Seismic Design of Steel Structures: North American Practice & Challenges for Industrial Buildings

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Santiago, Chile
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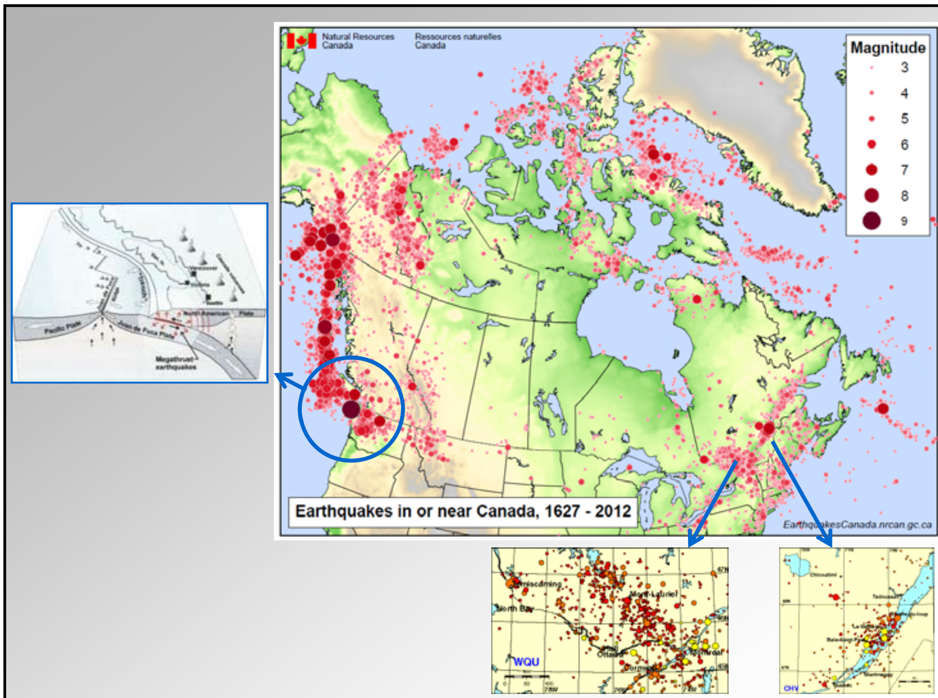
Group for Research in STRUCTURAL ENGINEERING

POLYTECHNIQUE MONTRÉAL



NORMA CHILENA OFICIAL

NCh 433.Of1996
Modificada en 2009

INSTITUTO NACIONAL DE NORMALIZACION • INN-CHILE

Diseño sísmico de edificios

NORMA CHILENA OFICIAL

NCh 2369.Of2003

INSTITUTO NACIONAL DE NORMALIZACION • INN-CHILE

Diseño sísmico de estructuras e instalaciones industriales

Earthquake-resistant design of industrial structures and facilities

NCh 433.Of1996
Modificada en 2009

NCh433

Tabla 5.1 - Valores máximos de los factores de modificación de la respuesta[1]

Sistema estructural	Material estructural	R	R_0	R ASCE/SEI 7-10
Pórticos	**Acero estructural**			
	a) Marcos corrientes (OMF)	4	5	3-1/2
	b) Marcos intermedios (IMF)	5	6	4-1/2
	c) Marcos especiales (SMF)	7	11	8
	d) Marco de vigas enrejadas (STMF)	6	10	7
	Hormigón armado	7	11	
Muros y sistemas arriostrados	**Acero estructural**			
	a) Marcos concéntricos corrientes (OCBF)	3	5	3-1/4
	b) Marcos concéntricos especiales (SCBF)	5,5	8	6
	c) Marcos excéntricos (EBF)	6	10	8
	Hormigón armado	7	11	
	Hormigón armado y albañilería confinada			
	- Si se cumple el criterio *A* [2]	6	9	
	- Si no se cumple el criterio *A* [2]	4	4	
	Madera	5,5	7	
	Albañilería confinada	4	4	

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations Including Structural Height, h_n (ft) Limits[c] — Seismic Design Category: B	C	D^d	E^d	F^e
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
25. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	160	160	100
26. Steel special plate shear walls	14.1	7	2	6	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[h]	NP[h]	NP[h]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[i]	NP[i]	NP[i]

NCh 2369.Of2003

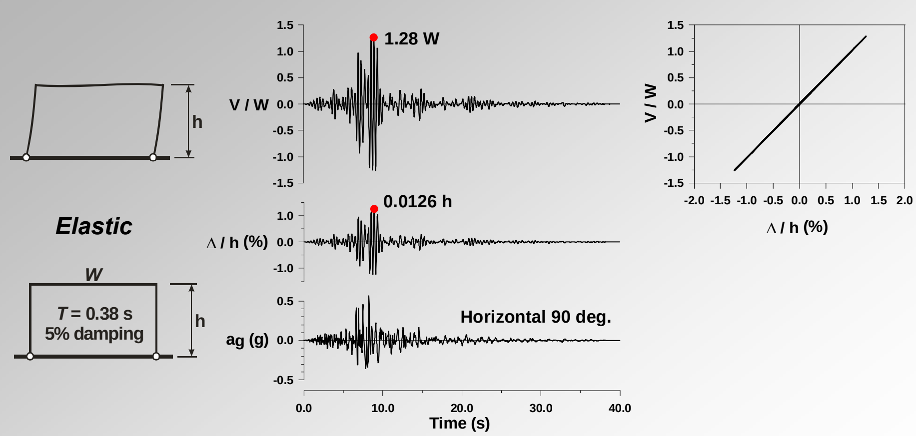
Tabla 5.6 - Valores máximos del factor de modificación de la respuesta

	Sistema resistente	*R*
1.	Estructuras diseñadas para permanecer elásticas	1
2.	Otras estructuras no incluidas o asimilables a las de esta lista[1)]	2
3.	Estructuras de acero	
3.1	Edificios y estructuras de marcos dúctiles de acero con elementos no estructurales dilatados	5
3.2	Edificios y estructuras de marcos dúctiles de acero con elementos no estructurales no dilatados e incorporados en el modelo estructural	3
3.3	Edificios y estructuras de marcos arriostrados, con anclajes dúctiles	5
3.4	Edificios industriales de un piso, con o sin puente grúa, y con arriostramiento continuo de techo	5
3.5	Edificios industriales de un piso, sin puente-grúa, sin arriostramiento continuo de techo, que satisfacen 11.1.2	3
3.6	Naves de acero livianas que satisfacen las condiciones de 11.2.1	4
3.7	Estructuras de péndulo invertido[2)]	3
3.8	Estructuras sísmicas isostáticas	3
3.9	Estructuras de plancha o manto de acero, cuyo comportamiento sísmico está controlado por el fenómeno de pandeo local	3

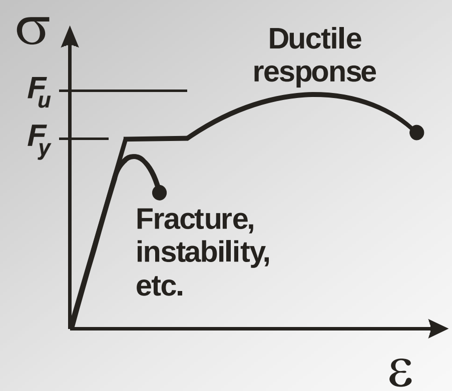
Plan

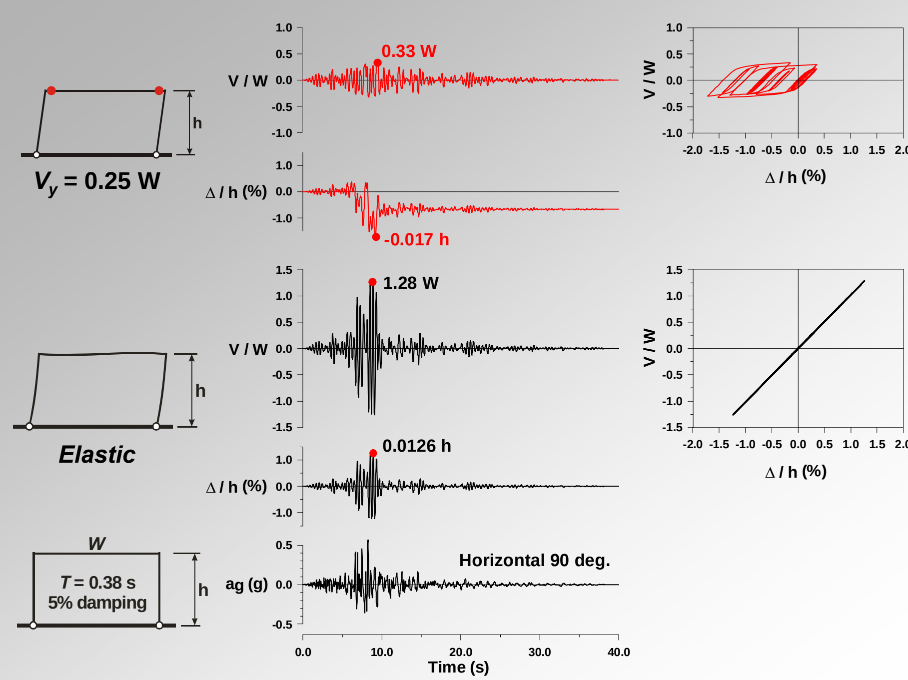
- Ductile seismic design
- Braced frame systems
  - Concentrically braced frames
  - Eccentrically braced frames 341-10
  - Buckling restrained braced frames
- Moment frames and plate wall systems
- Heavy industrial buildings: challenges

Ductile Seismic Design



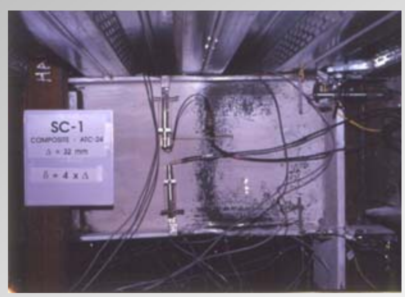
Take advantage of the high ductility of steel



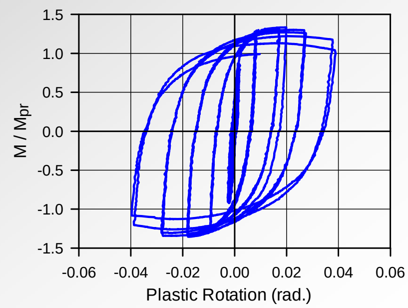


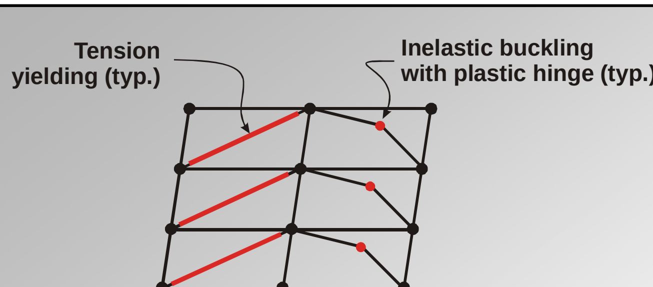
M. Englehardt

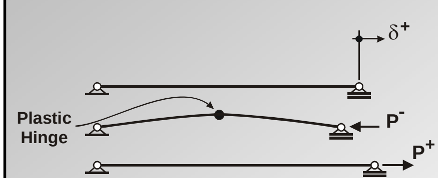
Ecole Polytechnique of Montreal, 1996

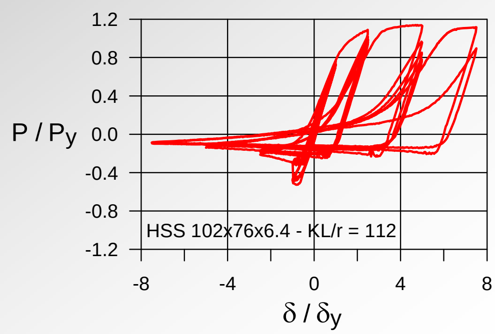


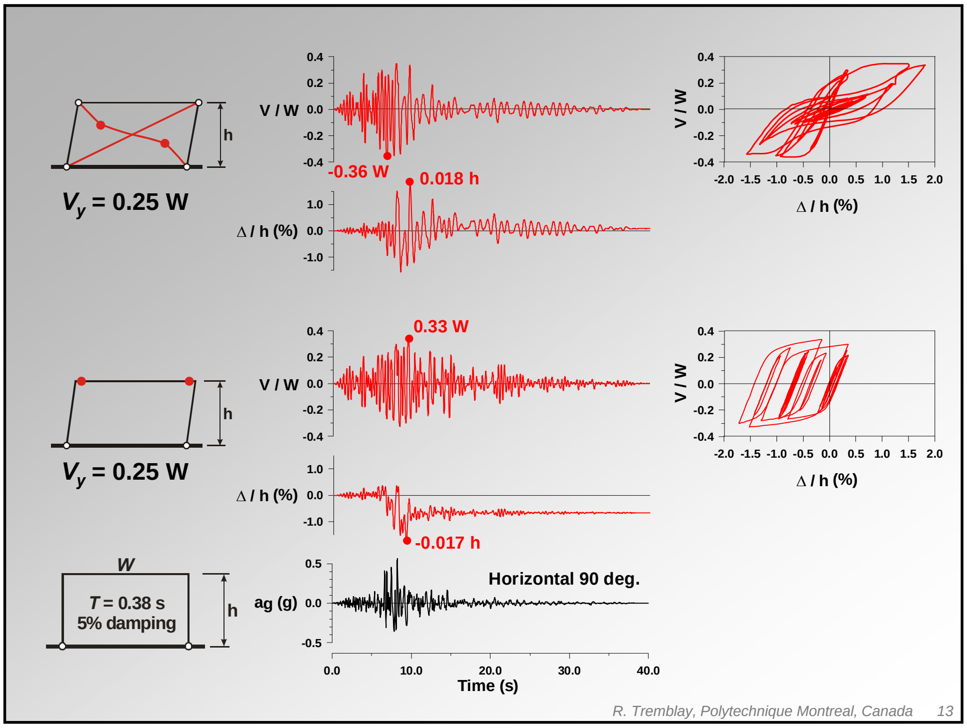
Ecole Polytechnique of Montreal, 1996











V_y = 0.25 W
V / W
-0.36 W
0.018 h
Δ / h (%)
0.33 W
V_y = 0.25 W
-0.017 h
W
T = 0.38 s
5% damping
ag (g)
Horizontal 90 deg.
Time (s)
Δ / h (%)
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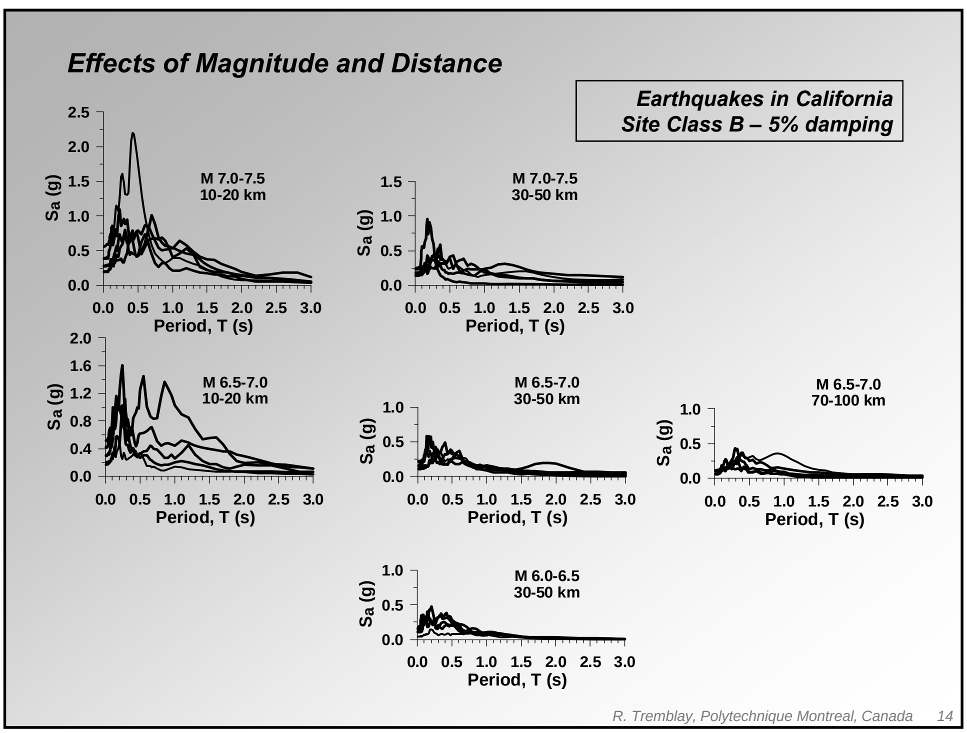

Effects of Magnitude and Distance
Earthquakes in California
Site Class B – 5% damping
M 7.0-7.5
10-20 km
M 7.0-7.5
30-50 km
M 6.5-7.0
10-20 km
M 6.5-7.0
30-50 km
M 6.5-7.0
70-100 km
M 6.0-6.5
30-50 km
Sa (g)
Period, T (s)
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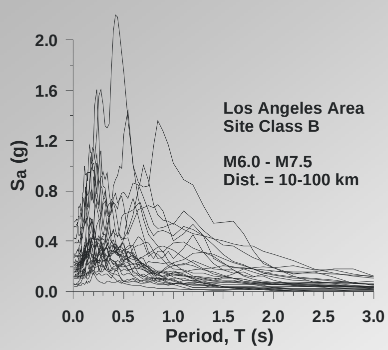
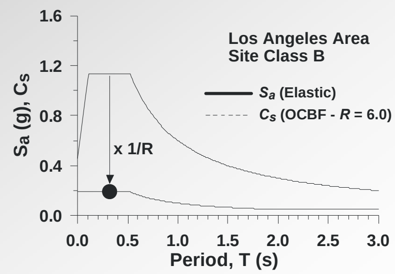
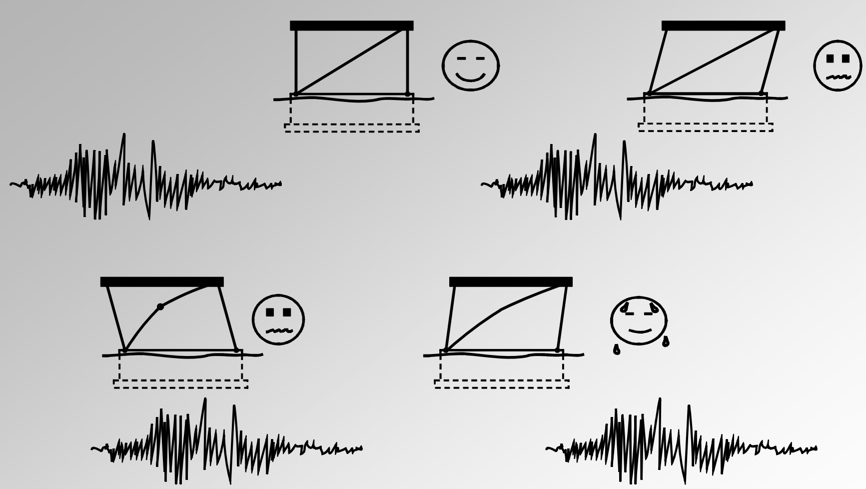
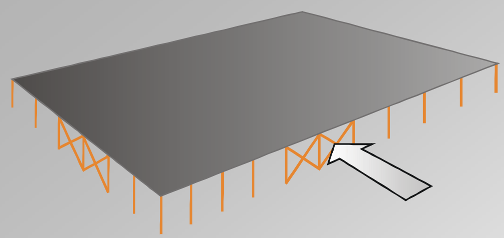
Design Spectra

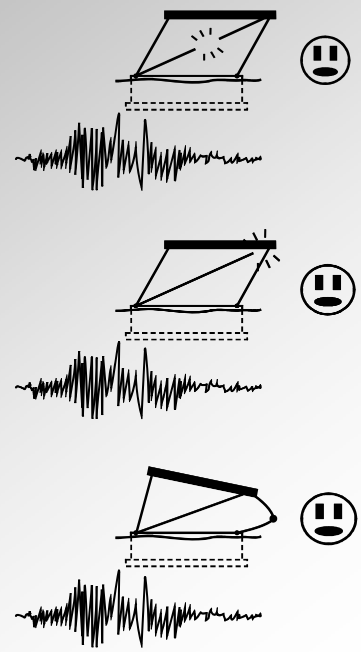
& Design for Inelastic Response



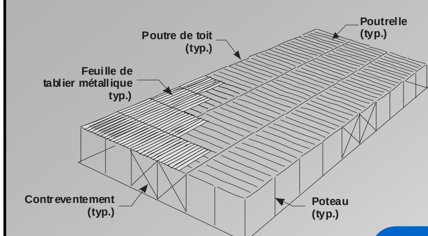




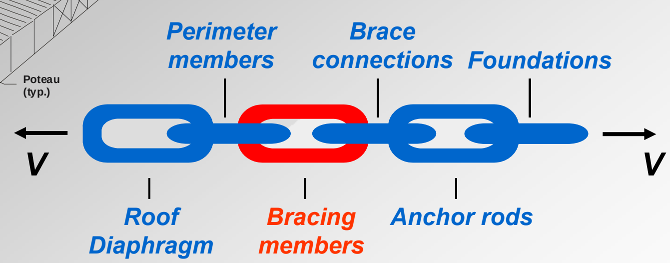




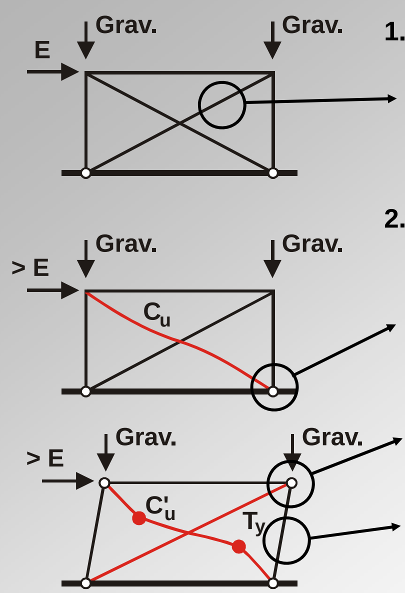
Capacity Design Procedure



$$V = \frac{V_e}{R}$$



Two-Step "Capacity Design" Procedure:



1. Select Braces:

Design for gravity + E
Check *KL*/*r*, *b*/*t*, etc. for ductile response

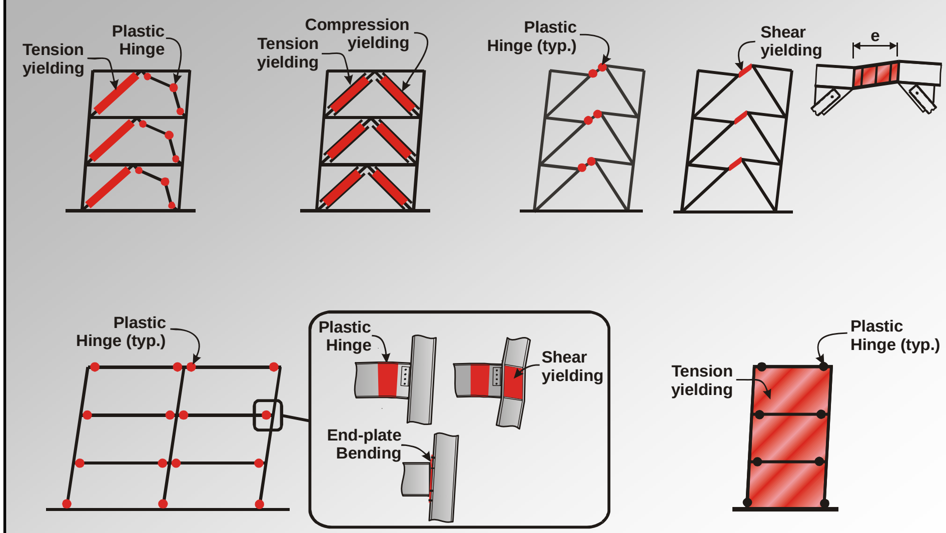
2. Design other elements :

Gusset plates designed in compression for the expected brace compressive strength

Gusset plate designed in tension for the expected brace tensile strength

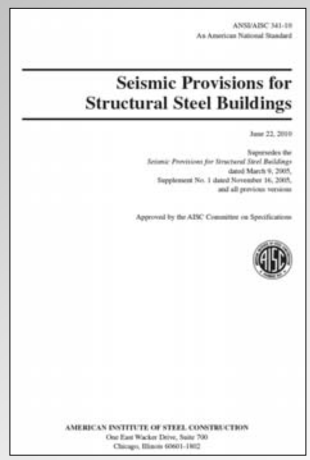
Column designed for gravity plus expected brace tensile strength

Ductile Seismic Force Resisting Systems



ASCE 7-10

AISC 341-10



AISC 360-10

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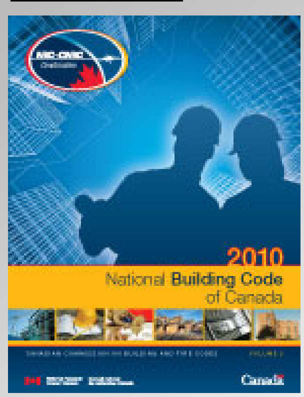
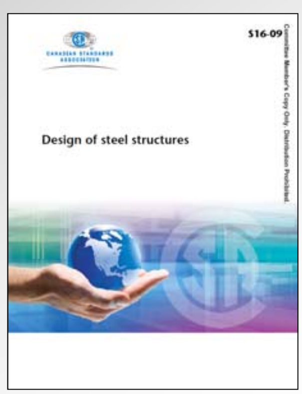
NBCC 2010



CSA S16-09



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Building code main objective is to protect life and prevent structural collapse under ground motions

In view of high level of ground motion considered (2% in 50 years) and the difficulty in predicting ground motions and their effects on structures …

… ductile seismic design represents an effective strategy to achieve code objectives by controlling the system response and force demand.

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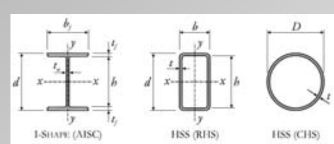
Control of Local Buckling



Element	Ratio	Highly ductile members λ_{hd}	Moderately ductile Members λ_{md}
Flange of I-sections	$b_f/2t_f$	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$
Web of I-sections in beams or columns[1,2]	h/t_w	For $C_a \leq 0125$: $2.45\sqrt{\frac{E}{F_y}}(1-0.93C_a)$ For $C_a > 0.125$: $0.77\sqrt{\frac{E}{F_y}}(2.93-C_a) \geq 1.49\sqrt{\frac{E}{F_y}}$	For $C_a \leq 0.125$: $3.76\sqrt{\frac{E}{F_y}}(1-2.75C_a)$ For $C_a > 0.125$: $1.12\sqrt{\frac{E}{F_y}}(2.33-C_a) \geq 1.49\sqrt{\frac{E}{F_y}}$
Web of I-sections used as braces	h/t_w	$1.49\sqrt{E/F_y}$	$1.49\sqrt{E/F_y}$
Wall of RHS[3]	$(b-3t)/2t$ $(d-3t)/2t$	$0.55\sqrt{E/F_y}$	$0.64\sqrt{E/F_y}$
Wall of CHS[4]	D/t	$0.038E/F_y$	$0.044E/F_y$

[1] $C_a = P_u/\phi_c A F_y$, where $\phi_c = 0.9$, P_u = required axial strength, A = cross-section area.
[2] When $C_a \leq 0.125$ for I-shaped beams, $h/t_w \leq 2.45\sqrt{E/F_y}$ in SMFs and $3.76\sqrt{E/F_y}$ in IMFs.
[3] Limit = $1.12\sqrt{E/F_y}$ for moderately ductile RHS members used as beams or columns.
[4] Limit = $0.7\,E/F_y$ for moderately ductile CHS members used as beams or columns.

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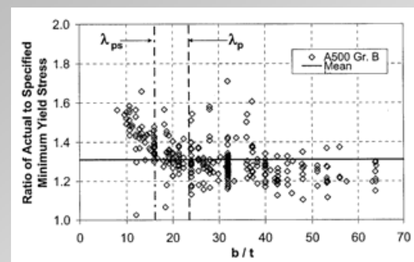
Expected (probable) material strength

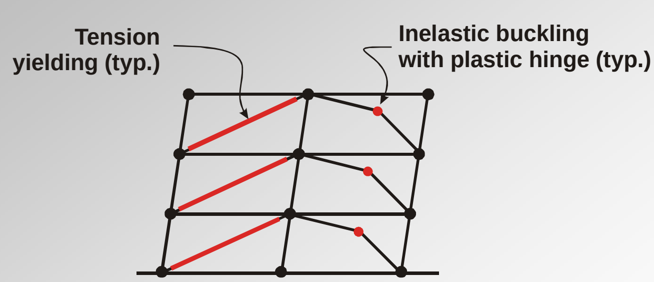


Liu, J. et al. (2007). AISC Eng.

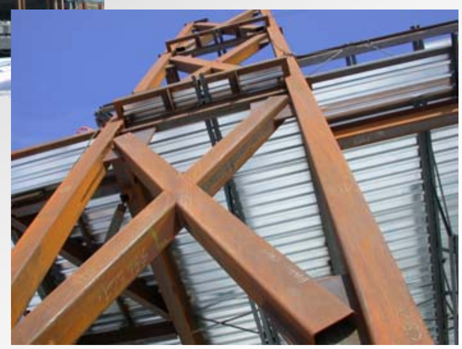
Application	Material	F_y (MPa)	F_u (MPa)	R_y (MPa)	R_t (MPa)
Structural shapes	ASTM A36	250	400	1.5	1.2
	ASTM A572, grade 345	345	450	1.1	1.1
	ASTM A913, grade 415	415	520	1.1	1.1
	ASTM A913, grade 450	450	550	1.1	1.1
	ASTM A992	345	450	1.1	1.1
Plates	ASTM A36	250	400	1.3	1.2
	ASTM A572, grade 290	290	415	1.3	1.0
	ASTM A572, grade 345	345	450	1.1	1.2
HSS	ASTM A500, grade C (round)	317	427	1.4	1.3
	ASTM A500, grade C (rectangular)	345	427	1.4	1.3
Pipes	ASTM A53, grade B	240	415	1.6	1.2

Concentrically Braced Frames (SCBFs)

Energy dissipated in bracing members through tensile yielding and flexural hinging



Connections and other members expected to remain essentially elastic



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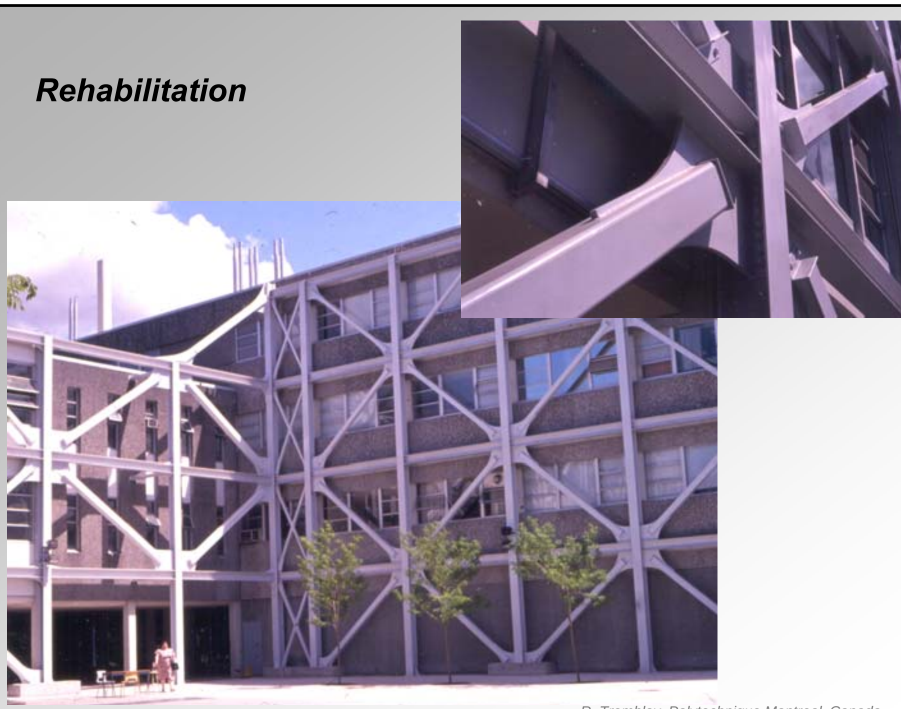
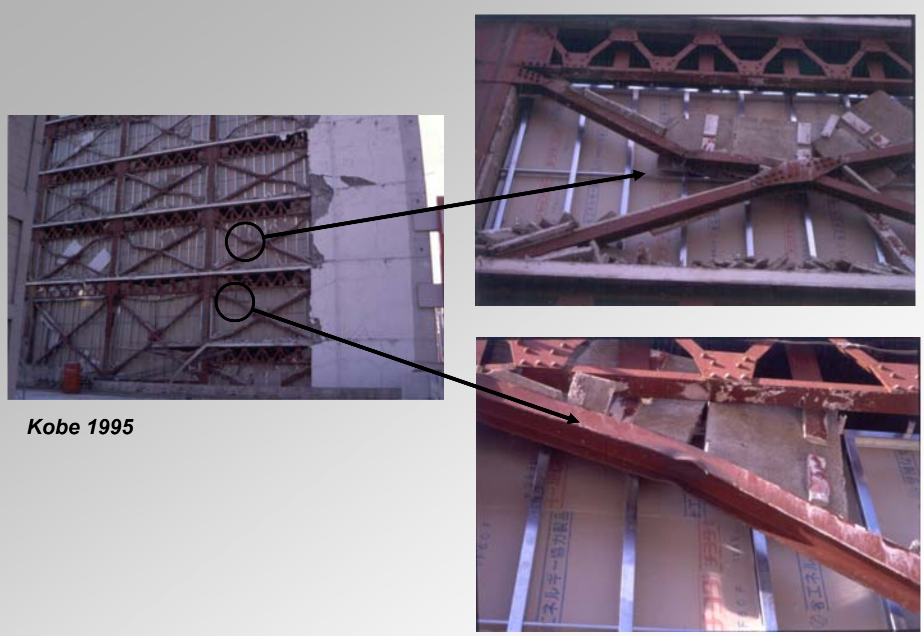
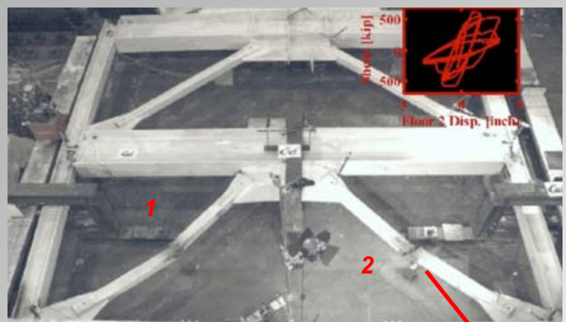
Rehabilitation





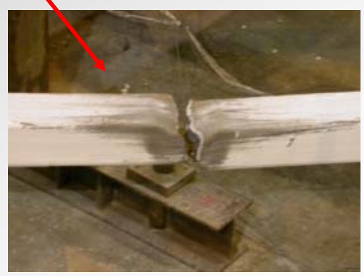
Kobe 1995

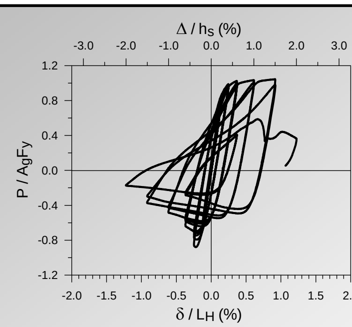
Northridge 1994
Photos from Peter Maranian, Brandow and Associates (P. Uriz Thesis, 2005)



Uriz and Mahin (2004)
Univ. of California, Berkeley

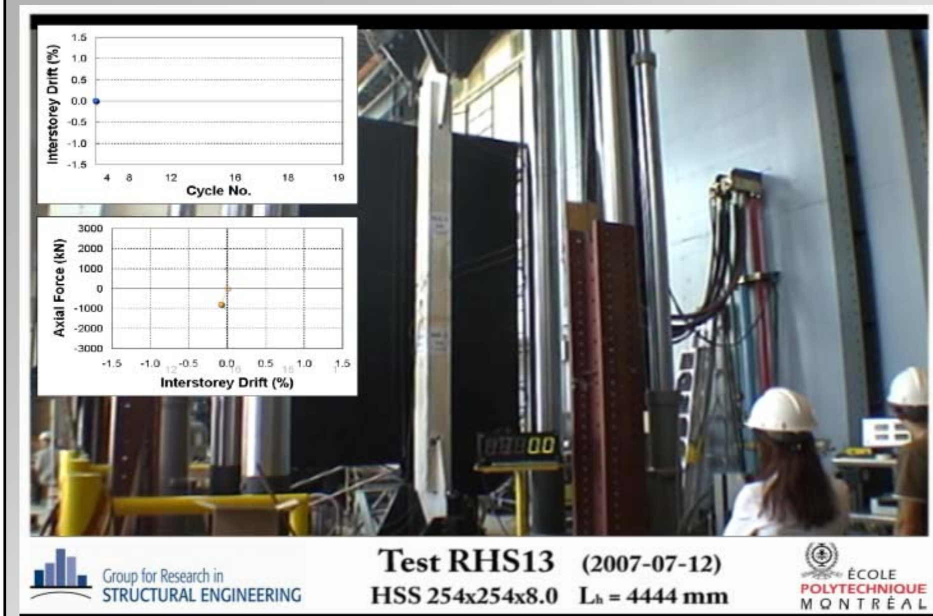
Fracture in 1^{st} cycle at $\Delta_1 \cong 2\% \ h_s$





HSS 254 x 254 x 12

b/t = 18, KL/r = 42


Interstorey Drift (%)
Cycle No.
Axial Force (kN)
Interstorey Drift (%)
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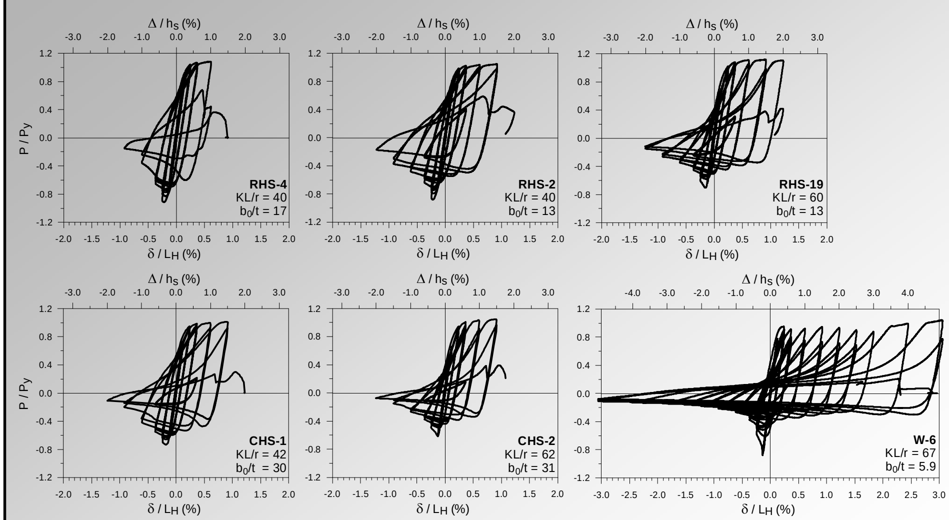
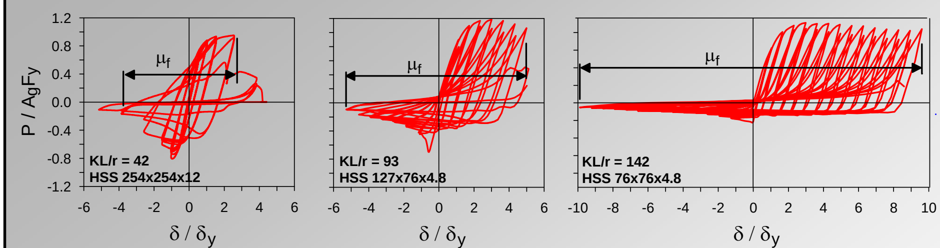
STRUCTURAL ENGINEERING
Test RHS13 (2007-07-12)
HSS 254x254x8.0 Lh = 4444 mm
ÉCOLE
POLYTECHNIQUE
MONTRÉAL

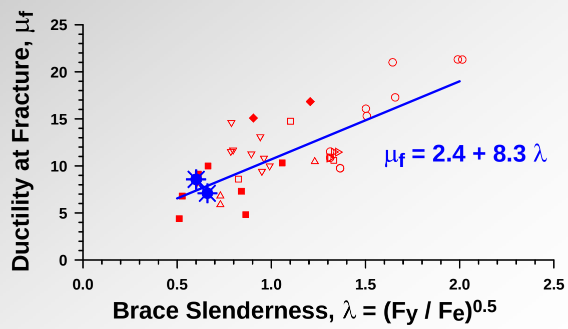

EST


15 Oct 2007
15 Oct 2007


W-1
EST
W-1


Δ / hS (%)
P / Py
δ / LH (%)
RHS-4
KL/r = 40
b0/t = 17
RHS-2
KL/r = 40
b0/t = 13
RHS-19
KL/r = 60
b0/t = 13
CHS-1
KL/r = 42
b0/t = 30
CHS-2
KL/r = 62
b0/t = 31
W-6
KL/r = 67
b0/t = 5.9





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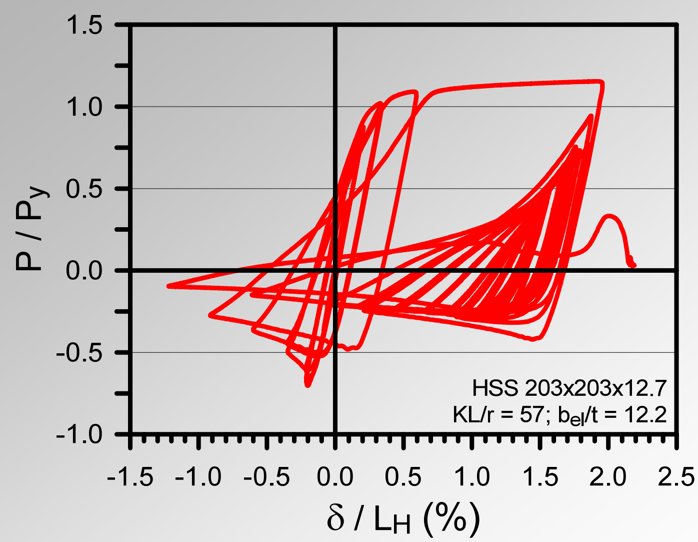

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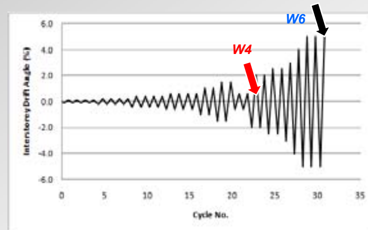
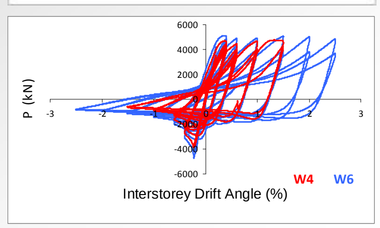
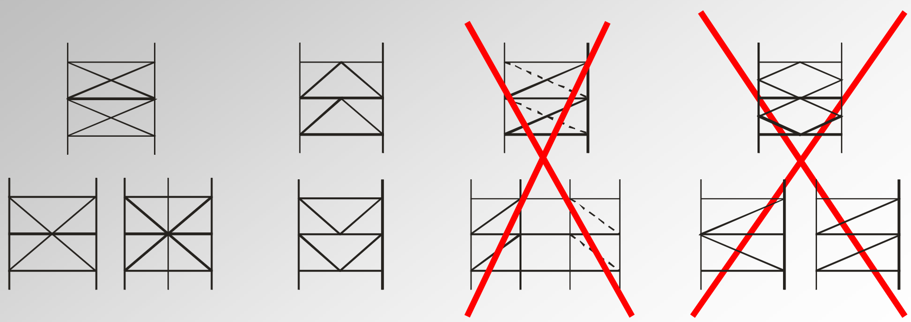
Specimen	W4	W6
Shape	W310x97	W250x115
A_g [mm^2]	12300	14600
KL/r [mm]	60	60
b_0/t [mm]	9.9	5.9





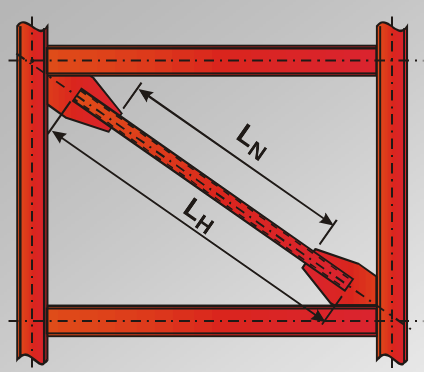
Design – Bracing Configuration

- **Along any braced line, between 30% & 70% of lateral load is resisted by tension braces**
- **Tension-only braced frames <u>not</u> permitted**
- **K-bracing not permitted**



Design – Bracing Members

- Braces must resist gravity + lateral loads
- P_n in tension and compression as per AISC 360-10
- KL/r $\leq$ 200
- Section must meet seismic λ_{hd} limits
- For built-up sections, individual components must meet KL/r limits and stitch subjected to shear under buckling must meet minimum shear strength



$KL_{out} \approx 0.9\ L_H$
$KL_{in} \approx 0.5\ L_N$

$KL_{out} \approx 0.5\ L_H$
$KL_{in} \approx 0.5\ L_N$

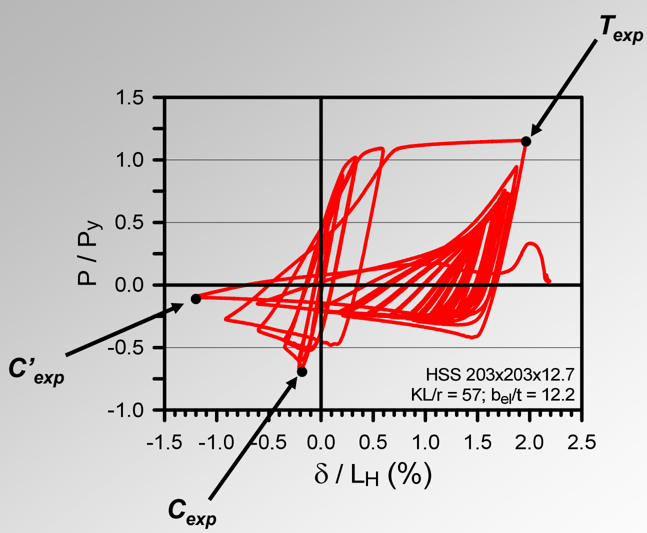
Bracing Configuration

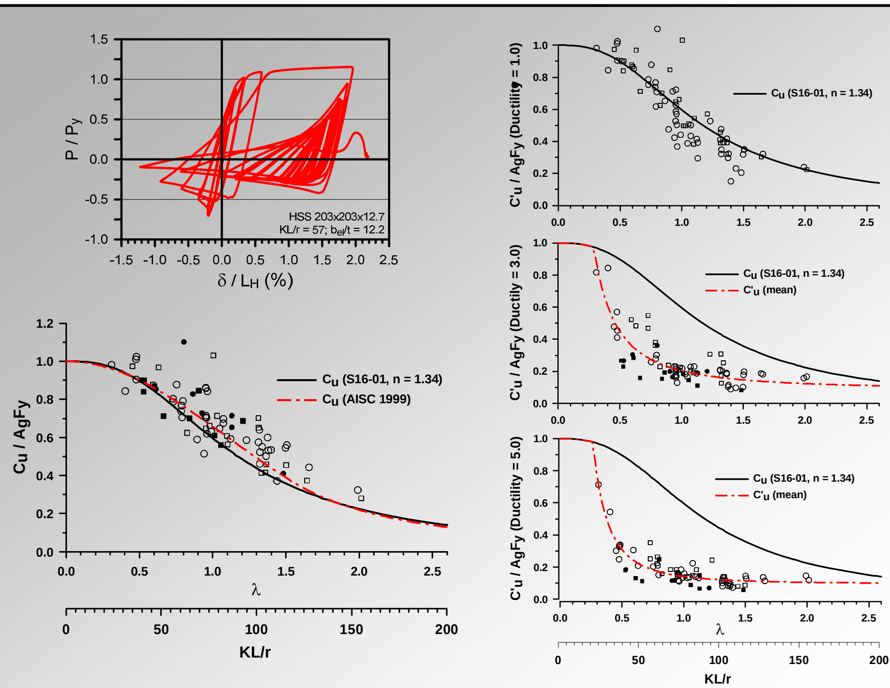
Tension-only braced frames permitted

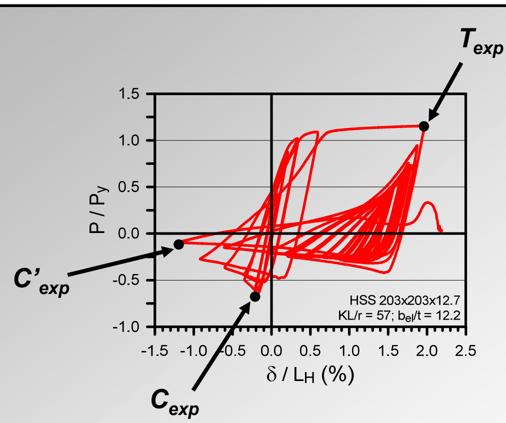
Bracing Members

Section must meets b/t limits that vary with KL/r

Design – Expected Brace Strengths







$T_{exp} = A\ R_y F_y$

$C_{exp} = A\ (1.12\ F_{cr})$ where $F_{cre} = F_{cr}$ with $R_y F_y$
$\leq A\ R_y F_y$

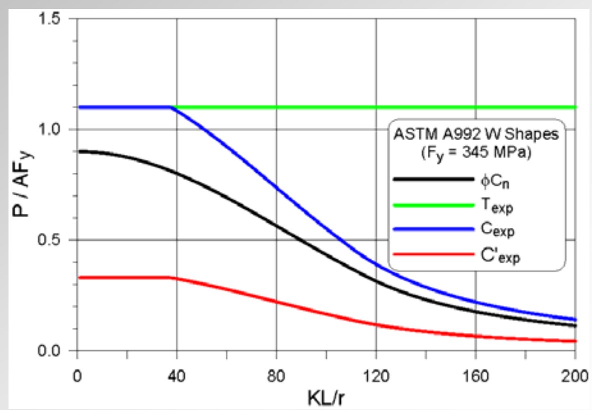
$C'_{exp} = 0.3\ C_{exp}$

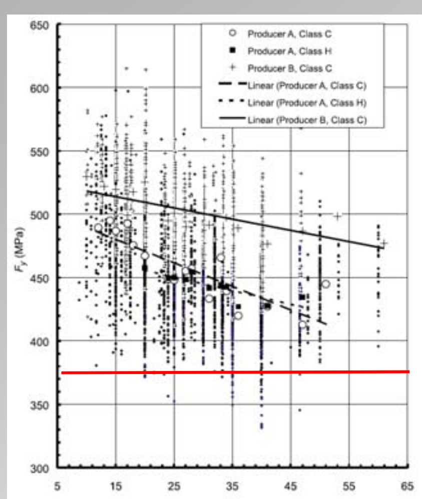
$T_{exp} = A\ R_yF_y$

$C_{exp} = A\ (1.12\ F_{cr})$ $,F_{cre} = F_{cr}$ with R_yF_y

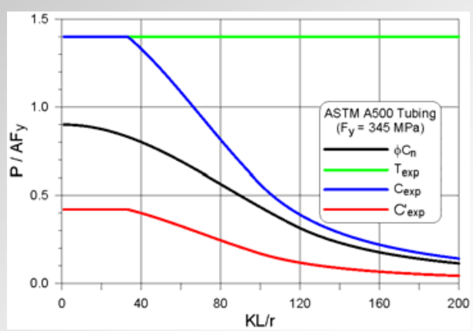
$\leq A\ R_yF_y$

$C'_{exp} = 0.3\ C_{exp}$





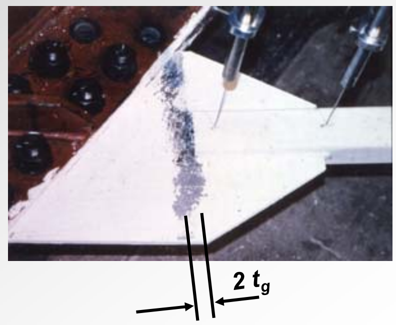
Schimdt and Bratlett (2002)



Design – Brace Connection

Must resist brace T_{exp} & 1.1 C_{exp}

Must allow for ductile rotational behavior or resist 1.1 x brace expected flexural strength

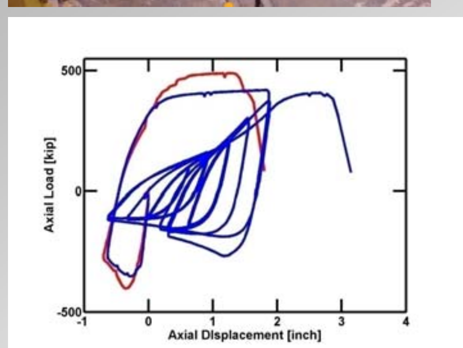


Archambault et al. (1995)
Tremblay and Bolduc (2002)
École Polytechnique,Montreal

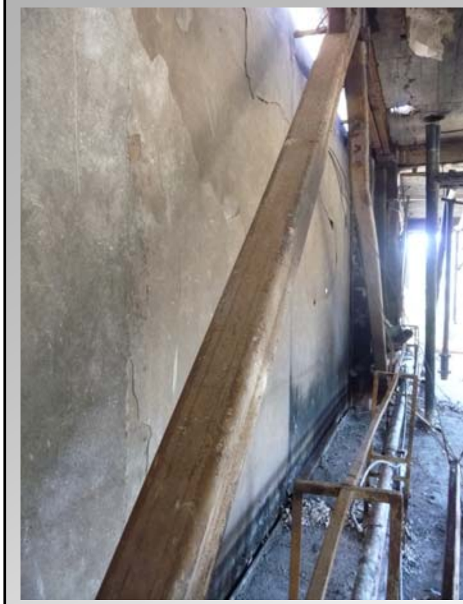
Kobe 1995

R. Tremblay, Polytechnique Montreal, Canada 57

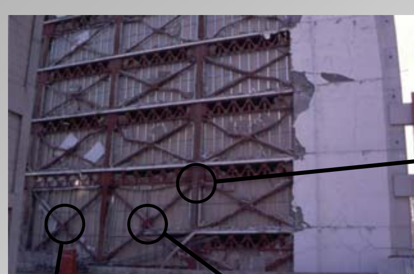
Yang and Mahin (2004)
Univ. of California, Berkeley

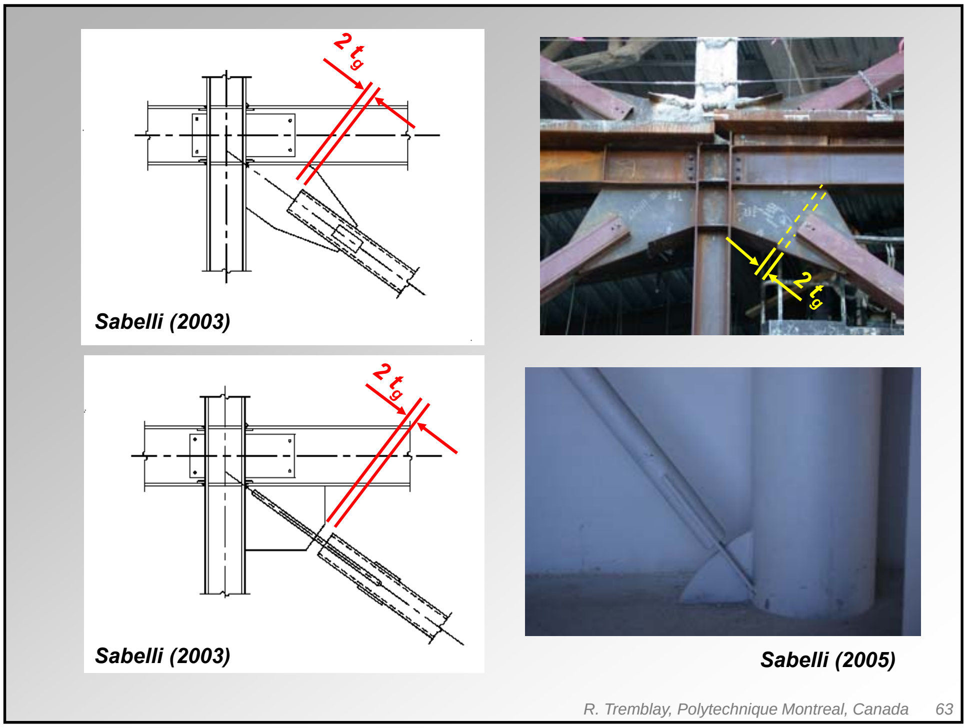


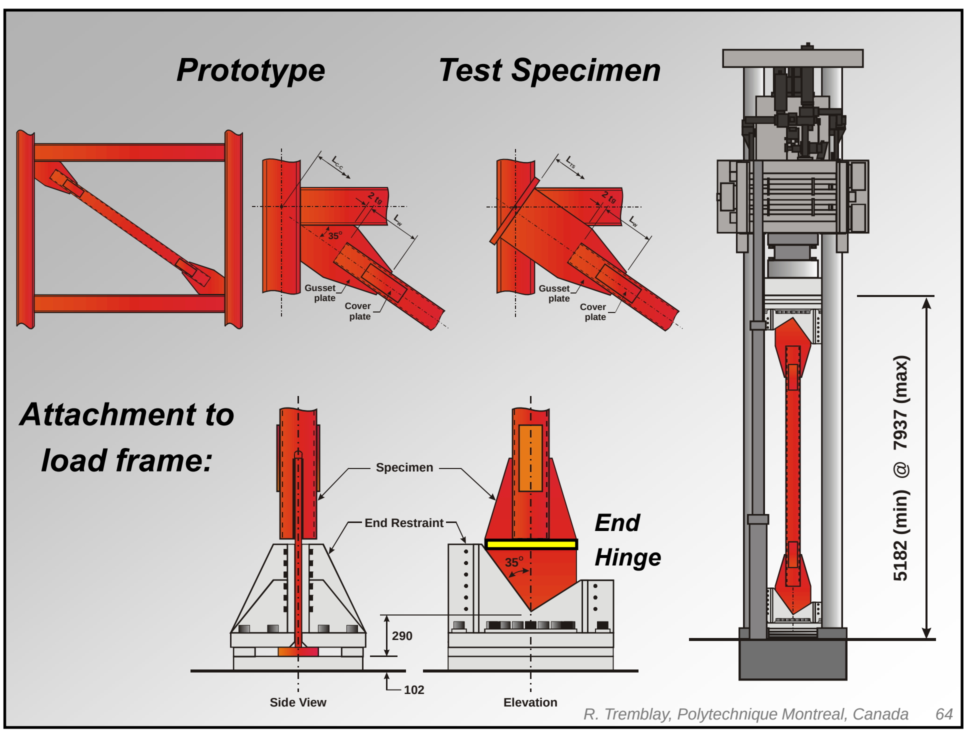
R. Tremblay, Polytechnique Montreal, Canada 58



Kobe 1995




2 t_g
Sabelli (2003)
2 t_g
2 t_g
Sabelli (2003)
Sabelli (2005)
R. Tremblay, Polytechnique Montreal, Canada 63


Prototype
Test Specimen
35°
Gusset plate
Cover plate
Gusset plate
Cover plate
Attachment to load frame:
Specimen
End Restraint
End Hinge
35°
290
102
Side View
Elevation
5182 (min) @ 7937 (max)
R. Tremblay, Polytechnique Montreal, Canada 64

Design – Columns and Beams

Must resist gravity loads plus two brace force scenarios:

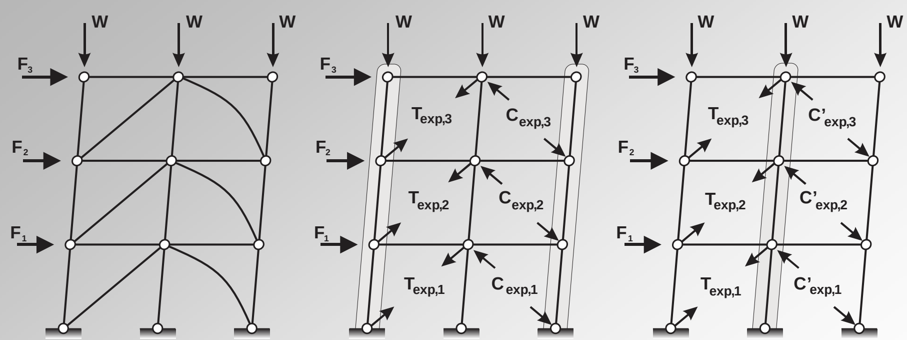
- **Upon first buckling & yielding (T_{exp} & C_{exp})**
- **In post-buckling range (T_{exp} & C'_{exp})**

Beams in V and inverted-V bracing must be continuous between columns

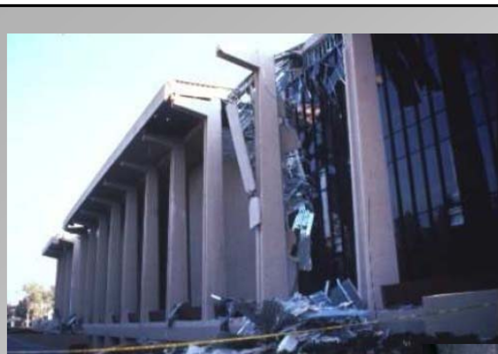
Column sections must meet λ_{hd}

Beam sections must meet λ_{md}

Brace force scenarios for columns:

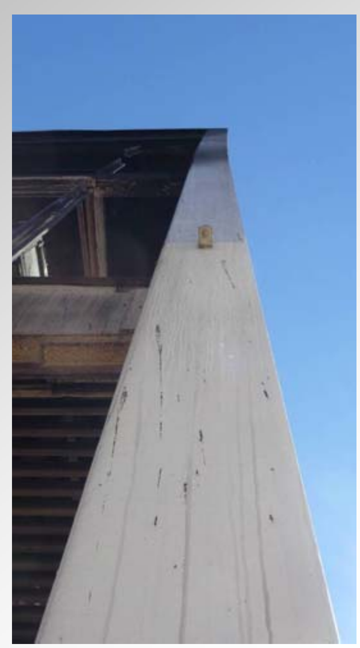


At Buckling ***Post-Buckling***

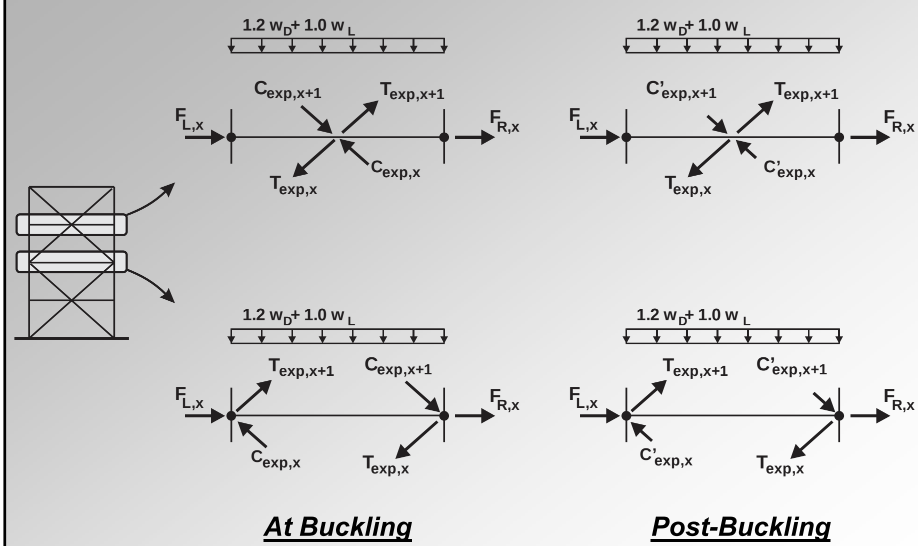


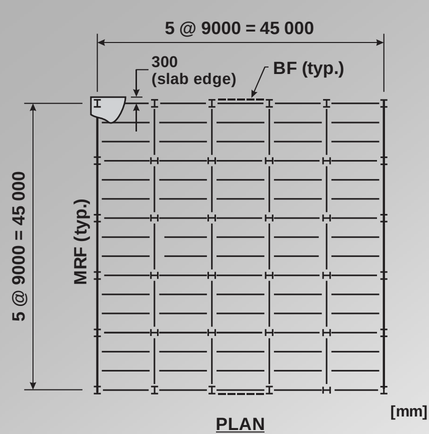
Northridge 1994
Photos from Finley 1999
(P. Uriz Thesis, 2005)

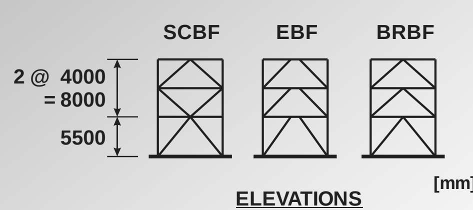
Taiwan 1999



Brace force scenarios for beams:







Gravity loads:
- Roof: Dead = 3.2 kPa
  - Live = 1.0 kPa
- Floor: Dead = 3.5 kPa
  - Partitions = 1.0 kPa
  - Live = 3.8 kPa
- Exterior walls = 1.5 kPa

Seismic Load Data (NCh433):
- Zone 2
- Soil Type C
- A= 0.30 g
- In-plane torsion omitted

Load Combinations:
- 1.2D + 1.6L
- 1.2D + 1.0L + 1.4E
- 0.9D + 1.4E

Seismic weight:
- P = 7720 kN (Level 9)
  - 12635 kN (Levels 2-8)
  - 12840 kN (Level 1)

Steel:
- BRB cores: Fyc = 260-290 MPa
- Other members: Fy = 345 MPa

Note: Redundancy factor, ρ, and seismic load effects with overstrength factor, Ω_0, as specified in ASCE 7-10 are not considered.

NORMA CHILENA OFICIAL **NCh 433.Of1996**
Modificada en 2009

INSTITUTO NACIONAL DE NORMALIZACION • INN-CHILE

Diseño sísmico de edificios

NCh433

...es máximos de los factores de modificación de la respuesta[1]

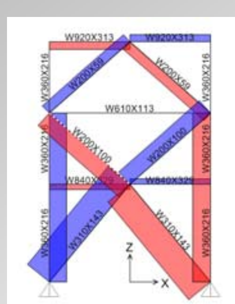
	Material estructural	R	R_o
	Acero estructural		
	a) Marcos corrientes (OMF)	4	5
Pórticos	b) Marcos intermedios (IMF)	5	6
	c) Marcos especiales (SMF)	7	11
	d) Marco de vigas enrejadas (STMF)	6	10
	Hormigón armado	7	11
	Acero estructural		
	a) Marcos concéntricos corrientes (OCBF)	3	5
	b) Marcos concéntricos especiales (SCBF)	5.5	8
	c) Marcos excéntricos (EBF)	6	10
	Hormigón armado	7	11
	Hormigón armado y albañilería confinada		
	- Si se cumple el criterio A[2]	6	9
	- Si no se cumple el criterio A[2]	4	4
Muros y sistemas arriostrados	**Madera**	5,5	7
	Albañilería confinada	4	4

Static method of analysis

$$Q_o = CIP$$

$$C = \frac{2.75 S A_0}{gR}\left(\frac{T'}{T^*}\right)^n$$

I =	1.0		A =	0.30
Zone =	2		P =	16598 kN/ Frame
Soil Type =	C		S =	1.05
R =	5.5		T' =	0.45 s
			n =	1.40
T* =	0.55 s			
	From analysis		C =	0.119
			Q_0 =	1974 kN / Frame



Brace Design

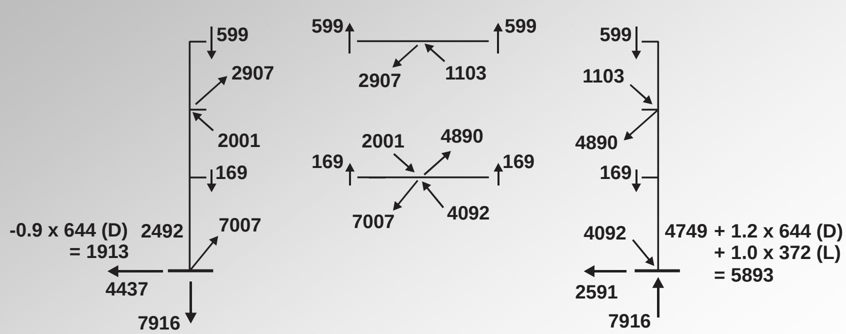
ϕ_c =	0.90
F_y =	0.345 kN/mm^2
R_yF_y =	0.385 kN/mm^2
λ_r for b/2t =	7.2
λ_r for h/t_w =	35.9
Limit KL/r =	200

KL	θ	P_D	P_L	P_E	$P_{u,1}$	$P_{u,2}$	P_u	Braces Shape	A_g	r_y	b/2t	h/t_w	KL/r
(mm)	(deg)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)		(mm^2)	(mm)	( )	( )	(mm)
6021	41.6	29	4	555	815	41	815	W200X59	7550	51.8	7.22	19.9	116.2
6021	41.6	87	51	930	1458	186	1458	W200X100	12700	53.8	4.43	12.5	111.9
7106	50.7	155	73	1559	2441	303	2441	W310X143	18200	78.5	6.77	19.8	90.5
		From Analysis											

Braces											Expected Brace Capacities			
Shape	A_g	r_y	b/2t	h/t_w	KL/r	F_e	F_{cr}	ϕP_n	$P_u/\phi P_n$	Check	T_{exp}	$F_{cr,Ry}$	C_{exp}	C'_{exp}
	(mm^2)	(mm)	( )	( )	(mm)	(kN/mm^2)	(kN/mm^2)	(kN)	(kN)		(kN)	(kN/mm^2)	(kN)	(kN)
W200X59	7550	51.8	7.22	19.9	116.2	0.146	0.128	871	0.94	OK	2907	0.128	1103	331
W200X100	12700	53.8	4.43	12.5	111.9	0.158	0.138	1578	0.92	OK	4890	0.138	2001	600
W310X143	18200	78.5	6.77	19.8	90.5	0.241	0.189	3103	0.79	OK	7007	0.197	4092	1227

Column Design

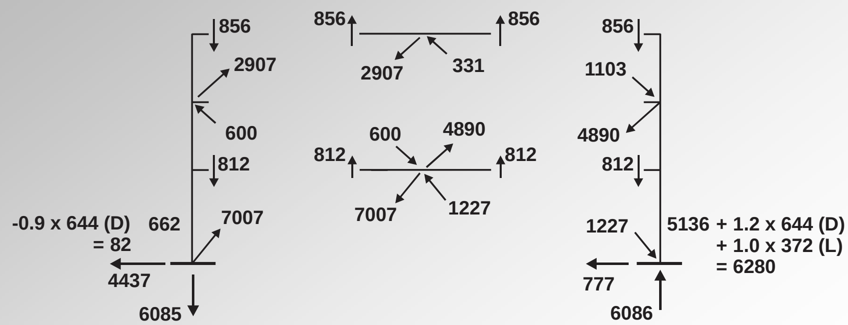
Expected Brace Capacities			
T_{exp}	$F_{cr,Ry}$	C_{exp}	C'_{exp}
(kN)	(kN/mm^2)	(kN)	(kN)
2907	0.128	1103	331
4890	0.138	2001	600
7007	0.197	4092	1227



At Buckling

Column Design

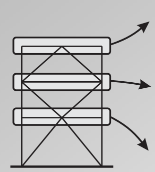
Expected Brace Capacities			
T_{exp} (kN)	$F_{cr,Ry}$ (kN/mm^2)	C_{exp} (kN)	C'_{exp} (kN)
2907	0.128	1103	331
4890	0.138	2001	600
7007	0.197	4092	1227

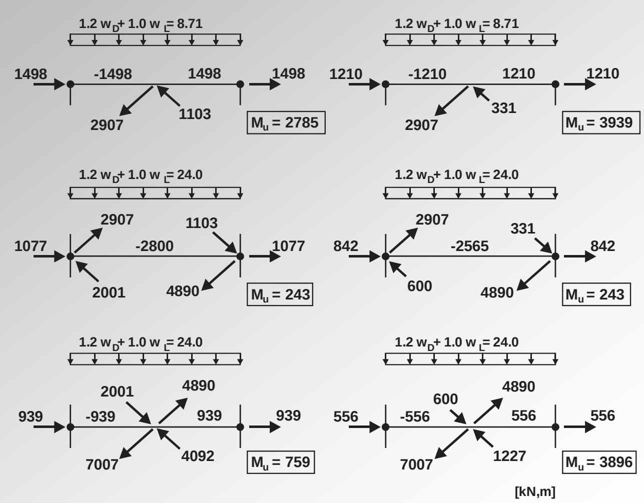


Post-Buckling

Beam Design

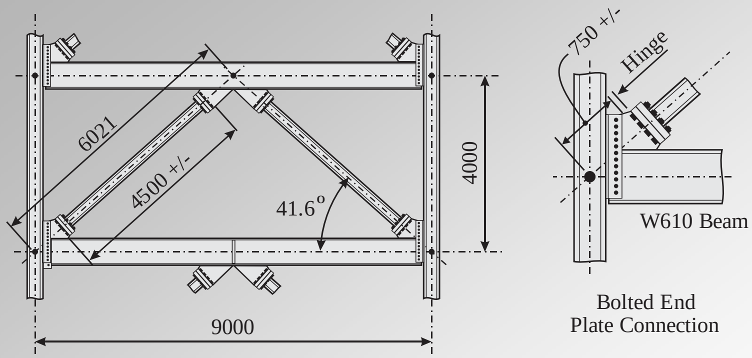
Expected Brace Capacities			
T_{exp} (kN)	$F_{cr,Ry}$ (kN/mm^2)	C_{exp} (kN)	C'_{exp} (kN)
2907	0.128	1103	331
4890	0.138	2001	600
7007	0.197	4092	1227





At Buckling ***Post-Buckling***

Consider more realistic brace KL

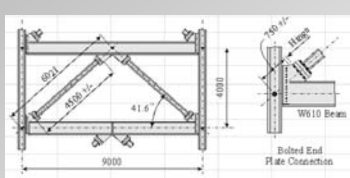


Brace sizes are reduced, increasing T*, reducing seismic loads …

T* = 0.55 s -> 0.65 s
C = 0.119 -> 0.093   (22% decrease in seismic loads)

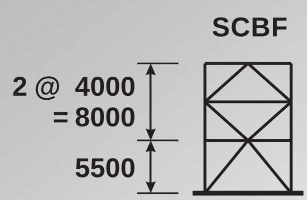
ϕ_c =	0.90	
F_y =	0.345	kN/mm^2
R_yF_y =	0.385	kN/mm^2
λ_r for b/2t =	7.2	
λ_r for h/t_w =	35.9	
Limit KL/r =	200	

								Braces					
KL	θ	P_D	P_L	P_E	$P_{u,1}$	$P_{u,2}$	P_u	Shape	A_g	r_y	b/2t	h/t_w	KL/r
(mm)	(deg)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)		(mm^2)	(mm)	( )	( )	(mm)
6021	41.6	29	4	555	815	41	815	W200X59	7550	51.8	7.22	19.9	116.2
6021	41.6	87	51	930	1458	186	1458	W200X100	12700	53.8	4.43	12.5	111.9
7106	50.7	155	73	1559	2441	303	2441	W310X143	18200	78.5	6.77	19.8	90.5
		From Analysis											



ϕ_c =	0.90	
F_y =	0.345	kN/mm^2
R_yF_y =	0.385	kN/mm^2
λ_r for b/2t =	7.2	
λ_r for h/t_w =	35.9	
Limit KL/r =	200	

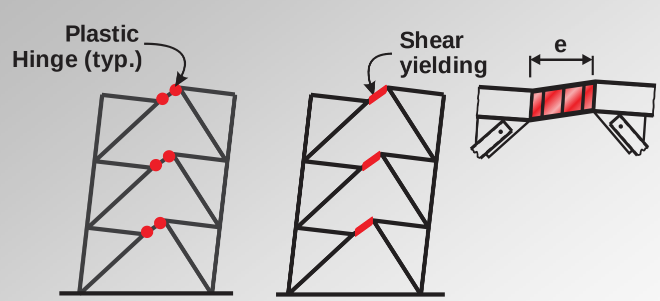
								Braces					
KL	θ	P_D	P_L	P_E	$P_{u,1}$	$P_{u,2}$	P_u	Shape	A_g	r_y	b/2t	h/t_w	KL/r
(mm)	(deg)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)		(mm^2)	(mm)	( )	( )	(mm)
4500	41.63	29	4	439	653	41	653	W200X59	7550	51.8	7.22	19.9	86.9
4500	41.63	66	40	736	1150	143	1150	W200X59	7550	51.8	7.22	19.9	86.9
5600	50.71	136	65	1234	1955	267	1955	W250X101	12800	65.8	6.56	18.9	85.1
Fixed		From Analysis											



	SCBF	SCBF
Brace KL	c/c	L_H
R =	5.5	5.5
T (s) =	0.55	0.65
Q_0 (kN) =	1974	1562
Steel (t) =	16.3	13.7
Q_{exp} (kN) =	7028	5090

Eccentrically Braced Frames

Energy is dissipated through shear or flexure in link beams



Connections and other members (including beams outside links) expected to remain essentially elastic

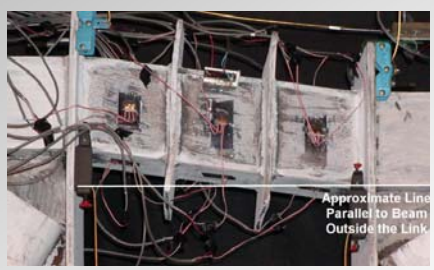
Filiatrault et al.

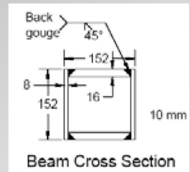
Residential buildings
Montreal
Martoni Cyr

Hockey Canadian, Montreal

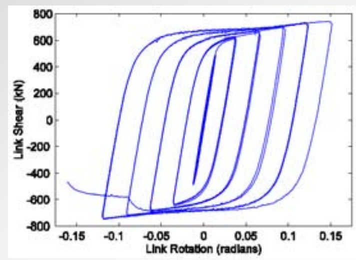
Wellington, New-Zealand 2013

Built-up rectangular tubular link beams



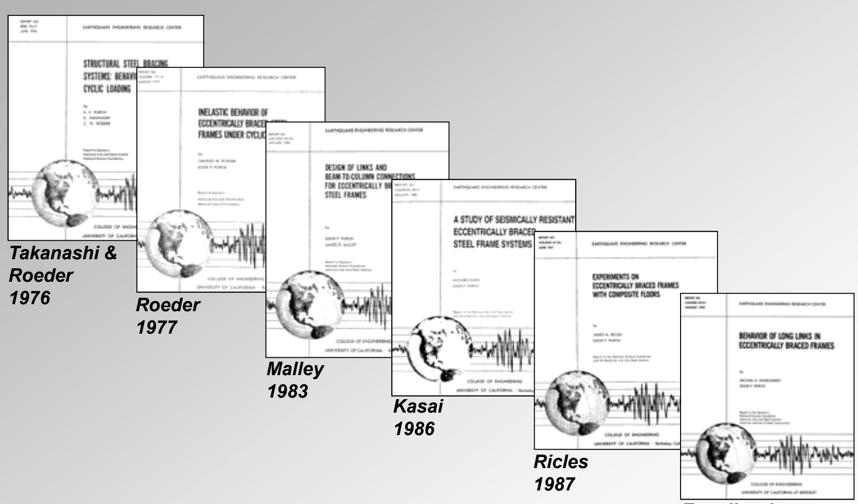


Contreventement non requis pour le segment ductile !



Berman, J.W, and Bruneau, M. 2008. Tubular Links for Eccentrically Braced Frames I: Finite Element Parametric Study. ASCE J. Struct. Eng., Vol. 134, No. 5, pp. 692-701.

System originally developed by Prof. Povov with …



Takanashi & Roeder 1976

Roeder 1977

Malley 1983

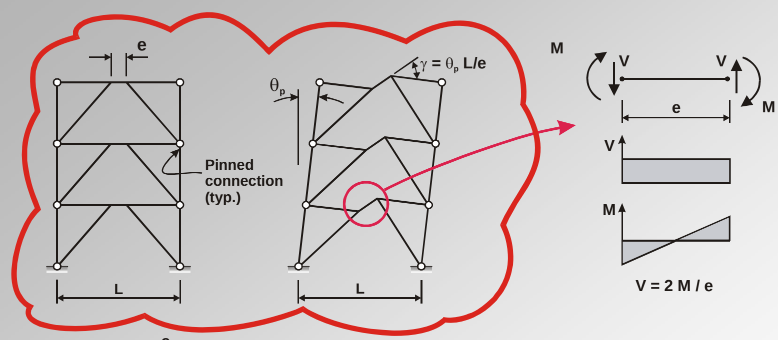
Kasai 1986

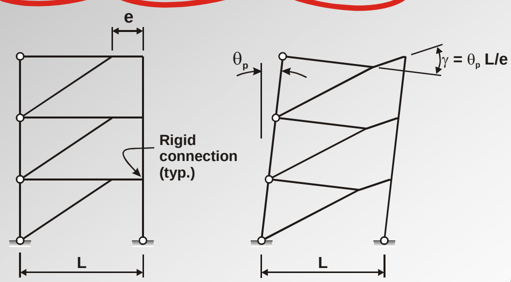
Ricles 1987

Engelhardt 1989

+ *others*

Design – Bracing Configuration





- ***Symmetrical***
- ***$P_f \cong 0$ in ductile links***
- ***Pinned beam-to-column joints***
- ***Shorter beam spans***
- ***Good clearance***

Design – Link Beams

Resistance can be governed by shear (short links) or flexure (long links). Resistances affected by axial load:

$$V_p = 0.6 F_y A_{lw} \text{ for } P_r/P_c \le 0.15$$
$$V_p = 0.6 F_y A_{lw} \sqrt{1-(P_r/P_c)^2} \quad \text{for } P_r/P_c > 0.15$$

$$M_p = F_y Z \text{ for } P_r/P_c \le 0.15$$
$$M_p = F_y Z\left(\frac{1-P_r/P_c}{0.85}\right) \text{for } P_r/P_c > 0.15$$

Section must meet λ_{hd} (λ_{md} for flanges if e $\le$ 1.6 M_p/V_p)

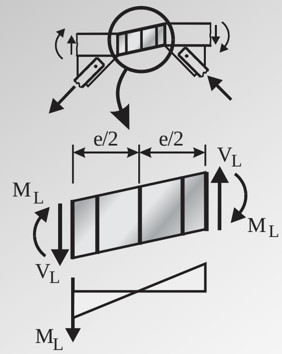
e $\ge$ d; upper limit on e applies in presence of axial load

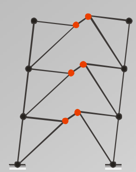
Must meet limits on plastic rotation (γ_p): from 0.02 rad for e $\le$ 1.6 M_p/V_p to 0.08 rad for e > 2.6 M_p/V_p

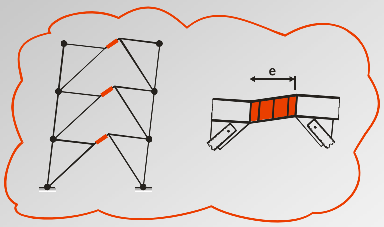
Need for end and intermediate stiffeners; depend on yielding mechanism & γ_p

For center links (M = 0 at e/2):

For shear yielding: $V_n = V_p$
For flexural yielding: $V_n = 2\,M_p/e$

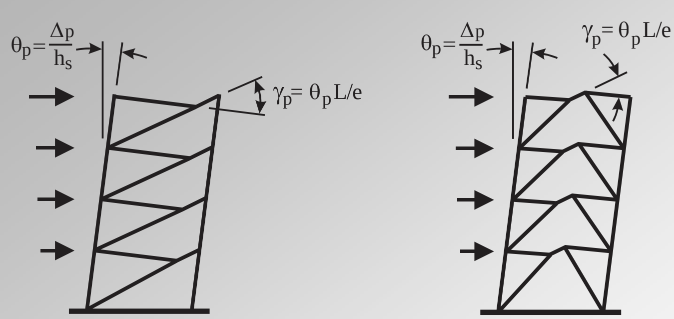






Shorter links generally preferred:

- ***Higher lateral frame stiffness***
- ***Higher energy dissipation capacity***
- ***Less stringent λ (λ_{md} for flanges)***
- ***Easier to design beams outside links***



The inelastic *link rotation angle* shall be determined from the inelastic portion of the *design story drift*. Alternatively, the inelastic link rotation angle is permitted to be determined from nonlinear analysis as defined in Section C3.

Design – Adjusted link shear strength

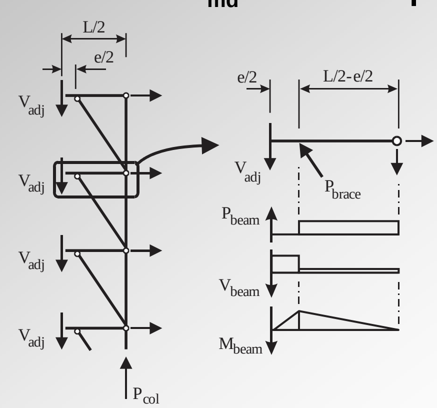
V_{adj} = 1.25 R_yV_n (I-shaped links)
= 1.40 R_yV_n (built-up tubular links)

May be multiplied by 0.88 for beams outside links and for columns in structures more than 3 storeys

Design – Columns and Beams

Must resist gravity loads plus V_{adj}

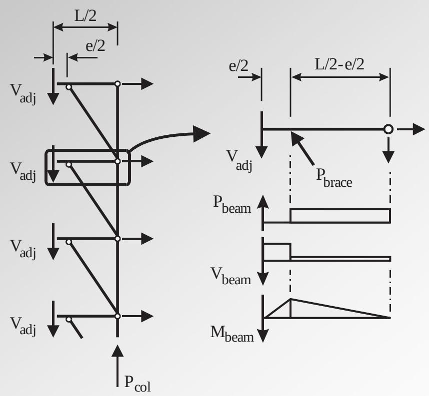
Column sections must meet λ_{hd}. Beam outside links and braces must meet λ_{md} section requirements



Design – Columns and Beams

Must resist gravity loads plus V_{adj}

Column sections must meet λ_{hd}. Beam outside links and braces must meet λ_{md} section requirements



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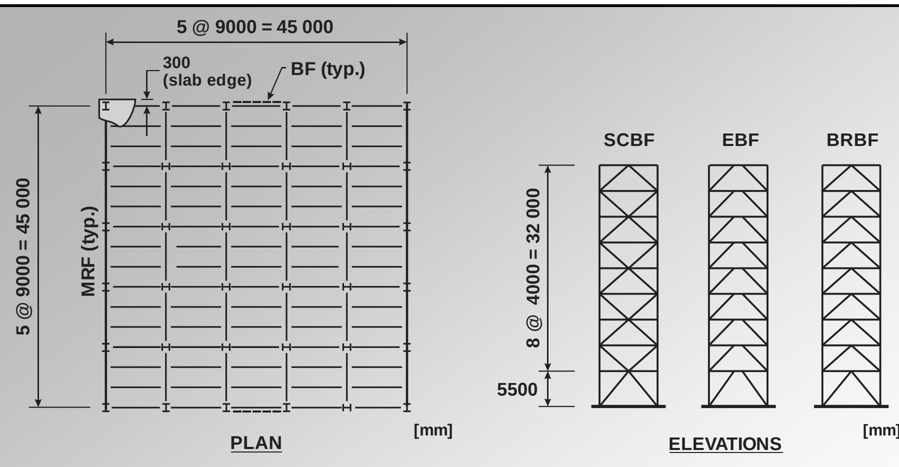

Gravity loads:
- Roof: Dead = 3.2 kPa
  - Live = 1.0 kPa
- Floor: Dead = 3.5 kPa
  - Partitions = 1.0 kPa
  - Live = 3.8 kPa
- Exterior walls = 1.5 kPa

Seismic Load Data (NCh433):
- Zone 2
- Soil Type C
- A= 0.30 g
- In-plane torsion omitted

Load Combinations:
- 1.2D + 1.6L
- 1.2D + 1.0L + 1.4E
- 0.9D + 1.4E

Seismic weight:
- P = 7720 kN (Level 9)
  - 12635 kN (Levels 2-8)
  - 12840 kN (Level 1)

Steel:
- BRB cores: Fyc = 260-290 MPa
- Other members: Fy = 345 MPa

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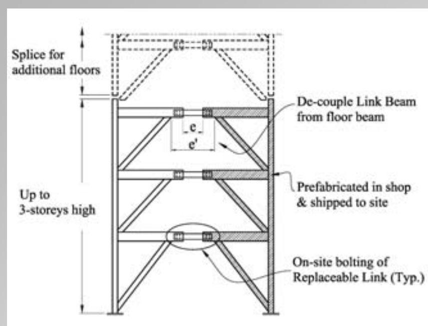
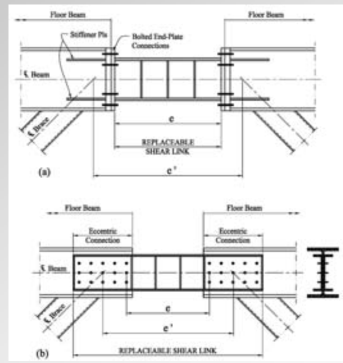
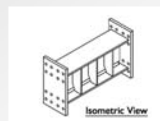
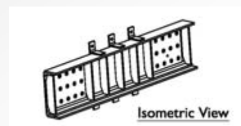
EBFs – Modular Links

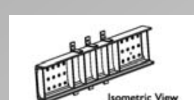
with N. Mansour & C. Christopoulos, Univ. of Toronto

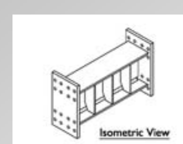


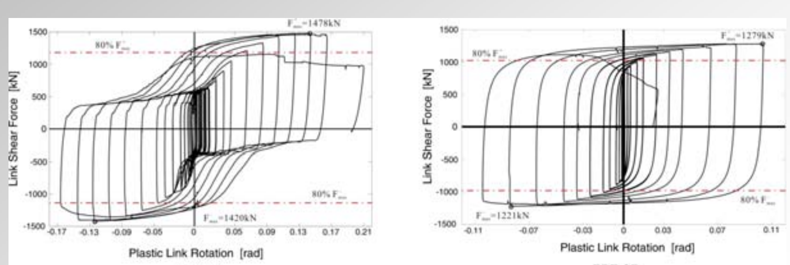


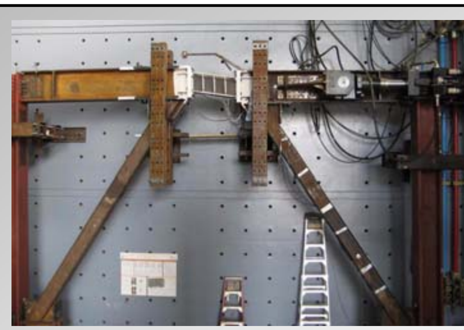


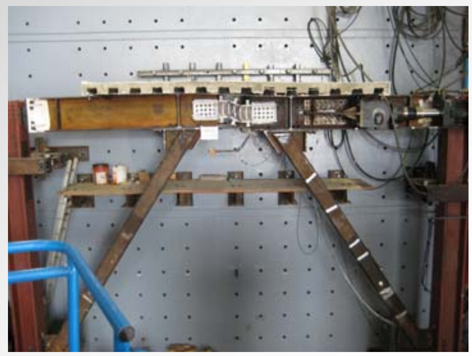


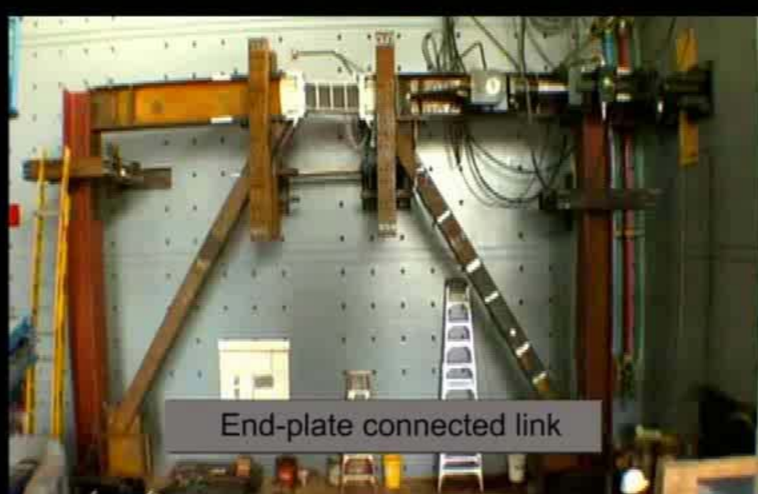


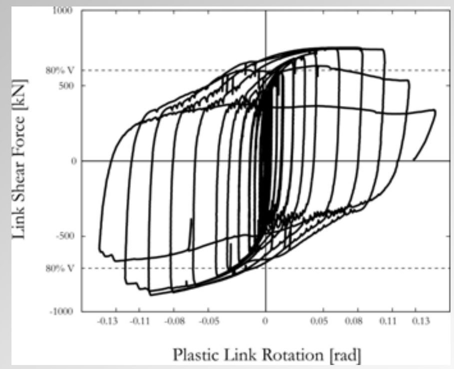









End-plate connected link



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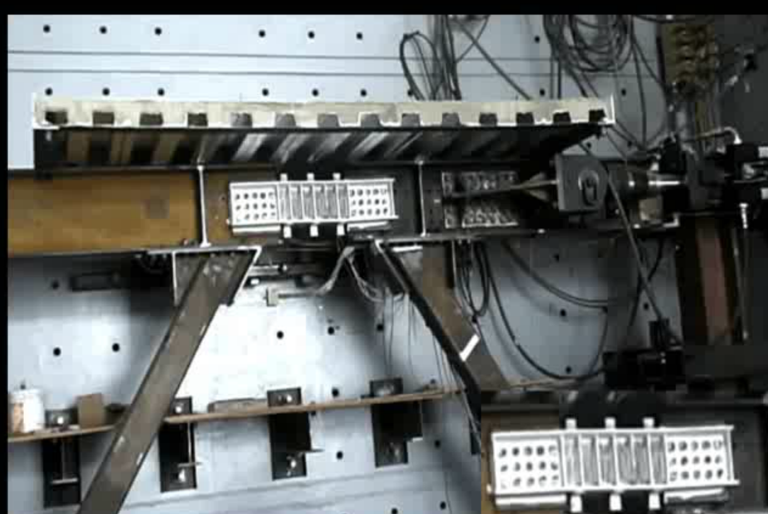
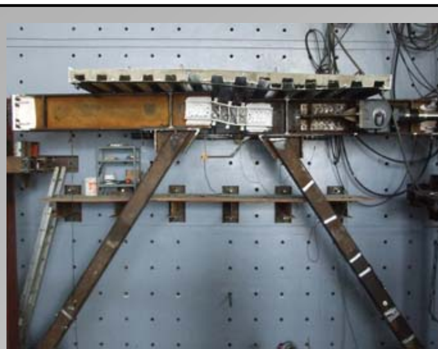
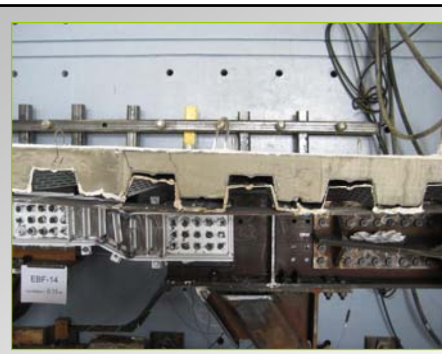
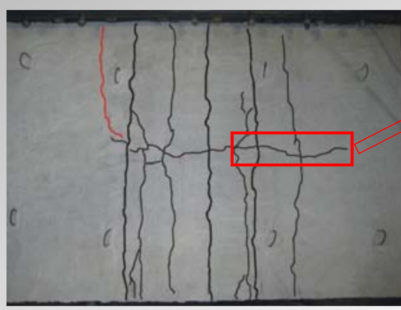


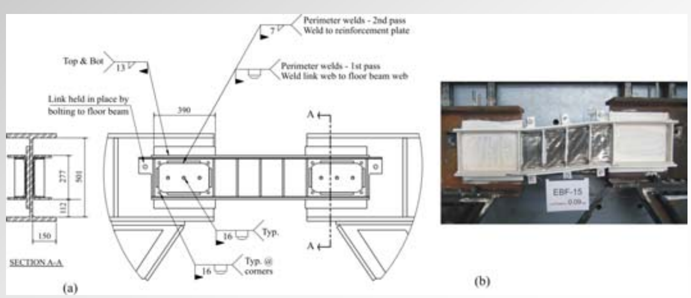




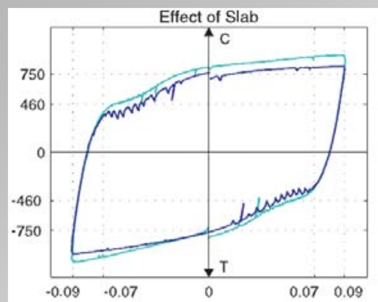


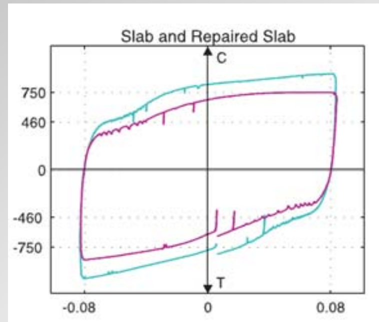
Repair using Self-compacting concrete (Sikacrete-08 SCC)

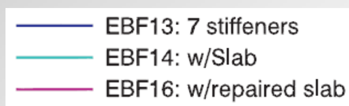
Repair of main transverse surface cracks using Low-viscosity injection resin (Sikadur 52)



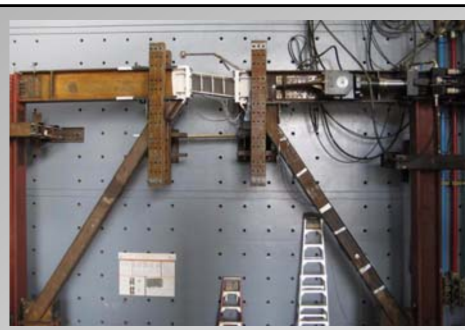
Comparisons for cycles at 0.08 rad.:

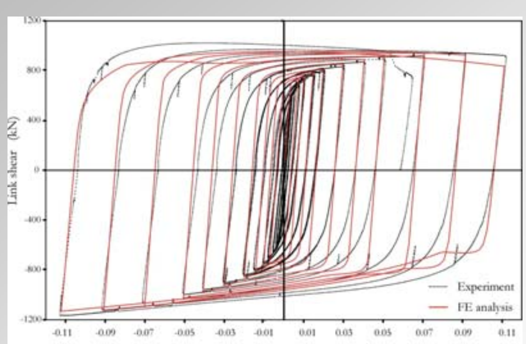


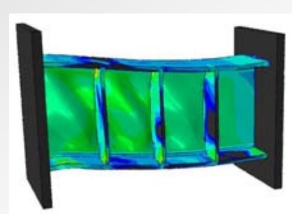




(¼" reinforcement plates)
(3/16" reinforcement plates)

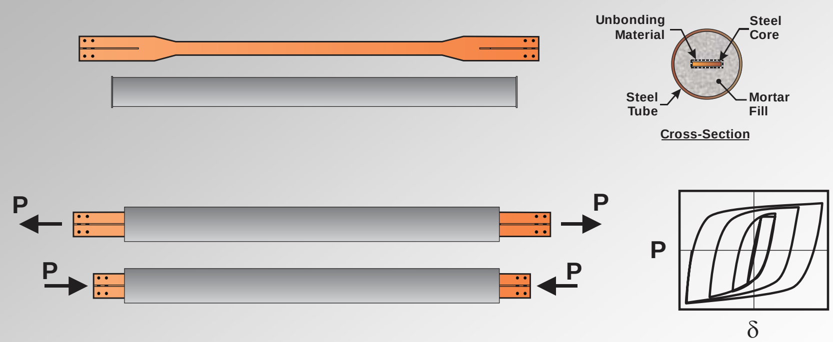




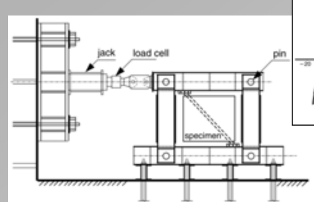


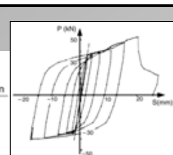
Buckling Restrained Braced Frames

Energy dissipated in bracing members through tensile and compression axial yielding

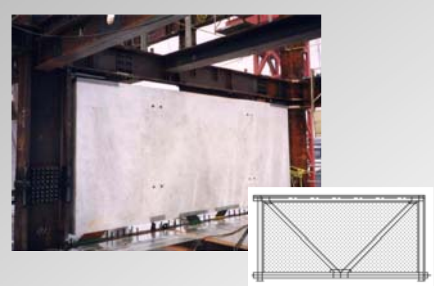


Connections and other members expected to remain essentially elastic

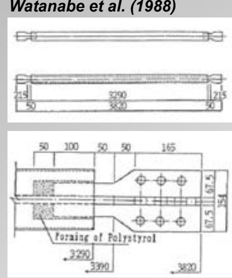
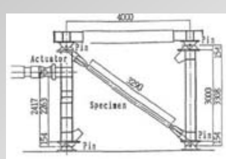
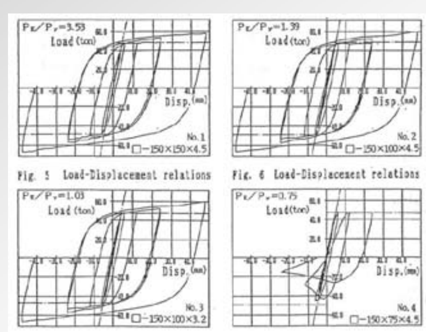




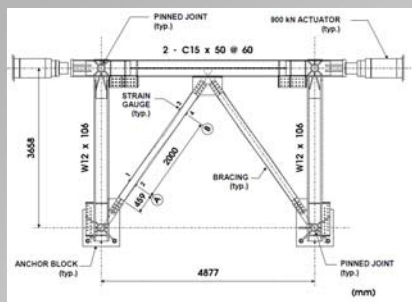
Wakabayashi et al. (1973)

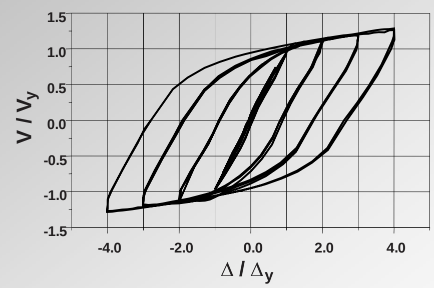


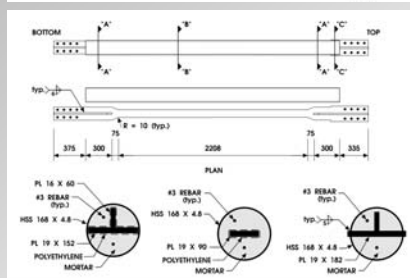
Watanabe et al. (1988)

Buckling Restrained Braced Frames







École Polytechnique 1998

Québec City (1999)

Québec City (1999)

Montreal (Canam)

Vancouver (RJC)

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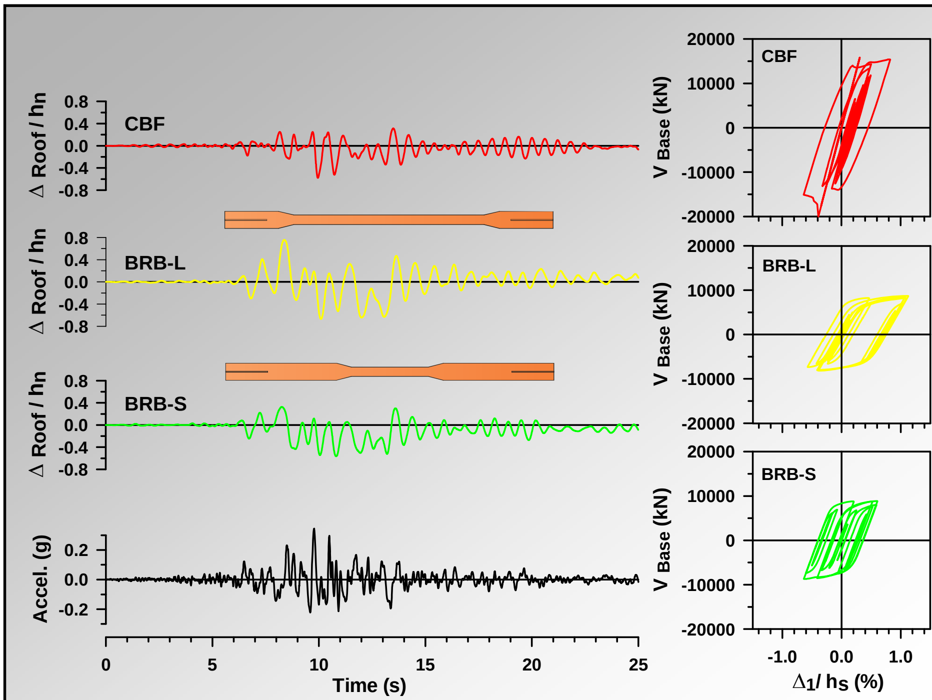

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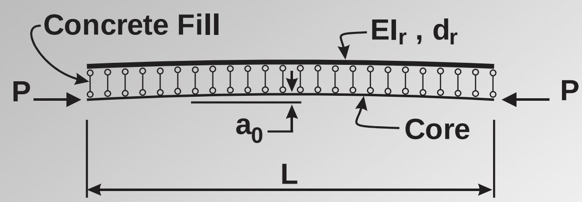
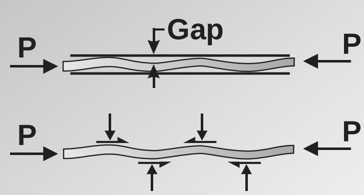
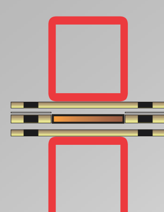
Global buckling:

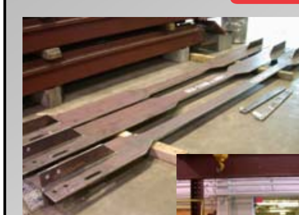


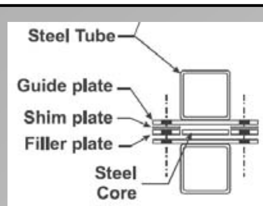
Local (core) buckling:

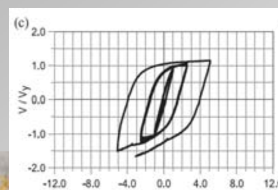


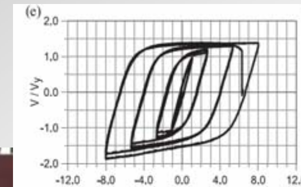
All-steel BRBs

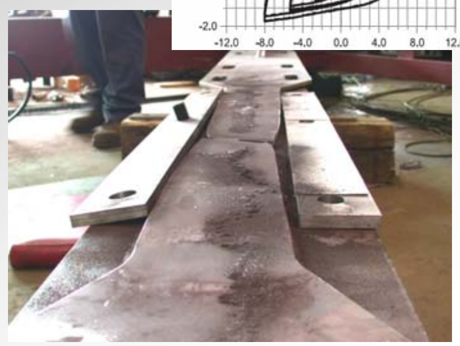












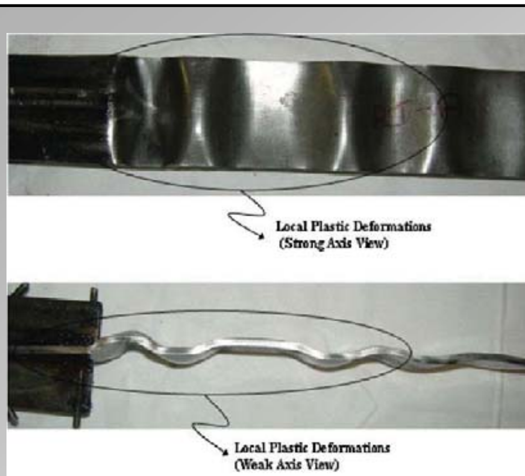
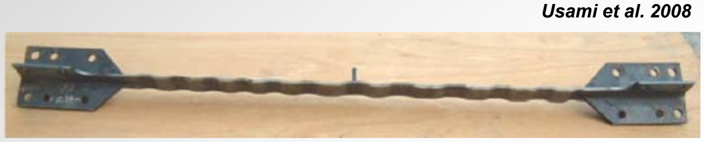
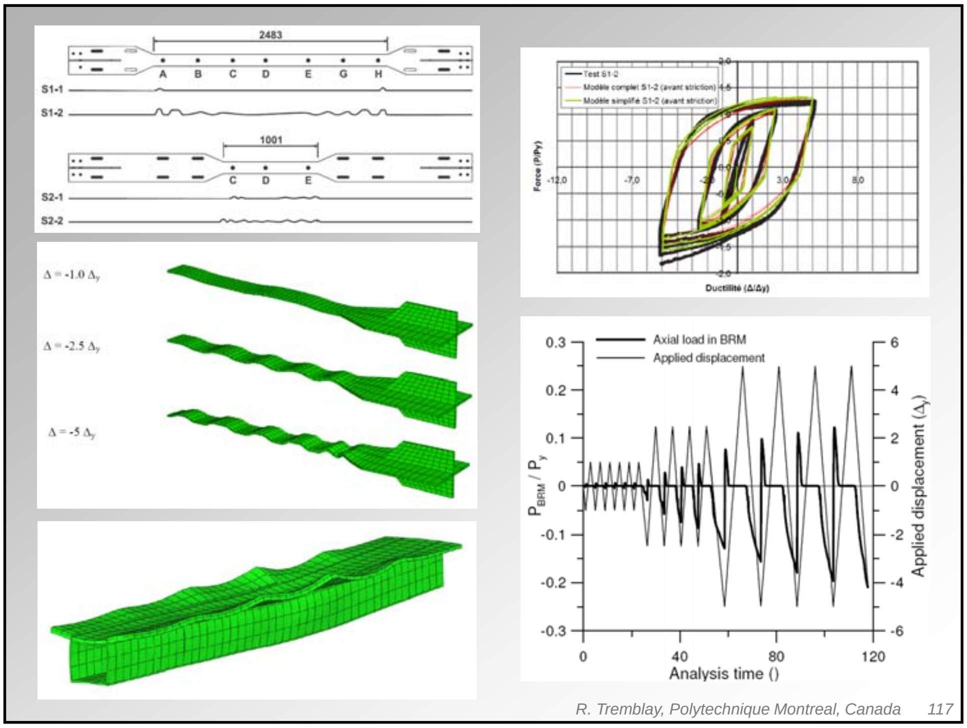


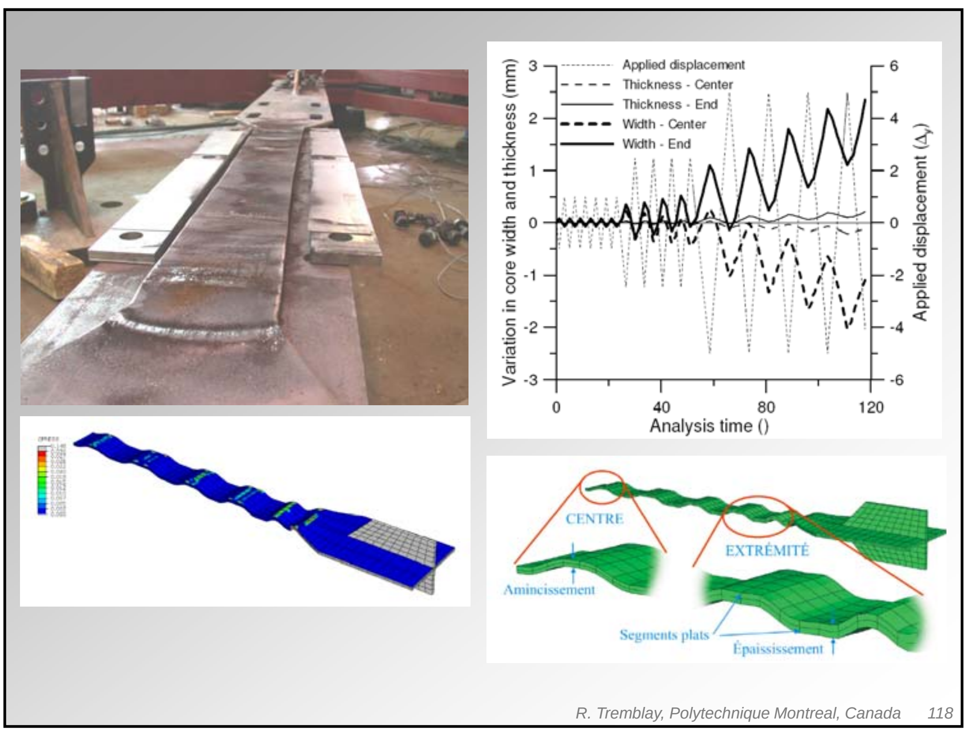
Figure 5. Buckling and local deformation patterns of specimens.

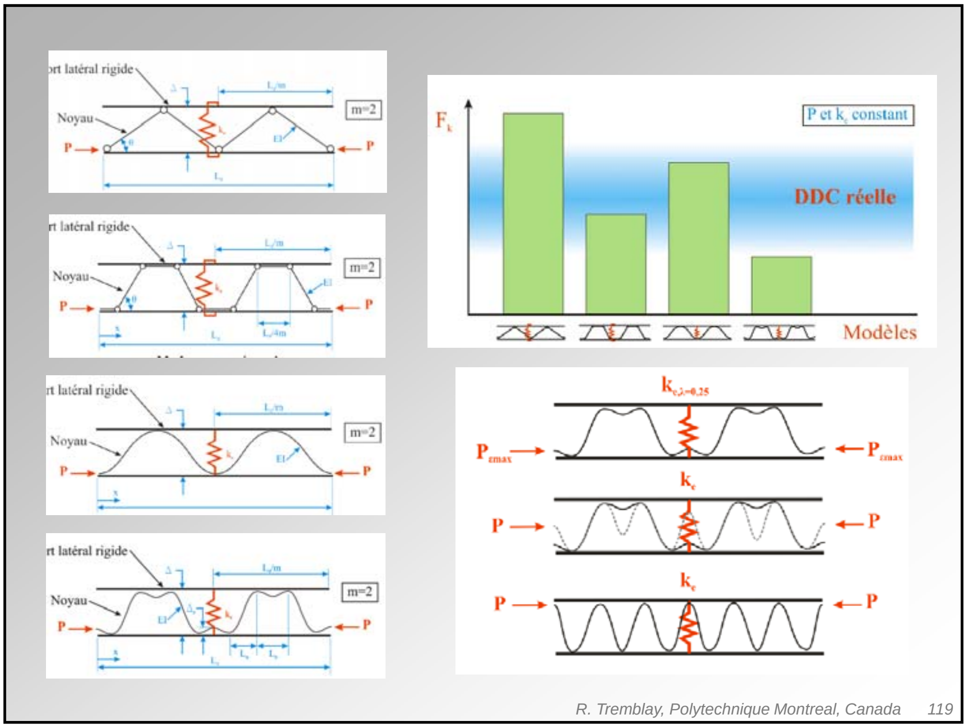
Eryasar & Topkaya 2010

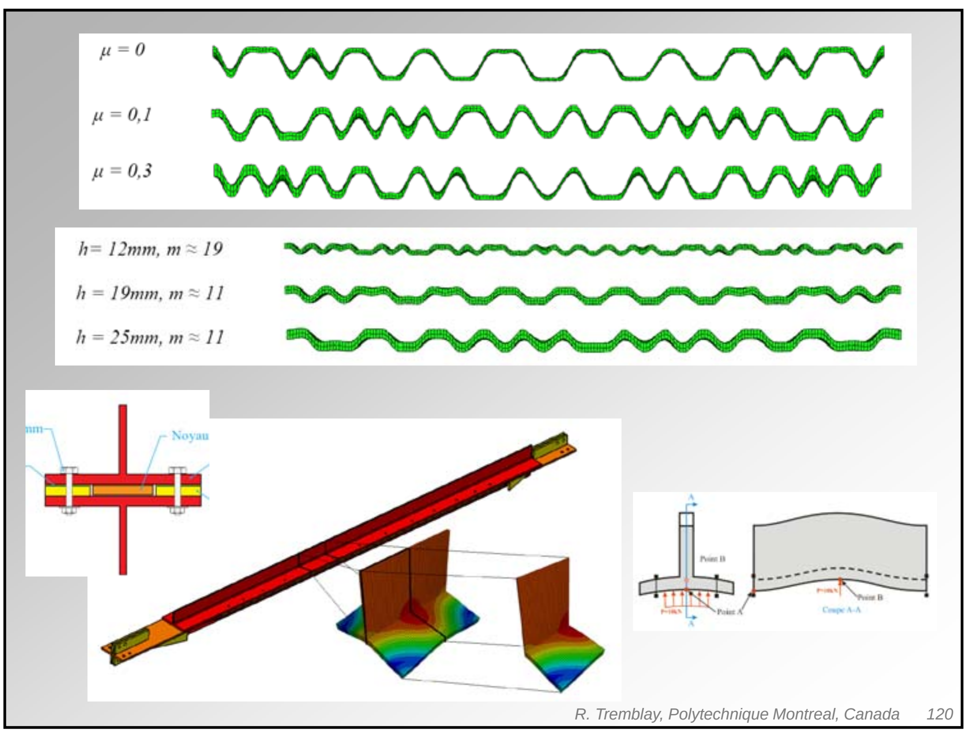
Usami et al. 2008




2483
A B C D E G H
S1-1
S1-2
1001
C D E
S2-1
S2-2
Δ = -1.0 Δy
Δ = -2.5 Δy
Δ = -5 Δy
Test S1-2
Modèle complet S1-2 (avant striction)
Modèle simplifié S1-2 (avant striction)
Force (P/Py)
Ductilité (Δ/Δy)
Axial load in BRM
Applied displacement
P_BRM / P_y
Applied displacement (Δy)
Analysis time ()
R. Tremblay, Polytechnique Montreal, Canada 117


Applied displacement
Thickness - Center
Thickness - End
Width - Center
Width - End
Variation in core width and thickness (mm)
Applied displacement (Δy)
Analysis time ()
CENTRE
EXTRÉMITÉ
Amincissement
Segments plats
Épaississement
R. Tremblay, Polytechnique Montreal, Canada 118


rt latéral rigide
Noyau
P
m=2
P et k constant
DDC réelle
Modèles
R. Tremblay, Polytechnique Montreal, Canada 119


μ = 0
μ = 0,1
μ = 0,3
h= 12mm, m ≈ 19
h = 19mm, m ≈ 11
h = 25mm, m ≈ 11
Noyau
R. Tremblay, Polytechnique Montreal, Canada 120

Specialized suppliers (U.S.)

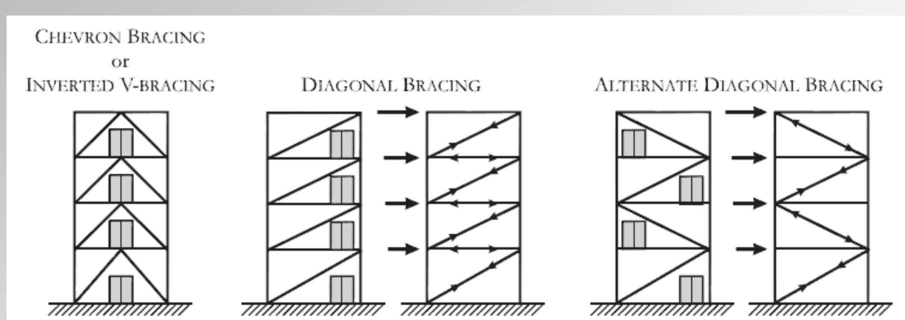
http://www.corebrace.com/

http://www.starseismic.net/

http://www.unbondedbrace.com/

Design – Bracing Configuration

- **No special requirements (BRB have nearly symmetrical response)**
- **May be controlled by drift limit or available BRB capacities**



Design – BRB members

Select A_c:

BRB assumed not resist gravity loads
=> required axial strength from lateral loads only

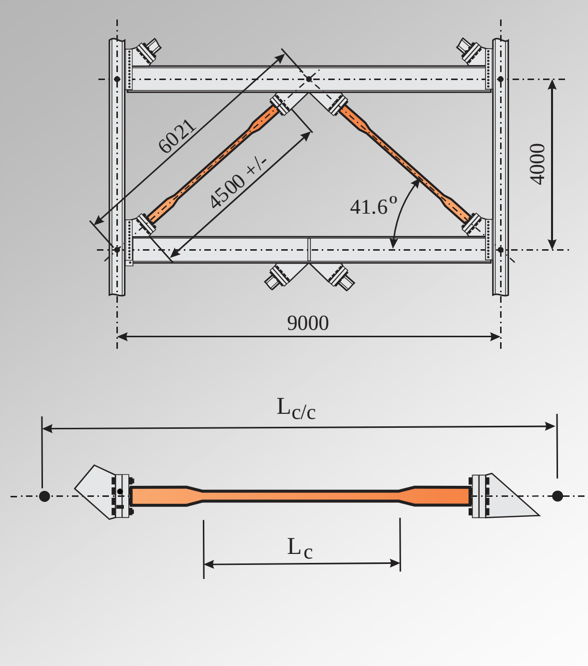
$P_n = A_c F_{yc}$ (compression & tension) with $\phi = 0.9$

Notes: use lower bound for F_{yc}
round-off A_c

Verify availability of BRB and test data

Determine stiffness factor KF for analysis

$= K_{Brace} / (EA_c/L_{c/c})$



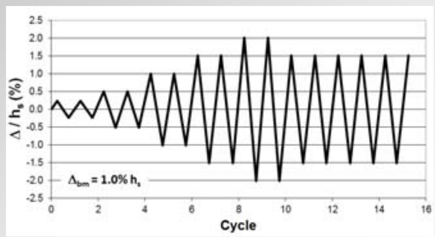
Qualification tests of BRBs:

Two cyclic tests: individual BRB + sub-assemblage

Specimen P_{ysc} within 50-120% of prototype

Specimen design, fabrication and quality control as for prototype

Test displacements based on design storey drift Δ_{bm}



Tests used to demonstrate performance of the member and connections and to provide design data

Design – BRB adjusted axial strength

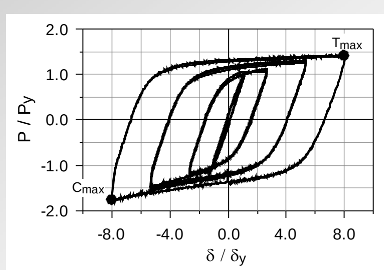
Adjusted (max) brace strengths from tests:

$T_{adj} = \omega A_c F_{yc}$

$C_{adj} = \omega \beta A_c F_{yc}$

with ω and β from C and T at the maximum test deformations

Note: use upper bound for F_{yc}



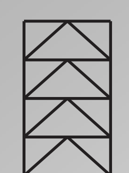
Design – Columns and Beams

Must resist gravity loads plus forces induced when the braces reach their adjusted strengths

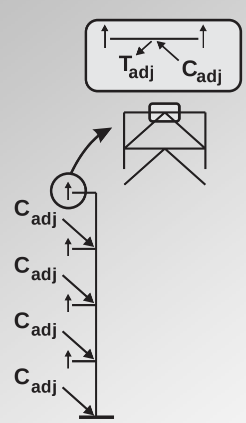
Section must meet λ_{hd} (λ_{md} for flanges if $e \leq 1.6\ M_p/V_p$)

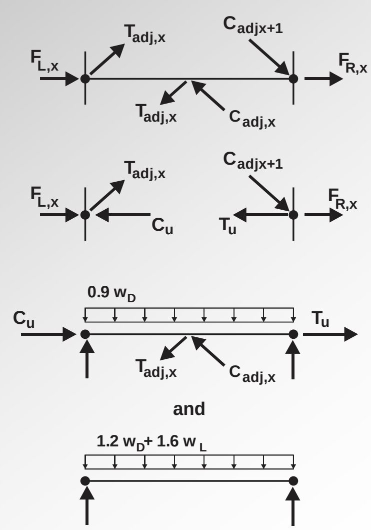
Beam Design

Column Design



Chevron BRB



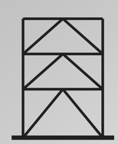


Communication with BRB suppliers

- At design stage, verify:
  - Range of F_{yc}
  - Strength factors ω & β
  - Stiffness factors KF
  - P_y range for which test data is available
- On drawings, specify:
  - Minimum P_y (or A_c & F_{yc}), with tolerances
  - Factors ω, β and KF
  - Req'd test brace axial deformations

F_{yc} = 260-290 MPa
KF = 1.5

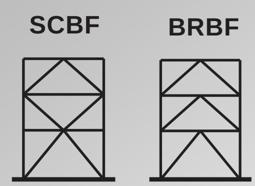
R = 6.0 (as EBF)
T = 0.99 s
Q_0 = 871 kN / Frame



ϕ_c = 0.90
F_y = 0.260 kN/mm²
R_yF_y = 0.290 kN/mm²

ω = 1.40
β = 1.10

Braces									Adjusted Strengths	
KL (mm)	θ (deg)	P_D (kN)	P_L (kN)	P_E (kN)	Level ( )	P_u (kN)	$A_{c,req}$ (mm^2)	$A_{c,sup}$ (mm^2)	T_{adj} (kN)	C_{adj} (kN)
6021	41.63	0	0	245	9	343	1465	1600	650	715
6021	41.63	0	0	411	2	575	2458	2600	1056	1161
7106	50.71	0	0	688	1	963	4117	4200	1705	1876

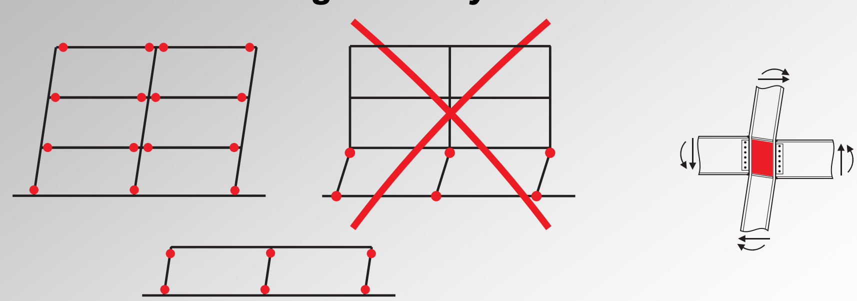


Brace KL	SCBF c/c	SCBF L_H	BRBF
R =	5.5	5.5	6.0
T (s) =	0.55	0.65	0.99
Q_0 (kN) =	1974	1562	871
Steel (t) =	16.3	13.7	4.1[1]
Q_{exp} (kN) =	7028	5090	2268

[1] Braces not included

Moment Resisting Frames

Energy dissipated by plastic hinging in beams and limited shear yielding in column panel zones. Plastic hinging in columns permitted at the base and in single-storey structures.



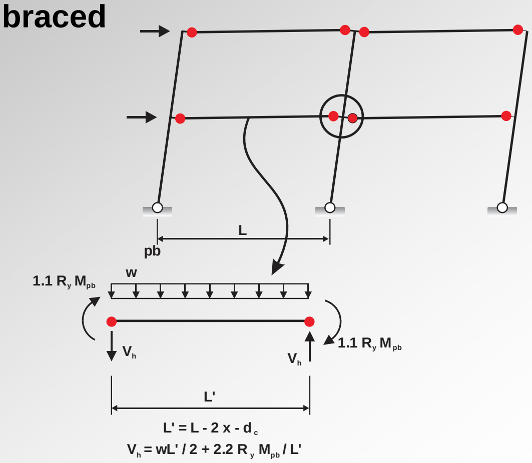
Connections and other members expected to remain essentially elastic

Design – Beams

Section must meet λ_{hd}

Must resist expected shear demand upon hinging

Must be laterally braced



Design – Columns

Section must meet λ_{hd}

Must satisfy “weak beam-strong column” criteria except for:

- Columns with $P_{uc} < 0.3\ A_cF_y$ in single-storey buildings or at the top storey of multi-storey buildings;
- Columns with $P_{uc} < 0.3\ A_cF_y$ when their total shear contribution < 20% of total storey shear resistance and 33% of storey shear resistance along their MF line; or
- Columns that have shear capacity to demand ratio 50% gretaer than in the storey above.

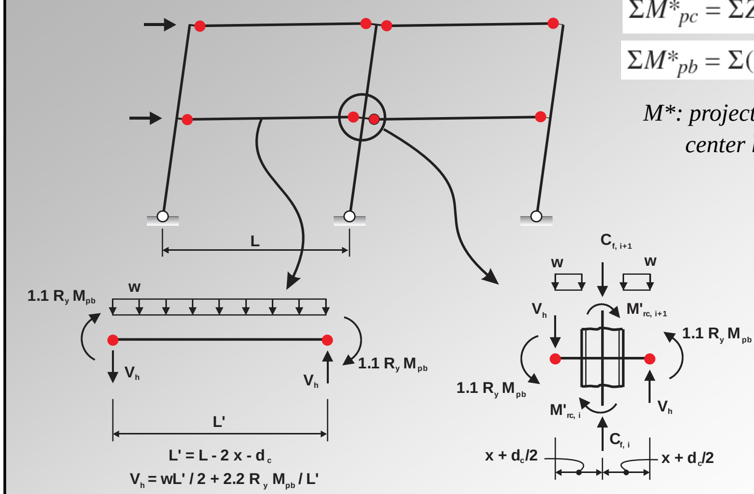
"Weak beam-strong column" criteria:

$$\frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0$$

$$\Sigma M^*_{pc} = \Sigma Z_c (F_{yc} - P_{uc}/A_g)$$

$$\Sigma M^*_{pb} = \Sigma (1.1 R_y F_{yb} Z_b + M_{uv})$$

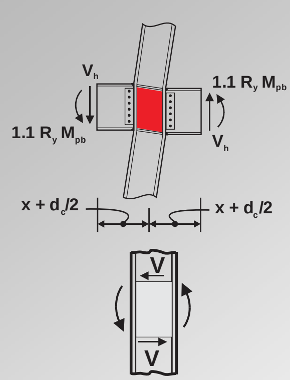
M: projected at member center lines*



Design – Column panel zone

Must meet: $t > (d_z + w_z)/90$

Shear strength, R_n:



The *nominal strength*, R_n, shall be determined as follows:

(a) When the effect of panel-zone deformation on frame *stability* is not considered in the analysis:

(i) For $P_r \le 0.4P_c$

$$R_n = 0.60 F_y d_c t_w \quad \text{(J10-9)}$$

(ii) For $P_r > 0.4P_c$

$$R_n = 0.60 F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \quad \text{(J10-10)}$$

(b) When frame stability, including plastic panel-zone deformation, is considered in the analysis:

(i) For $P_r \le 0.75P_c$

$$R_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right) \quad \text{(J10-11)}$$

(ii) For $P_r > 0.75P_c$

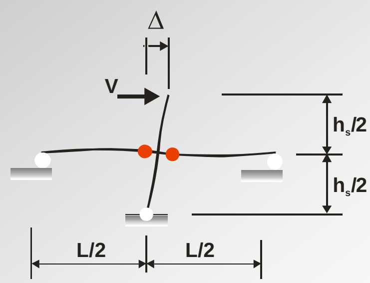
$$R_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2 P_r}{P_c}\right) \quad \text{(J10-12)}$$

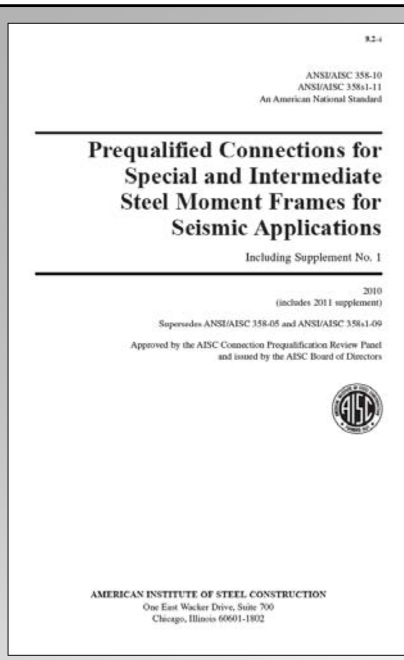
Design – Beam-to-column connections

Must accommodate 4% storey drift angle

Measured flexural resistance at column face at 4% storey drift angle $\geq$ 80% M_{pb}

Performance considered as demonstrated if pre-qualified connections are used; otherwise must be demonstrated through physical cyclic testing





ANSI/AISC 358-10
ANSI/AISC 358s1-11
An American National Standard

Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

Including Supplement No. 1

2010
(includes 2011 supplement)

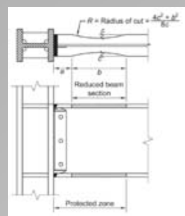
Supersedes ANSI/AISC 358-05 and ANSI/AISC 358s1-09

Approved by the AISC Connection Prequalification Review Panel and issued by the AISC Board of Directors

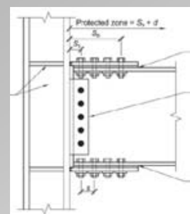
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

http://www.aisc.org

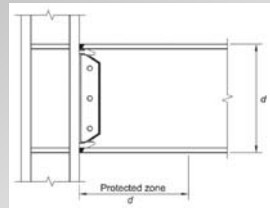
- ***Design requirements***
- ***Welding requirements***
- ***Bolting requirements***
- ***Requirements for 6 pre-qualified connections***



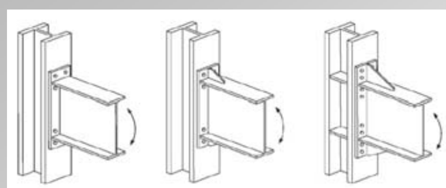
Reduced Beam Section



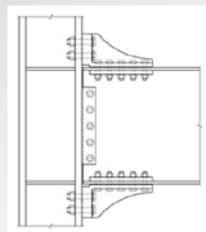
Bolted Flange Plate



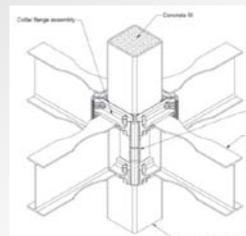
Welded Unreinforced Flange Welded Web



Bolted End Plate



Kaiser Bolted Bracket



Conxtech Conxl

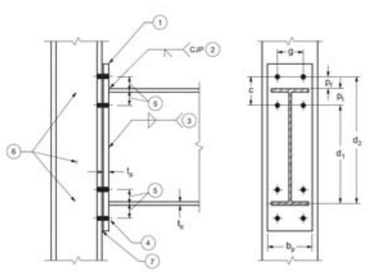


Figure 4
Bolted Unstiffened End Plate (BUEP) Connection

3. Probable Maximum Moment at Plastic Hinge

The *probable maximum moment at the plastic hinge* shall be:

$$M_{pr} = C_{pr} R_y F_y Z_e \qquad (2.4.3\text{-}1)$$

where

M_{pr} = probable maximum moment at plastic hinge, kip-in. (N-mm)

R_y = ratio of the expected yield stress to the specified minimum yield stress F_y as specified in the AISC *Seismic Provisions*

Z_e = effective plastic section modulus of the section (or connection) at the location of the plastic hinge, in.^{3} (mm^3)

C_{pr} = factor to account for the peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. Unless otherwise specifically indicated in this Standard, the value of C_{pr} shall be:

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \qquad (2.4.3\text{-}2)$$

where

F_y = specified minimum yield stress of the yielding element, ksi (MPa)

F_u = specified minimum tensile strength of the yielding element, ksi (MPa)

6.10. DESIGN PROCEDURE

Connection geometry is shown in Figures 6.2, 6.3 and 6.4 for the 4E, 4ES and 8ES connections, respectively.

1. End-Plate and Bolt Design

Step 1. Determine the sizes of the connected members (beams and column) and compute the moment at the face of the column, M_f.

$$M_f = M_{pr} + V_u S_h \qquad (6.10\text{-}1)$$

where

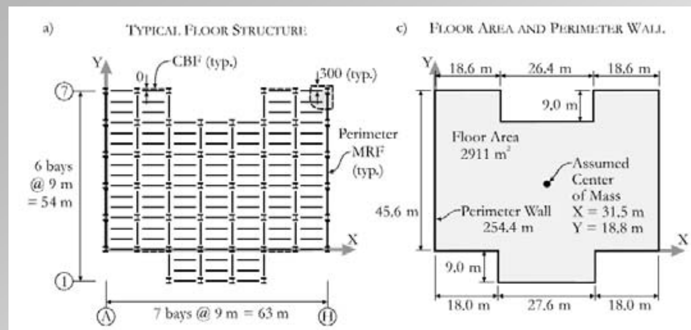
M_{pr} = probable maximum moment at plastic hinge, kip-in. (N-mm), given by Equation 2.4.3-1

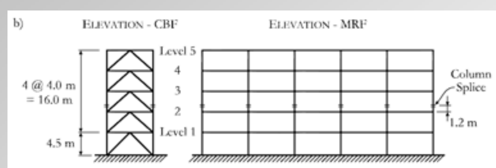
S_h = distance from face of column to plastic hinge, in. (mm)
= the lesser of $d/2$ or $3b_{bf}$ for an unstiffened connection (4E)
= $L_{st} + t_p$ for a stiffened connection (4ES, 8ES)

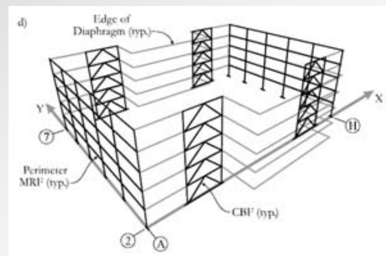
V_u = shear force at end of beam, kips (N)

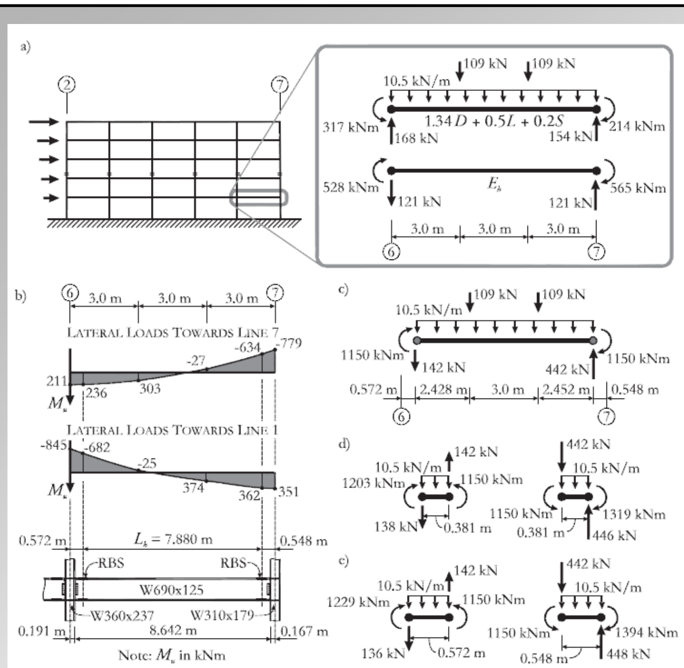
$$= \frac{2M_{pr}}{L_h} + V_{gravity} \qquad (6.10\text{-}2)$$

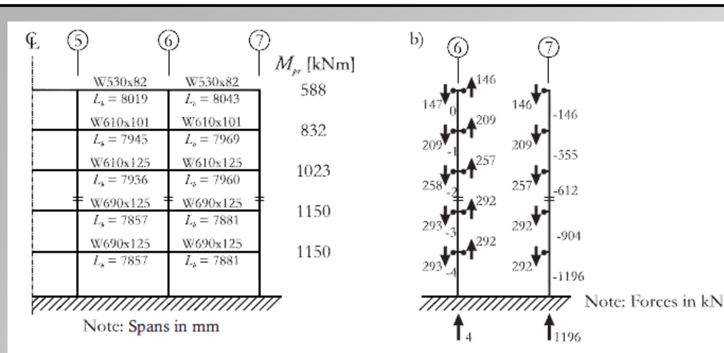
MRF Example

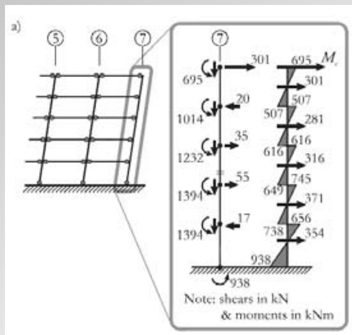

a)
TYPICAL FLOOR STRUCTURE
CBF (typ.)
300 (typ.)
Perimeter MRF (typ.)
6 bays @ 9 m = 54 m
7 bays @ 9 m = 63 m
c)
FLOOR AREA AND PERIMETER WALL
18.6 m
26.4 m
18.6 m
9.0 m
Floor Area 2911 m²
Assumed Center of Mass X = 31.5 m Y = 18.8 m
45.6 m
Perimeter Wall 254.4 m
9.0 m
18.0 m
27.6 m
18.0 m


b)
ELEVATION - CBF
ELEVATION - MRF
Level 5
4
3
2
Level 1
4 @ 4.0 m = 16.0 m
4.5 m
Column Splice
1.2 m


d)
Edge of Diaphragm (typ.)
Perimeter MRF (typ.)
CBF (typ.)


a)
10.5 kN/m
109 kN
109 kN
317 kNm
168 kN
1.34D + 0.5L + 0.2S
154 kN
214 kNm
528 kNm
121 kN
E_h
121 kN
565 kNm
3.0 m
3.0 m
3.0 m
b)
LATERAL LOADS TOWARDS LINE 7
LATERAL LOADS TOWARDS LINE 1
L_h = 7.880 m
RBS
W690x125
W360x237
W310x179
8.642 m
Note: M_u in kNm
c)
1150 kNm
142 kN
442 kN
1150 kNm
0.572 m
2.428 m
3.0 m
2.452 m
0.548 m
d)
1203 kNm
1319 kNm
138 kN
446 kN
0.381 m
e)
1229 kNm
1394 kNm
136 kN
448 kN



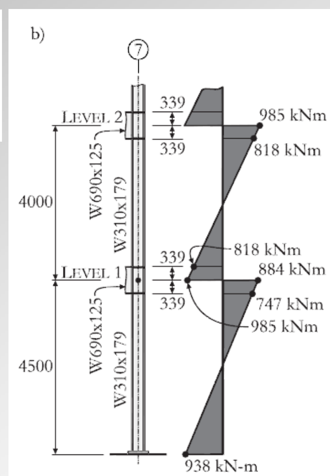


At Level 1:

ΣM^*_{pb} = 656 + 738 = 1394 kN-m

$$\text{Level 1: } M_{pc,1} = 3050 \times 10^3\left(0.345 - \frac{2270}{22\,700}\right)\frac{1}{10^3} = 747 \text{ kN-m}$$

$$\text{Level 2: } M_{pc,2} = 3050 \times 10^3\left(0.345 - \frac{1742}{22\,700}\right)\frac{1}{10^3} = 818 \text{ kN}$$

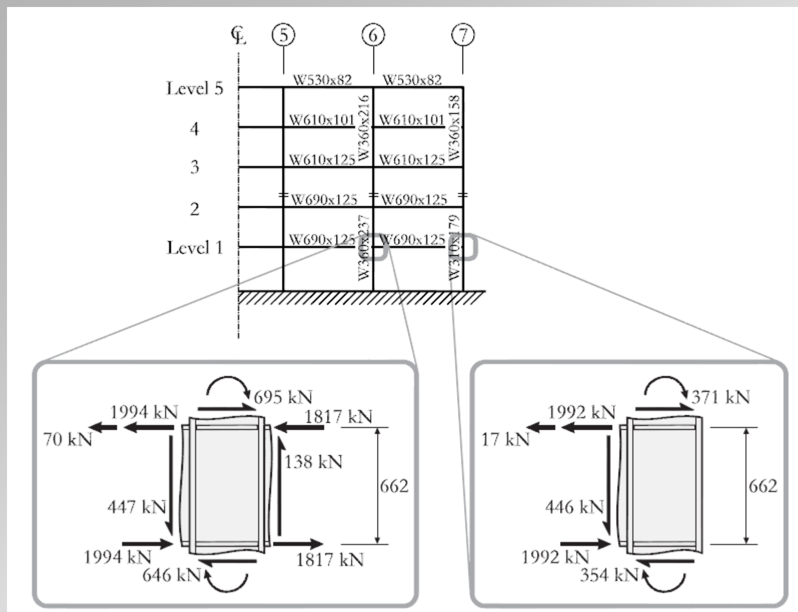


$$M^*_{pc,1-1} = 747 + 339\frac{(747 + 938)}{4500 - 339} = 884 \text{ kN-m}$$

$$M^*_{pc,1-2} = 818 + 339\frac{(818 + 818)}{4000 - 339 - 339} = 985 \text{ kN-m}$$

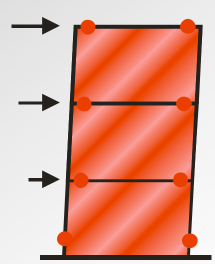
$$\Sigma M^*_{pc} = 884 + 985 = 1869 \text{ kN-m}$$

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} = \frac{1869}{1394} = 1.34 > 1.0$$



Steel Plate Shear Walls

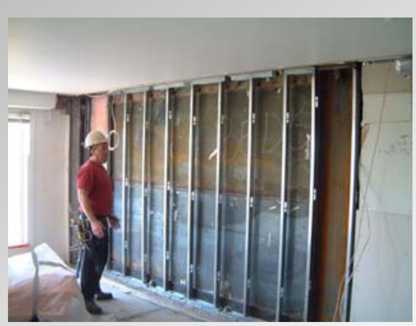
Energy dissipated through web plate (infill panel) yielding and plastic hinging in beams and at the base of columns

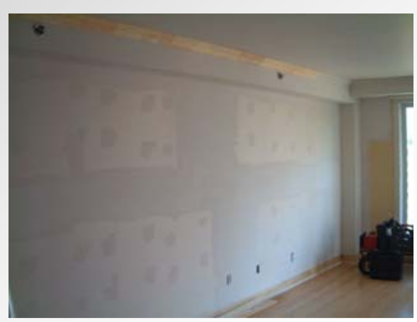


Connections and other members expected to remain essentially elastic

ING St-Hyacinthe
Quirion Metal

R. Tremblay, Polytechnique Montreal, Canada 147

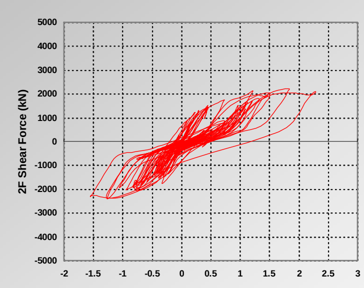


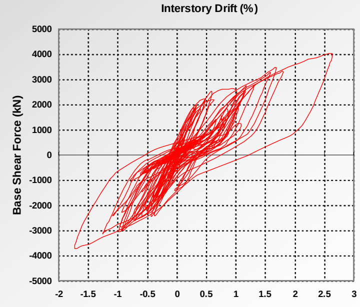


Louis Crépeault
Groupe Technika

R. Tremblay, Polytechnique Montreal, Canada 148

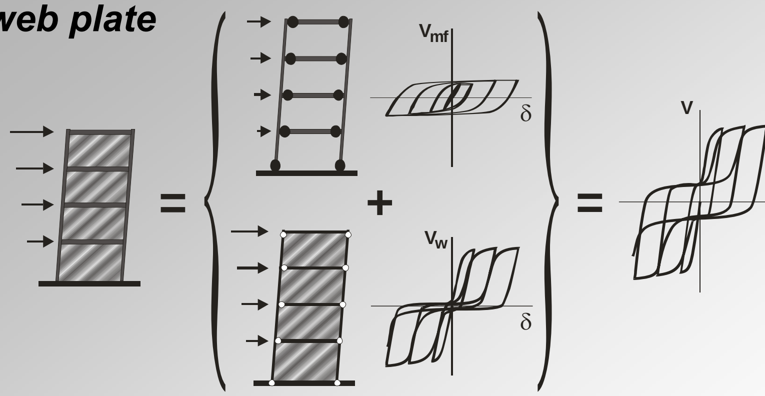
University of Alberta (1997)





Work by Bruneau, Tsai et al.

MF + web plate

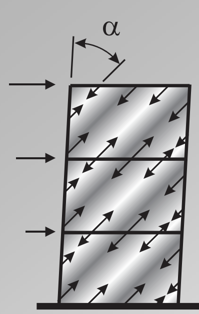


Distribution of V_x from analysis

Infill panel designed for 100% V_x

Beams M_{pb} such that $V_{MF} = 2\,\phi\, M_{pb}/h_s \geq 0.25\, V_x$

Design of web plate

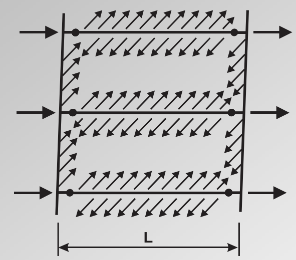


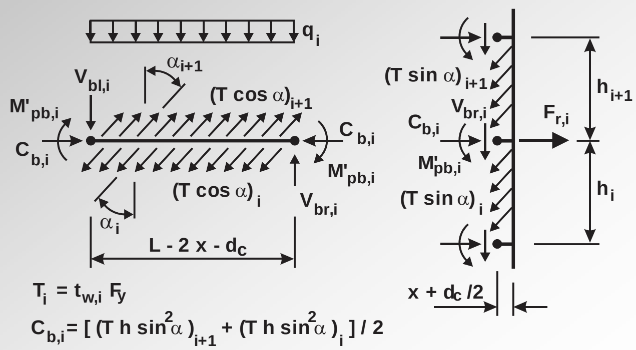
$$V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha$$

α = angle of web yielding in degrees, as measured relative to the vertical. The angle of inclination, α, is permitted to be taken as 40°, or is permitted to be calculated as follows:

$$\tan^4 \alpha = \frac{1 + \frac{t_w L}{2 A_c}}{1 + t_w h \left(\frac{1}{A_b} + \frac{h^3}{360 I_c L} \right)}$$

Design of beams (HBE) and columns (VBE)





Beam-to-Column Connections

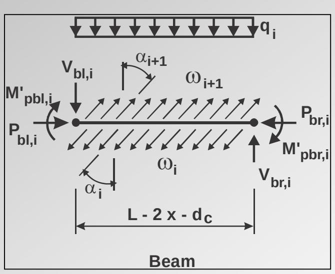
27.9.3.4

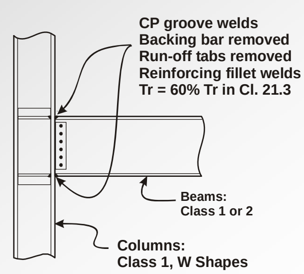
The factored resistance of the beam web-to-column connection shall equal or exceed the effects of gravity loads and tension forces in the infill plates, as determined in accordance with Clause 27.9.2.2, acting above and below the beams, combined with shears induced by moments of $1.1R_yM_{pb}$ acting at plastic hinge locations. The moments acting in the beam plastic hinges may be taken as $1.18(1.1R_yM_{pb})(1-C_f/\phi C_y)$, where C_f is the beam axial load due to the tension forces in the infill plates and C_y is the axial yield resistance of the beam.

27.9.6 Beam-to-column joints and connections

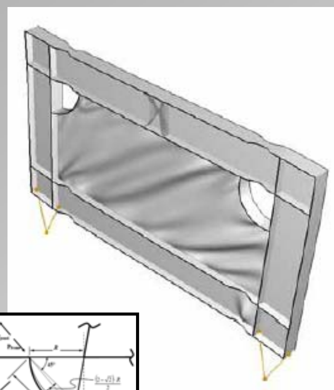
Beam-to-column joints and connections shall meet the requirements of Clause 27.4.4, except that Clause 27.4.4.2(b) shall not apply.

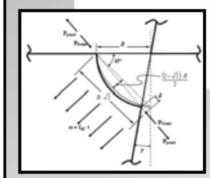
Type LD MRF





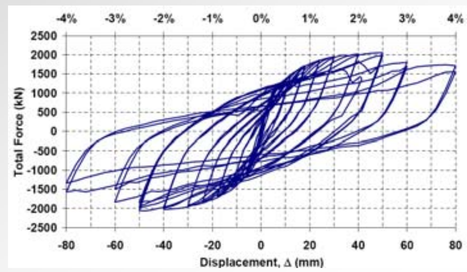
Corner cut-outs





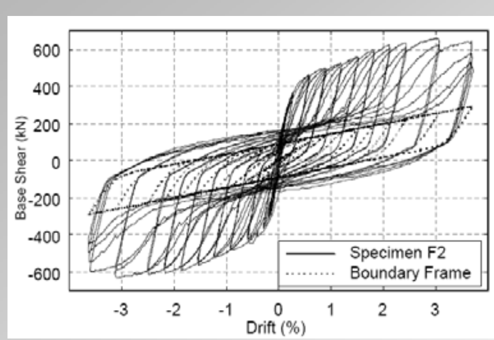
Purba, R. and Bruneau, M. 2007. Design Recommendations for Perforated Steel Plate Shear Walls. Report MCEER-07-0011, SUNY Buffalo, NY.

Vian, D. and Bruneau, M. 2004. Testing of Special LYS Steel Plate Shear walls. Proc. 13th WCEE, Vancouver, BC. Paper No. 978.



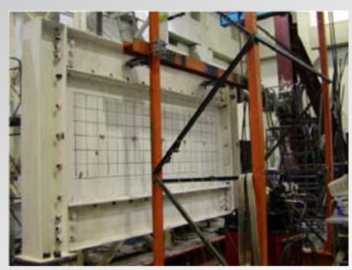
Optimization of infill plates

Use of thin infill plates

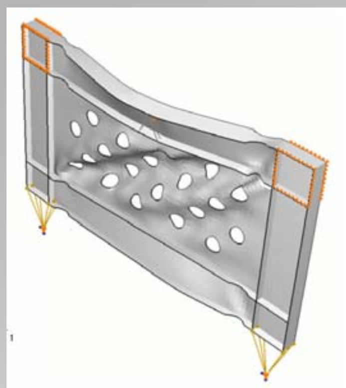


0.9 mm thick ASTM A1008 (cold-rolled, carbon, commercial steel sheet)

Berman, J.W. and Bruneau, M. 2005. Experimental Investigation of Light-Gauge Steel Plate Shear Walls. ASCE J. of Struct. Eng., 131, 2, 259-267.



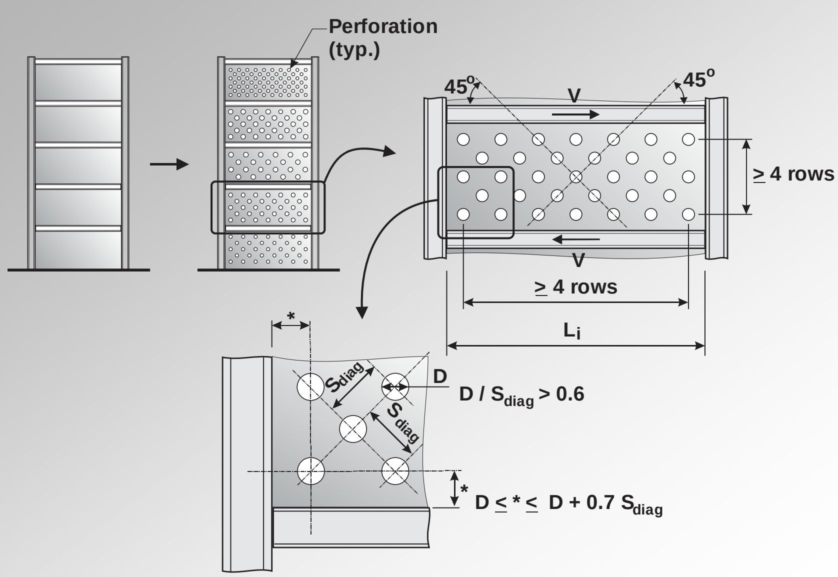
Perforated infill plates



Purba, R. and Bruneau, M. 2007. Design Recommendations for Perforated Steel Plate Shear Walls. Report MCEER-07-0011, SUNY Buffalo, NY.

Vian, D. and Bruneau, M. 2004. Testing of Special LYS Steel Plate Shear walls. Proc. 13th WCEE, Vancouver, BC. Paper No. 978.

$$V_n = 0.42 F_y t_w L_{cf}\left(1 - \frac{0.7D}{S_{diag}}\right)$$



R. Tremblay, Polytechnique Montreal, Canada 159

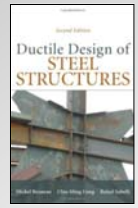
R. Tremblay, Polytechnique Montreal, Canada 160

Warning!

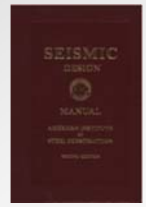
Only basic design requirements have been discussed; several other requirements must be applied including those related to loads and load combinations, demand critical welds, protected zones, bracing, quality control, etc.

Only the systems designed and detailed for high ductility have been introduced; provisions also exist for other systems exhibiting moderate and limited ductility that may be more appropriate for some applications.

Bruneau, M., Sabelli, R., and Uang C.-M. (2003) Ductile Design of Steel Structures, 2nd ed., Wiley



AISC. (2013) Seismic Design Manual, 2nd ed., AISC



Filiatrault, A., Tremblay, R., Christopoulos, C., Foltz, B., and Pettinga, D. (2013) Elements of Earthquake Engineering and Structural Dynamics, 3rd ed., Presses Internationales Polytechnique (PIP)

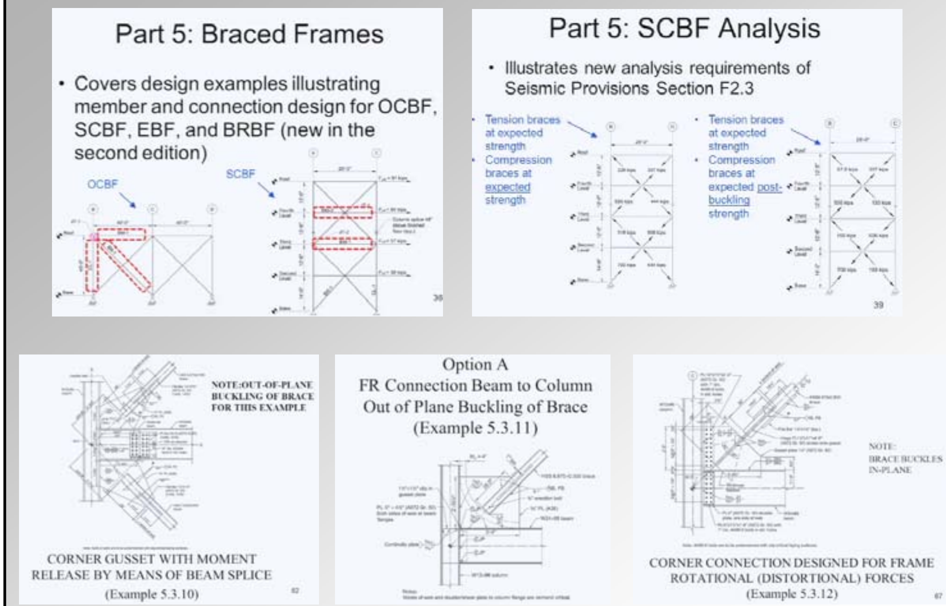
2nd Ed. Seismic Manual Table of Contents

- Part 1: General Design Considerations
- Part 2: Analysis
- Part 3: Systems Not Specifically Detailed for Seismic Resistance
- Part 4: Moment Frames
- Part 5: Braced Frames

2nd Ed. Seismic Manual Table of Contents

- Part 6: Composite Moment Frames
- Part 7: Composite Braced Frames and Shear Walls
- Part 8: Diaphragms, Collectors, and Chords
- Part 9: Provisions and Standards (Seismic Provisions and Prequalified Connections for Moment Frames)
- Part 10: Engineered Damping Systems

7



Seismic Design of Heavy Industrial Buildings : Challenges

Observations / Issues:

- **Structures are not buildings:**
  - Irregular structures with heavy point masses and loads and low damping
  - Design is process driven and structure will likely be modified to accommodate changes to the process
  - Equipment may interact with the structure
  - …
- **Structures may have limited redundancy**
- **Damage under severe earthquakes must be limited:**
  - Structures may contain hazardous material
  - No or short downtimes
- **Application of ductile seismic systems not practical, often impossible**
- **Current building code provisions not suitable**

Possible avenues:

Ductility (or alternative similar approach) needed to accommodate uncertainty in ground motions and seismic response

1. **Simple code provisions to control inelastic demand: low R factors , use of dynamic analysis, etc.**
2. **Use of ductile anchorage systems with minimum stretch lengths in combination with shear keys**
3. **Use ductile fuses in key structural elements to control the force demand**
4. **Where applicable, use sliding or rocking systems to control input**

Above avenues are listed in order of ease of implementation implementation and flexibility for future process changes; however, the latest ones could be more effective

In Canada, industrial structures are buildings and National Building Code (NBCC) applies.

New Annex proposed for inclusion in CSA S16-14 and which would be referenced in NBCC 2015:

ANNEX M (Informative)

Seismic design of industrial steel structures

Note: *This informative (non-mandatory) Annex has been written in normative (mandatory) language to facilitate adoption where users of the Standard or regulatory authorities wish to adopt it formally as additional requirements to this Standard.*

M.1 General

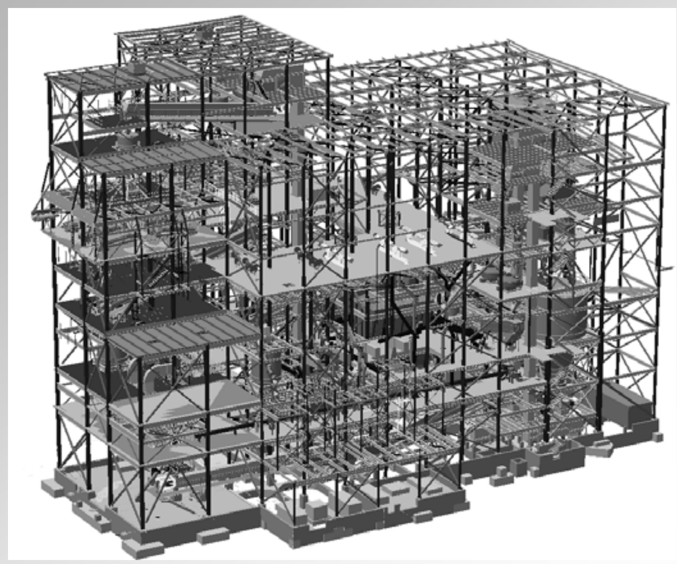
M.1.1 Scope

This Appendix applies to industrial type structures that are expected not to respond to seismic ground motions in a fashion similar to conventional buildings because of non-uniform distribution of mass, strength and stiffness in the building, absence of clearly defined floors, or reduced damping due to limited architectural components. The intended use of these structures are essentially to support equipment and material for an industrial process that may significantly affect the structure seismic response, and do not include the shelter of persons. These provisions do not apply to warehouses or to office buildings for industrial complexes. These provisions do not apply to nuclear facilities.

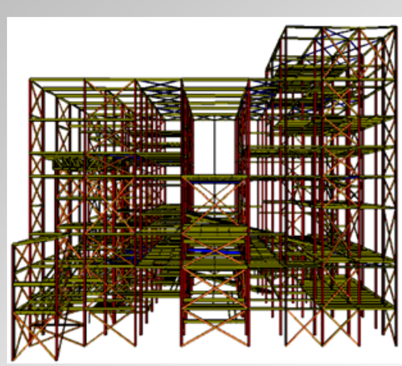
Recent related research work:

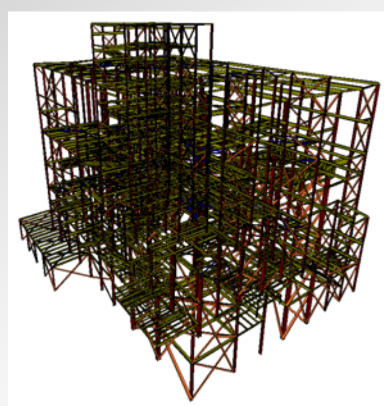
1. ***Demand prediction from RS analysis***
2. ***Use/design of ductile anchorage***
3. ***Ductile structural fuses***
4. ***Multi-tiered braced frames***

Demand Prediction from RS Analysis

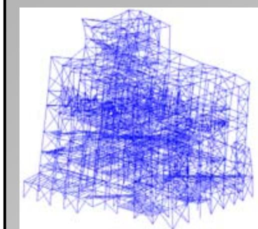


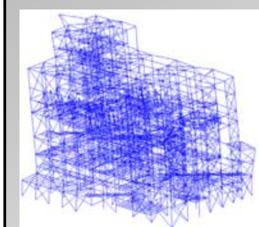
- Plan dimensions: 36 m x 60 m x 40 m
- Heavy equipment, including 1200t & 750t tanks
- Irregularities in mass and stiffness
- Montreal Site Class C
- Static, Response Spectrum & Linear response history analyses





Structure has a large number of contributing modes





		Masses modales effectives					
MODE	T (s)	X (E-O) (%)	Y (Vert.) (%)	Z (N-S) (%)	Cumul-X (%)	Cumul-Y (%)	Cumul-Z (%)
1	1.456	0.0	0.0	0.2	0.0	0.0	0.2
2	1.383	53.2	0.0	2.1	53.2	0.0	2.4
3	1.323	1.5	0.0	63.4	54.7	0.0	65.7
4	1.114	0.2	0.0	0.2	54.8	0.0	65.9
5	1.069	7.1	0.0	0.0	62.0	0.0	65.9
6	1.047	4.2	0.0	0.2	66.2	0.0	66.1
7	1.024	0.0	0.0	0.0	66.2	0.0	66.1
8	0.979	3.8	0.0	0.7	70.0	0.0	66.9
9	0.955	0.7	0.0	5.1	70.7	0.0	72.0
10	0.880	0.7	0.0	0.0	71.4	0.0	72.0
11	0.860	1.2	0.0	0.0	72.6	0.0	72.0
12	0.850	1.1	0.0	0.0	73.7	0.0	72.0
13	0.844	2.7	0.0	0.0	76.4	0.1	72.0
14	0.807	0.3	0.0	0.0	76.7	0.1	72.0
15	0.803	0.0	0.0	0.0	76.7	0.1	72.0
...	...	...	...	...	...	...	...
70	0.382	0.1	0.0	0.1	90.0	0.3	86.2
...	...	...	...	...	...	...	...
95	0.327	0.5	0.0	0.4	92.9	0.4	90.3

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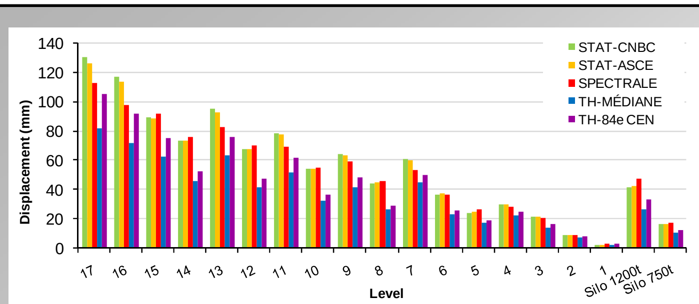

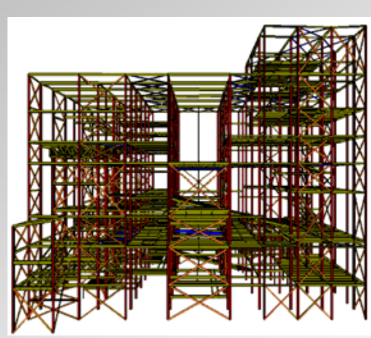

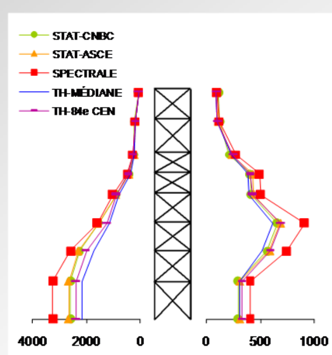

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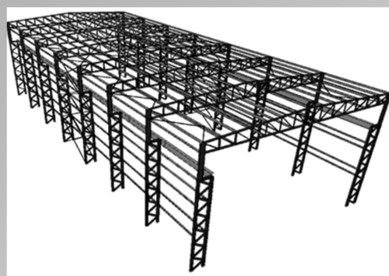
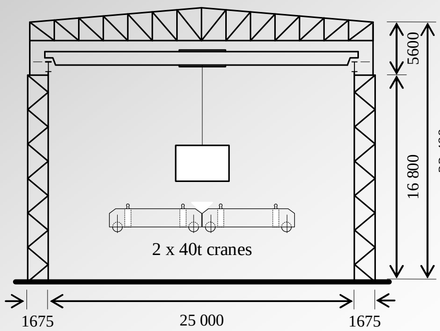
Use & Design of Ductile Anchors

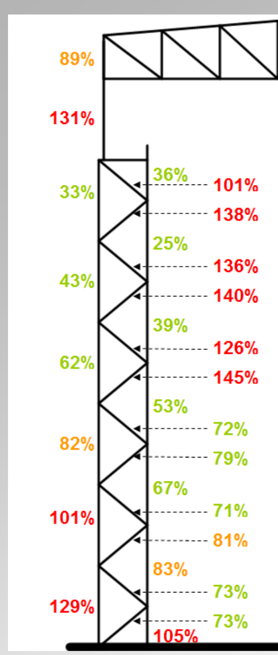


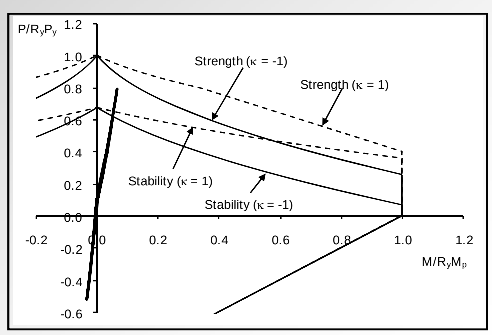


3 sites: Montreal, Vancouver & Seattle

Seismic force demand from linear response history analysis

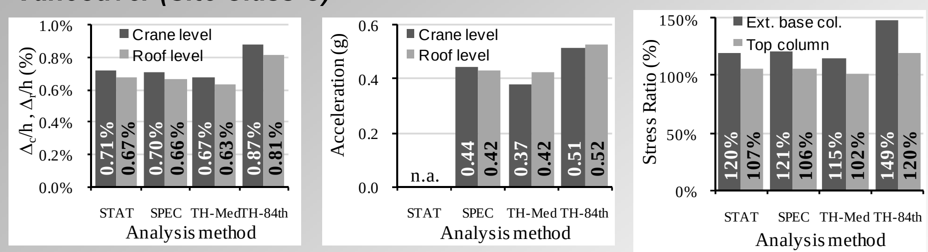
Ductile anchors at column bases



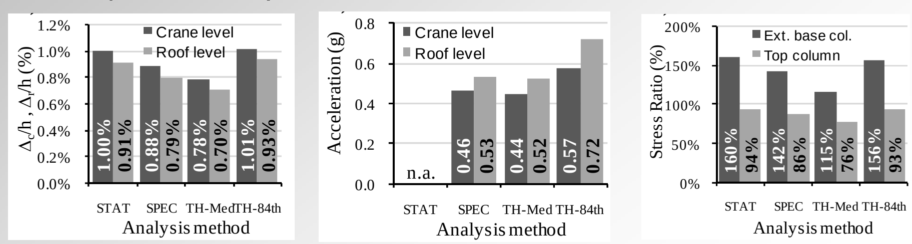


Elastic time history analysis

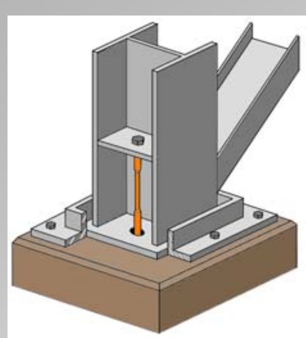
Vancouver (Site Class C)



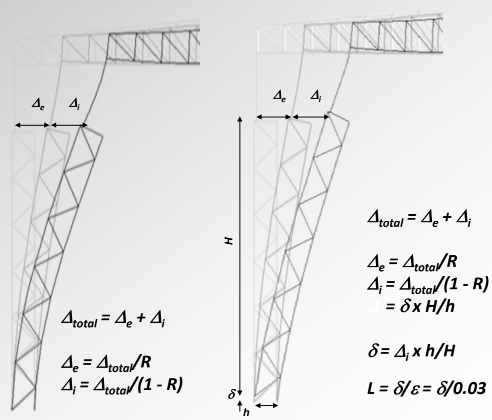
Seattle (Site Class D)



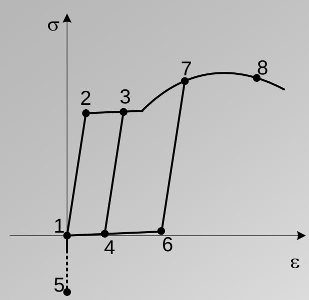
Ductile anchorage at column bases

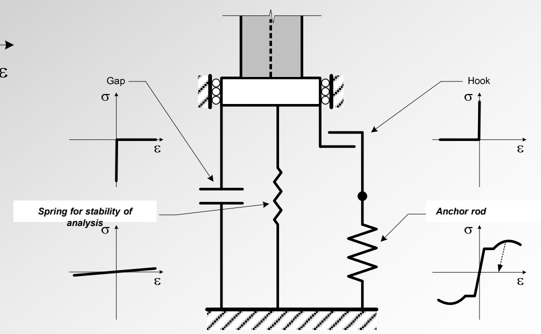


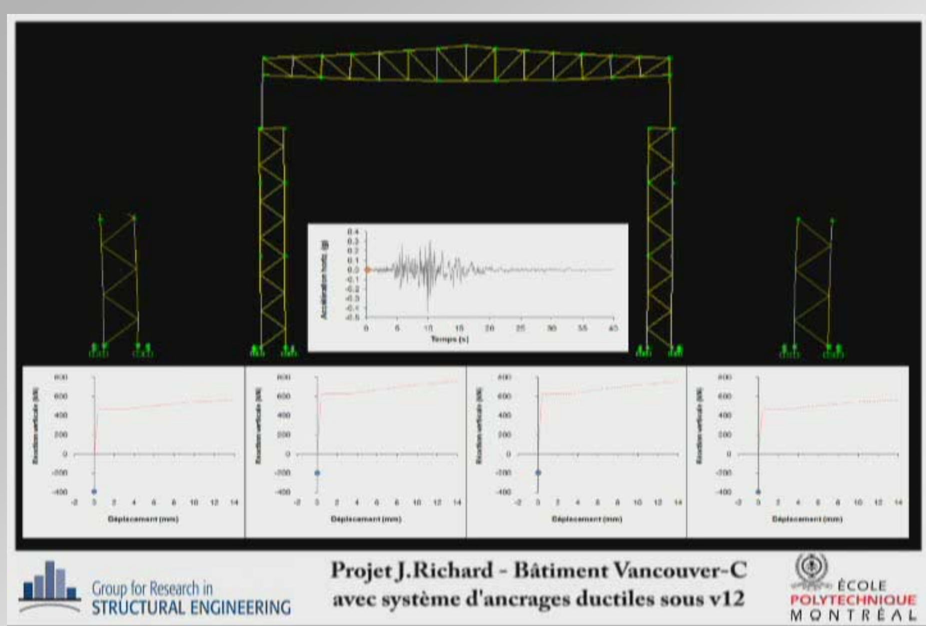
$$A_{fuse} = \frac{F^{+}_{vertical}}{R_{sh} F_{y,\,fuse}}$$

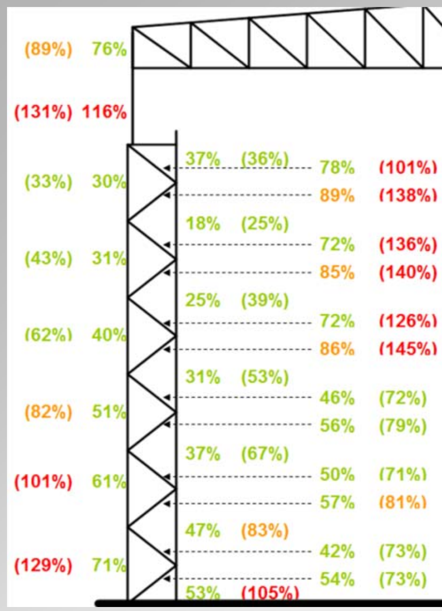


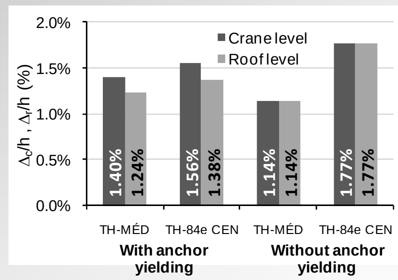
Inelastic time history analysis











NCh2369 (2003)

Is $8d_b$ or 250 mm suitable for all applications?

Detalles típicos

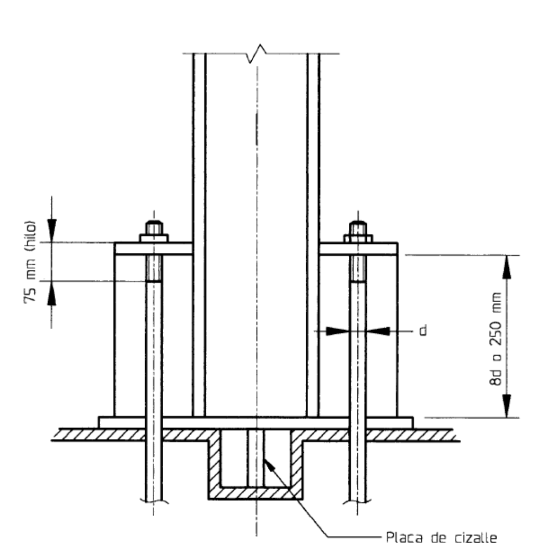
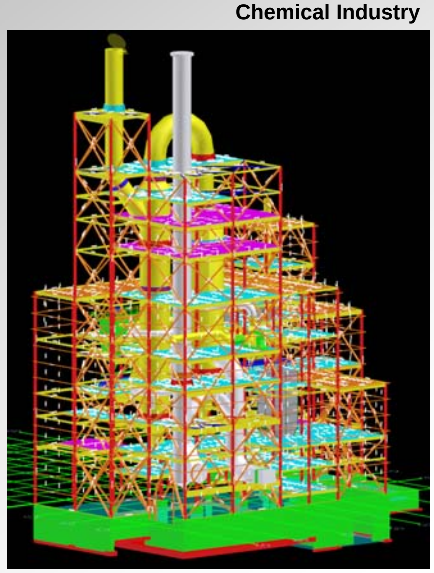
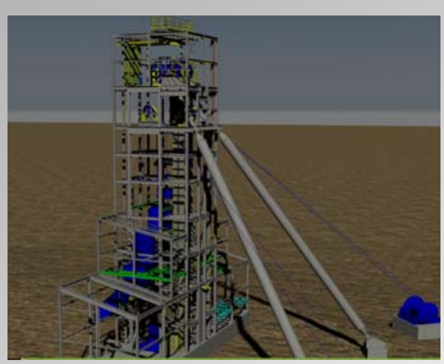


Figura A.1 - Base de columnas

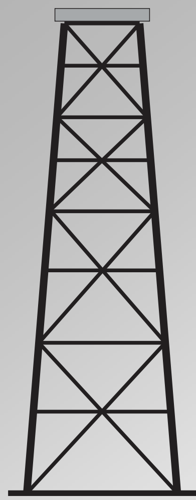
Other buildings to be examined

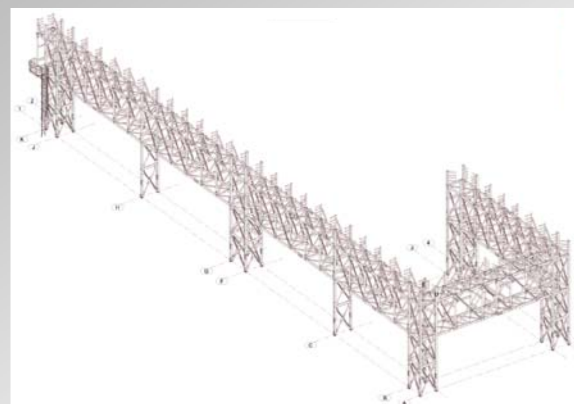
Chemical Industry





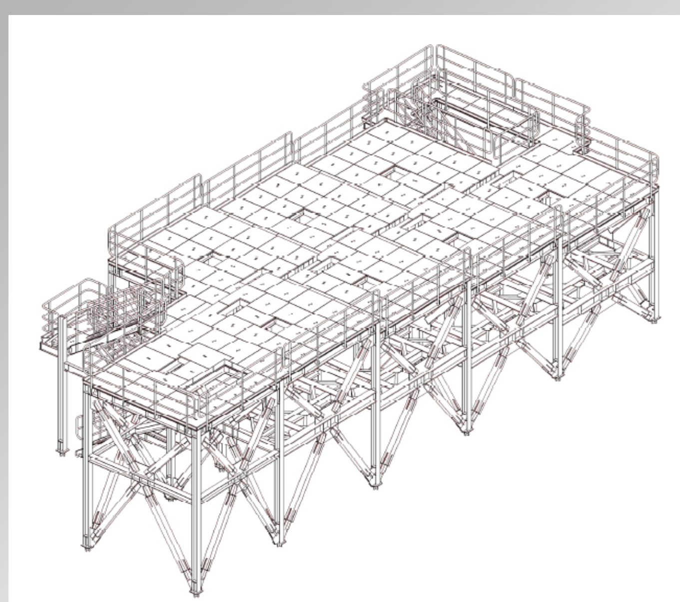
HeadFrame (Mining)





Pipe Racks

Conveyor Towers



Tank Supporting Structures

Ductile Structural Fuses

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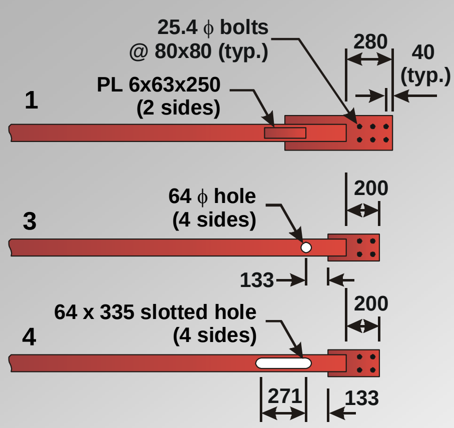
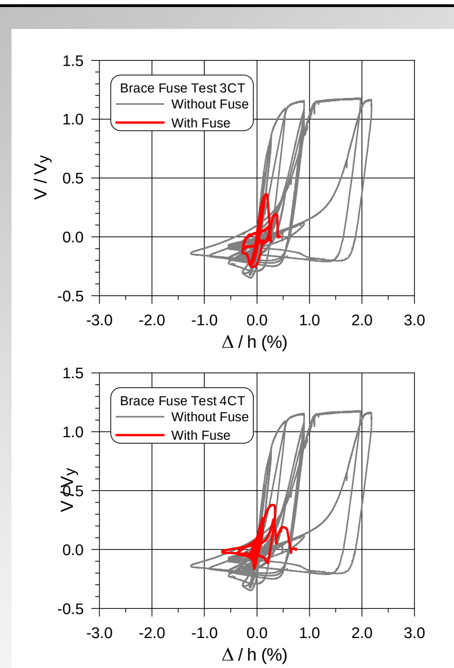


HS 102x102x4.8



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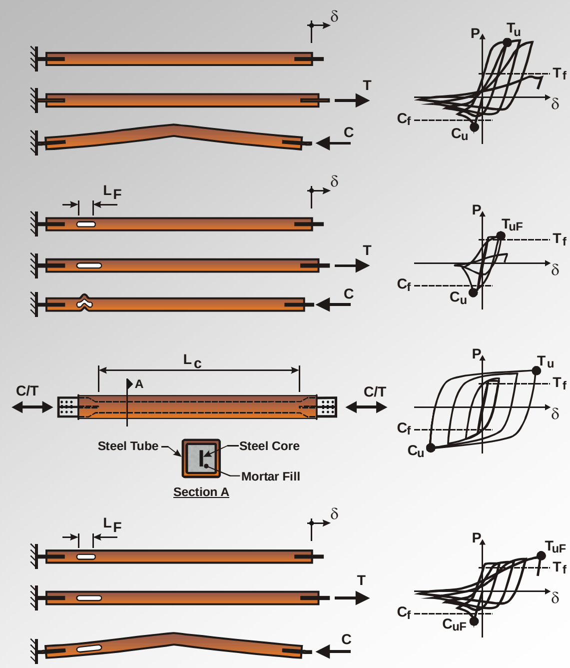
Fuses for HSS braces



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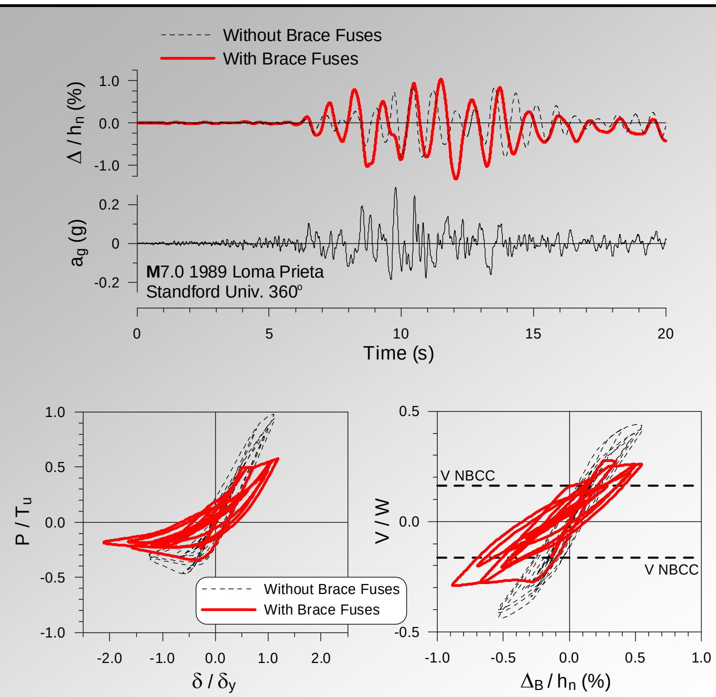

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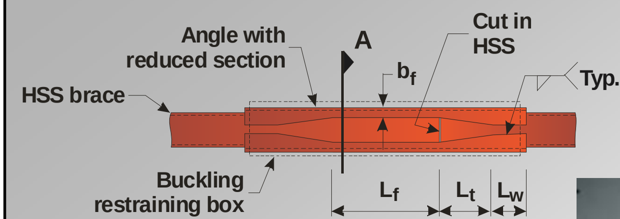

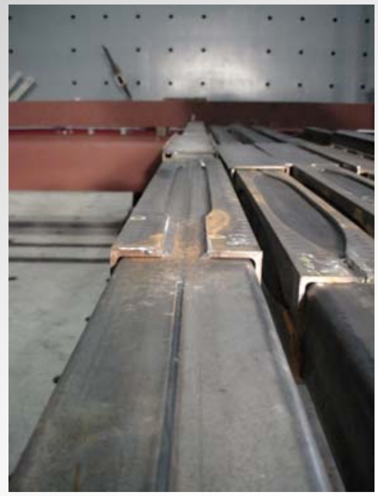

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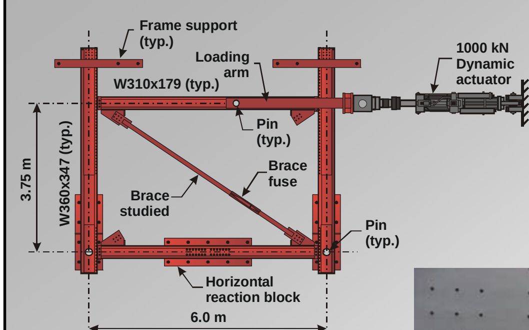

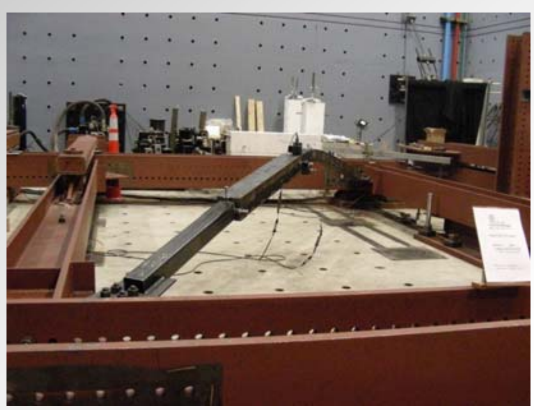

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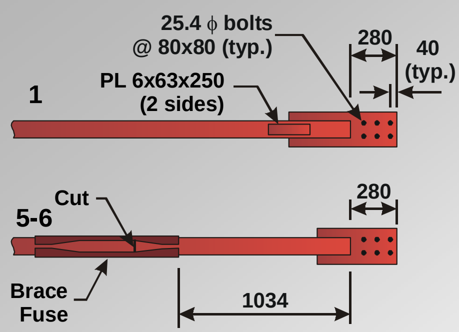
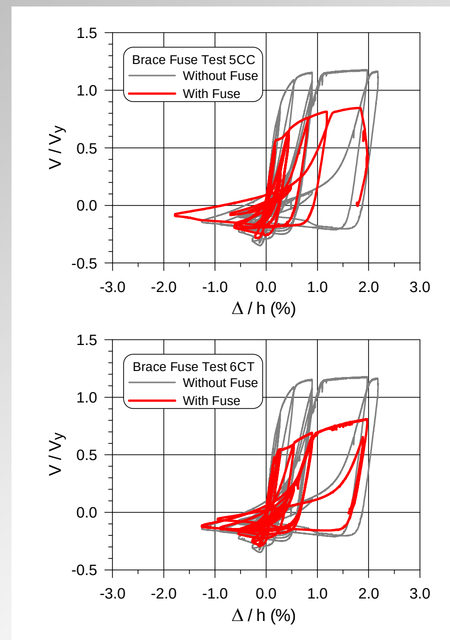
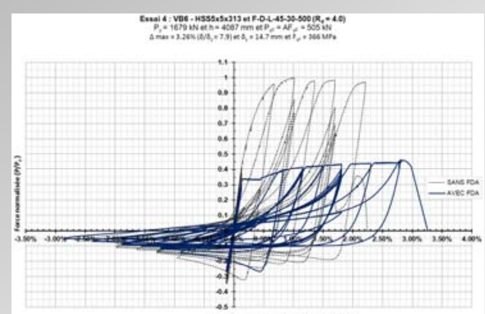
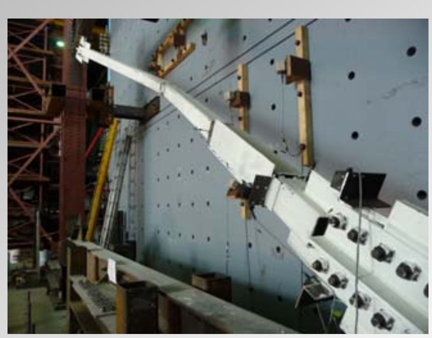
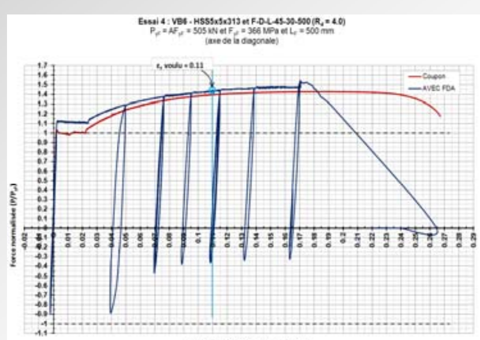

25.4 ϕ bolts
@ 80x80 (typ.)
280
40
(typ.)
PL 6x63x250
(2 sides)
1
Cut
5-6
280
Brace
Fuse
1034

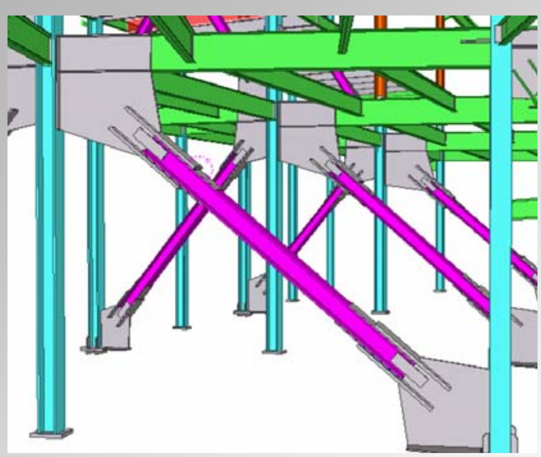

Brace Fuse Test 5CC
Without Fuse
With Fuse
V / Vy
Δ / h (%)
Brace Fuse Test 6CT
Without Fuse
With Fuse
V / Vy
Δ / h (%)

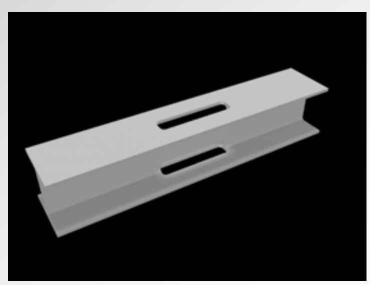


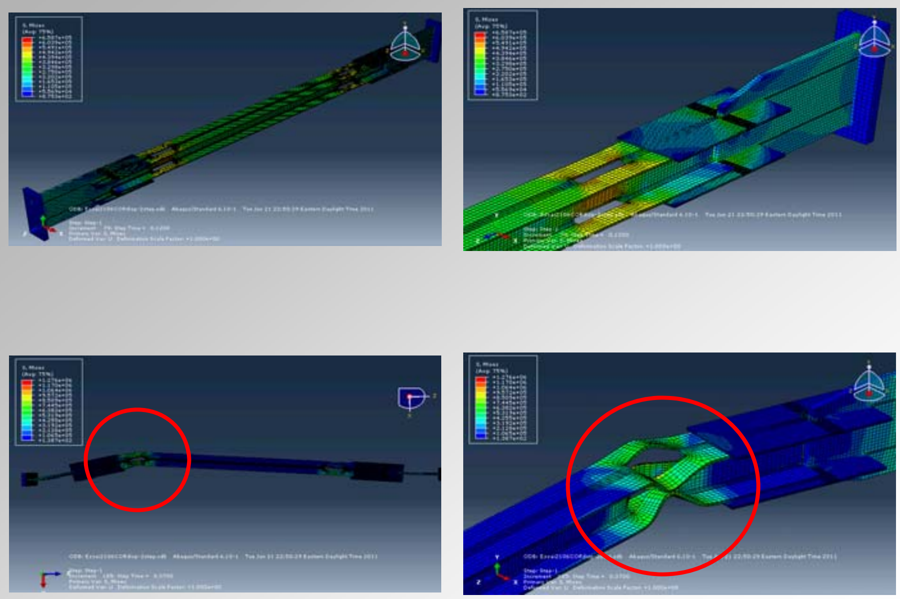


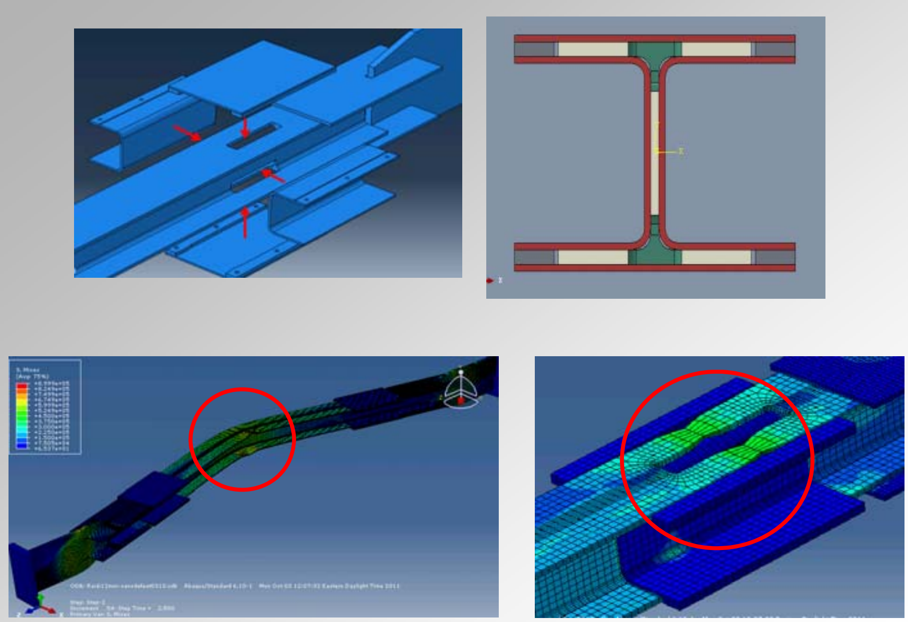


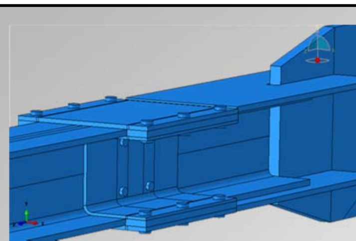
Fuses in W-Shapes (Canam Group, Montreal)











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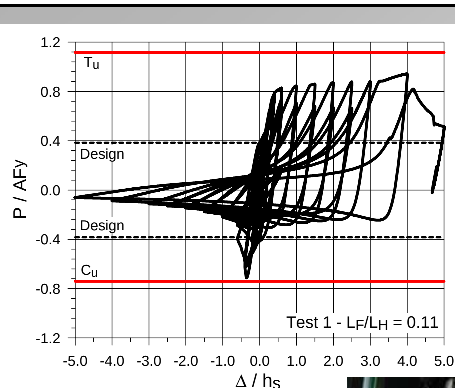

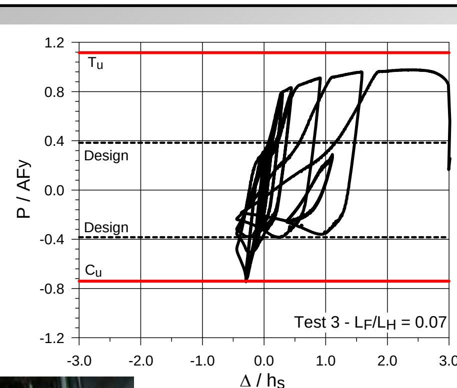

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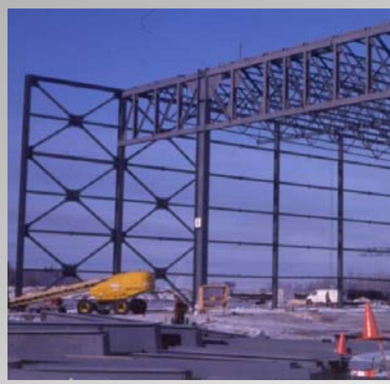
Multi-Tiered Braced Frames



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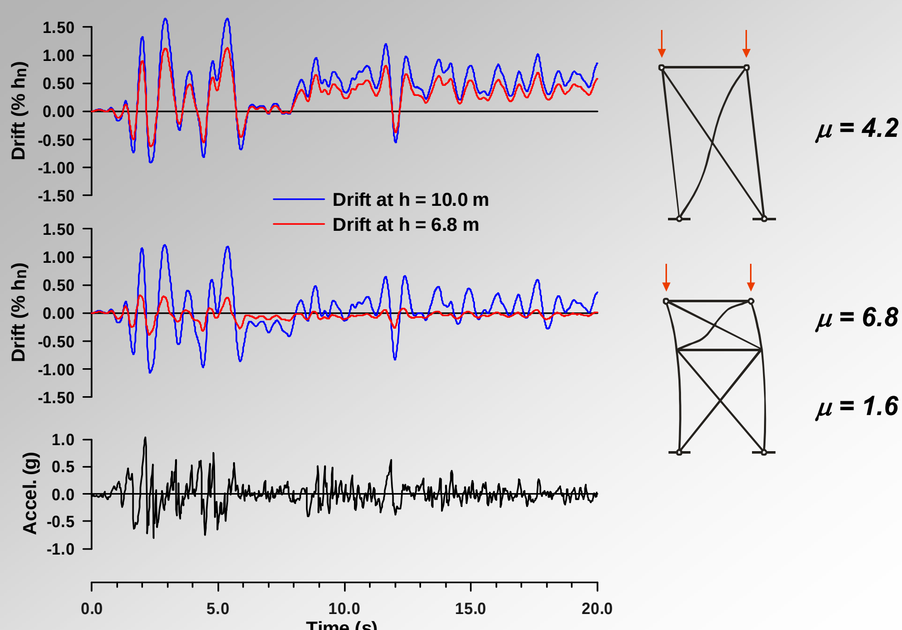
X-braced CBF vs Multi-tiered CBFs



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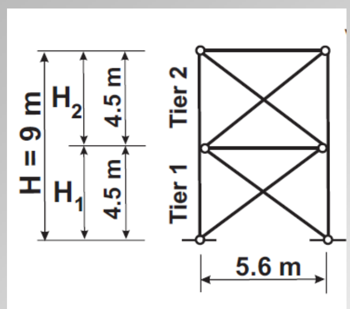
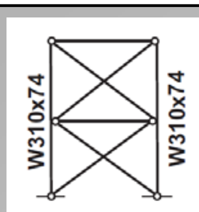
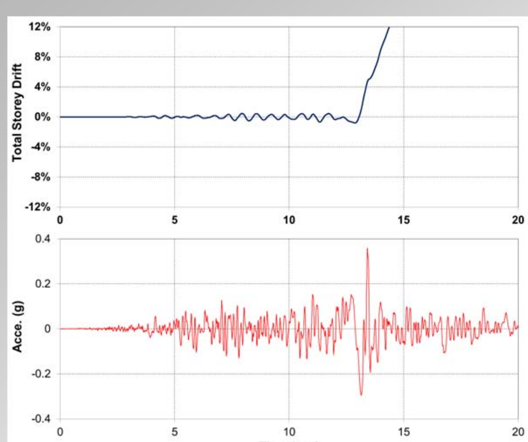
2-tiered CBFs
Designed in accordance with AISC 341-10
SCBF – R = 6.0
Los Angeles, CA - Site class D

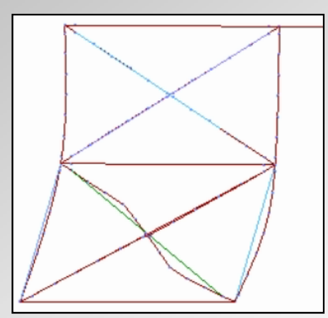


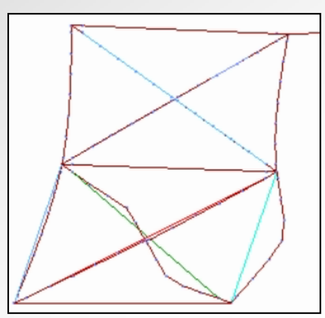
Braces: HSS 102x102x6.4
Columns: W310x174
Strut: W250x58

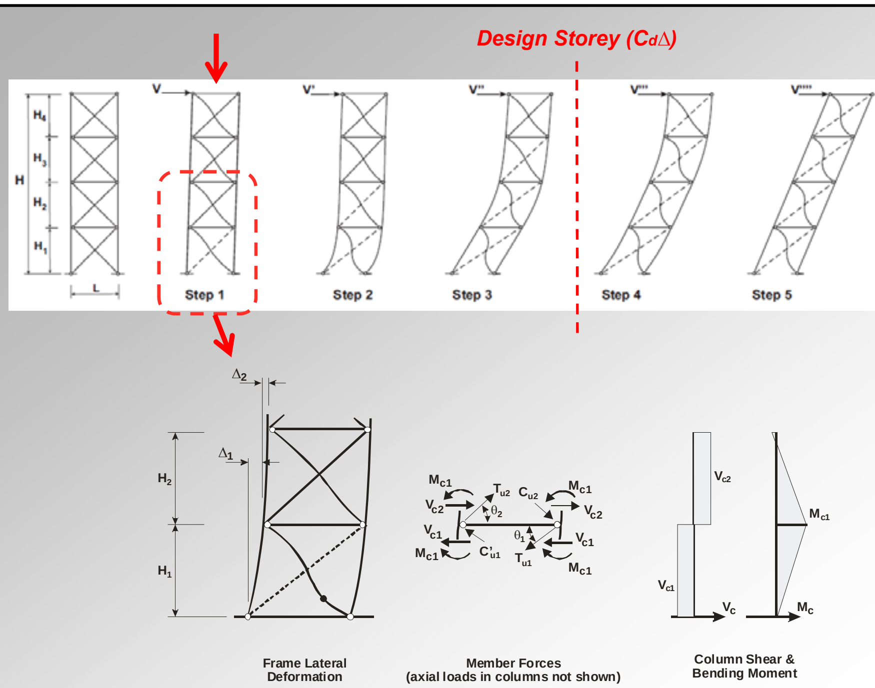
Design Storey Drift ($C_d\Delta/H$) = 0.86% < 2%











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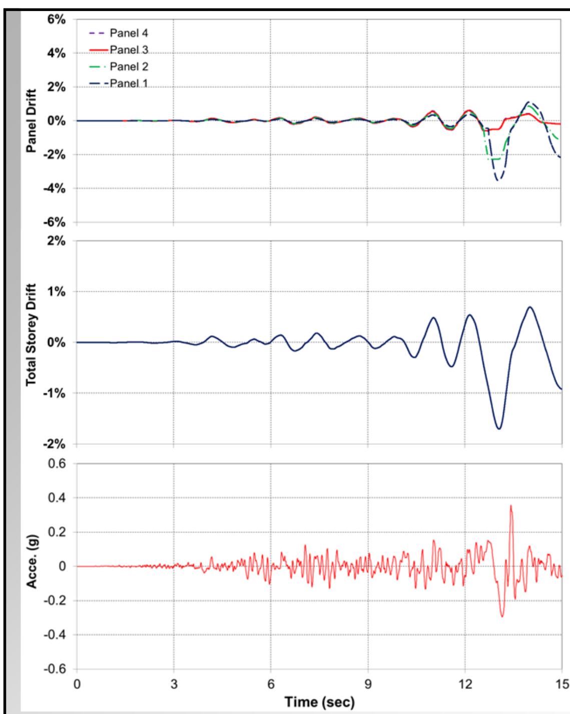

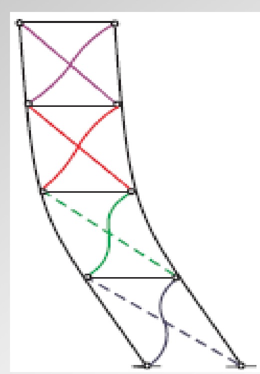

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Conclusions

- ***Design provisions to achieve ductile seismic performance for building steel structures are now available for application in practice***
- ***Design objective is to prevent structural collapse and structural damage & residual deformations are expected***
- ***Some issues still need to be addressed***
- ***Application of this design approach not suitable for heavy industrial applications; specific design provisions needed for these structures***