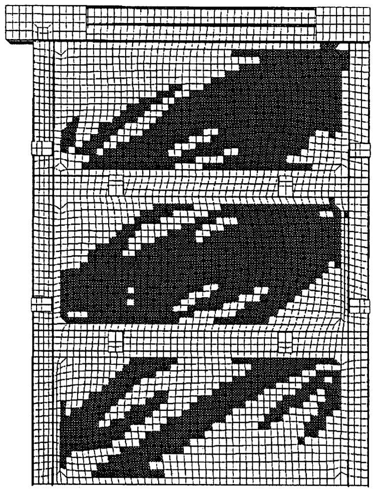


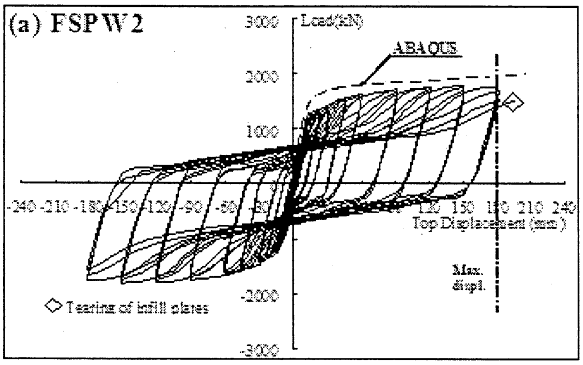
Coventional-BF

Strongback-BF

0713.16 Special Plate Shear Walls (SPSW)





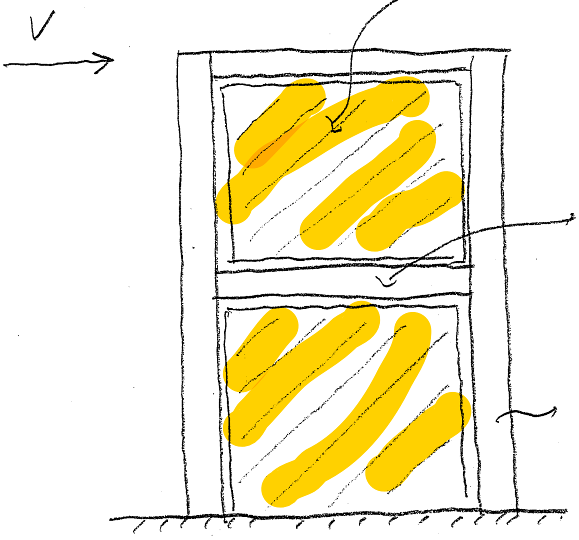


In case of steel plate wall with stiffened steel plate or thick plate
Shear yield or plate buckling is prevented, and flexural mode governed the behavior.

In case of steel plate wall with unstiffened steel plates
Plate buckling occurs under shear. Thus, shear deformation mode governs the behavior. Unlike other shear mode, local buckling occurs very early at loading. Thus, no overstrength is developed, and the behavior becomes very ductile.

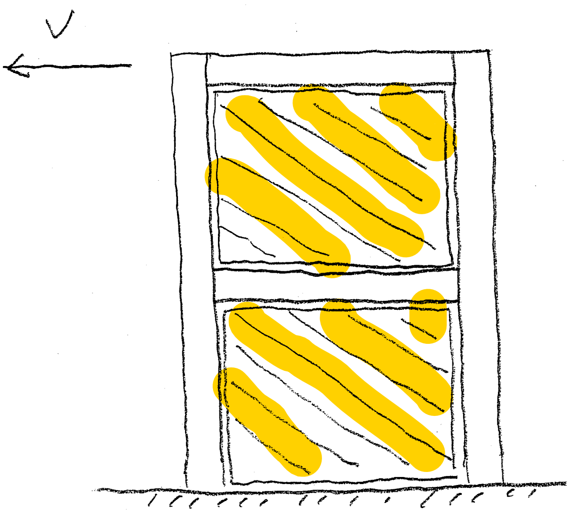
For this reason, the unstiffened steel plate wall show excellent ductility capacity without showing overstrength.

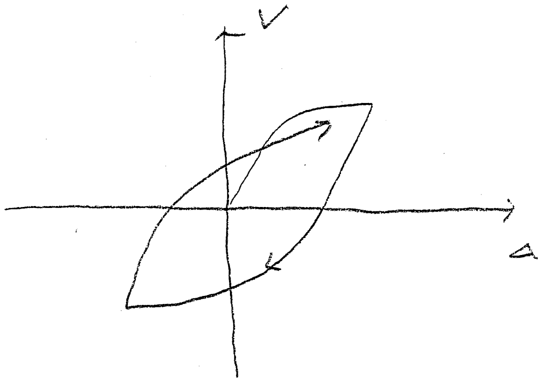
Stiffened steel plate
or
unstiffened
steel plate
- early local buckling
- tension field action
- tension brace either way of lateral force



horizontal boundary Element

vertical boundary element





- large deformation capacity
- high strength
- large energy dissipation capacity