SOME DESIGN ISSUES WITH SELECTION OF STEEL MATERIALS

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Presentation Outline

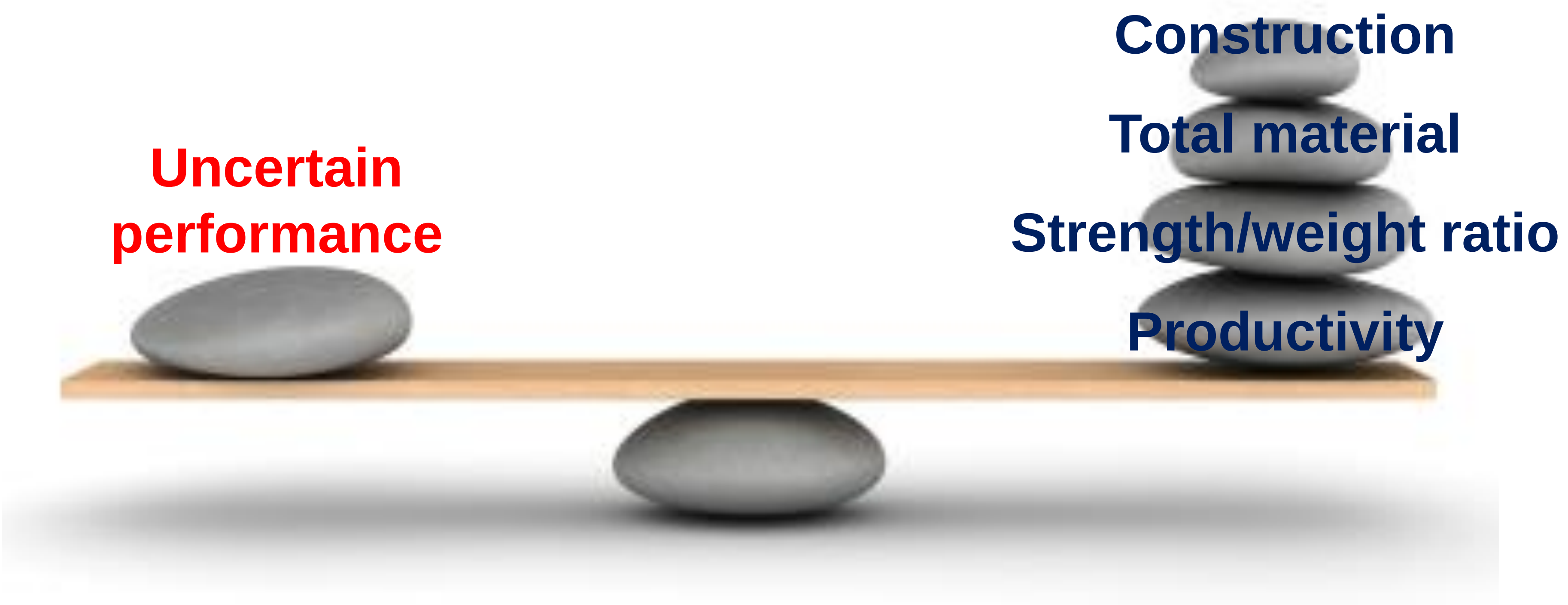
- High Strength Steel = Construction Productivity
  - Normal Strength (≤460 N/mm^2) vs. High Strength (≥460 N/mm^2) Structural Steel
  - Structural Steel vs. Reinforcing Steel
- Current Design Issues in using High Strength Reinforcing Bars in EC2
- Current Design Issues in using High Strength Steel Reinforced Concrete (SRC) Columns in EC4
- Current Restrictions in Extension of Existing Design Rules up to Grade S690 in EC3-1-12.

High Strength Steel

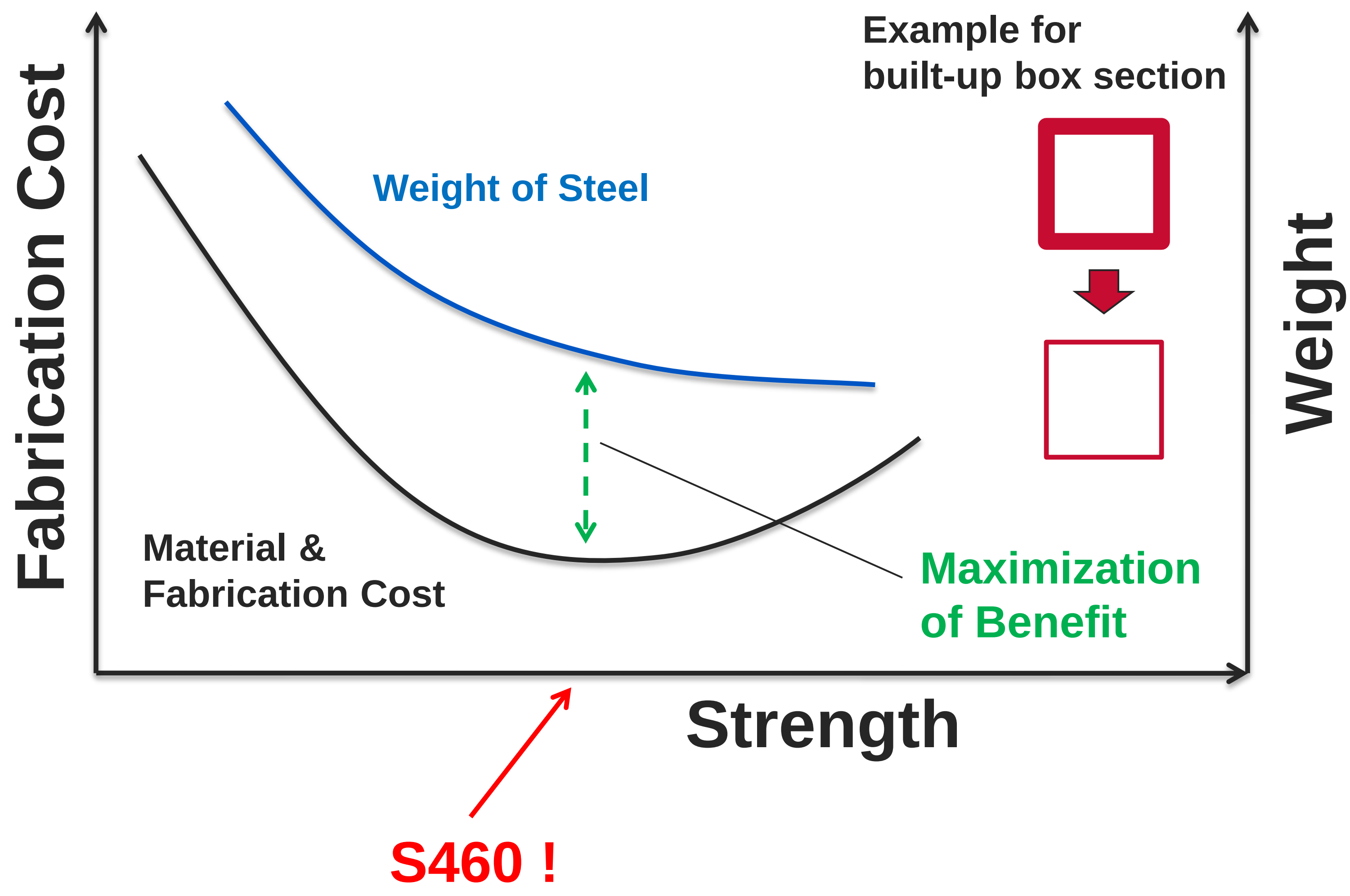
=

Construction Productivity

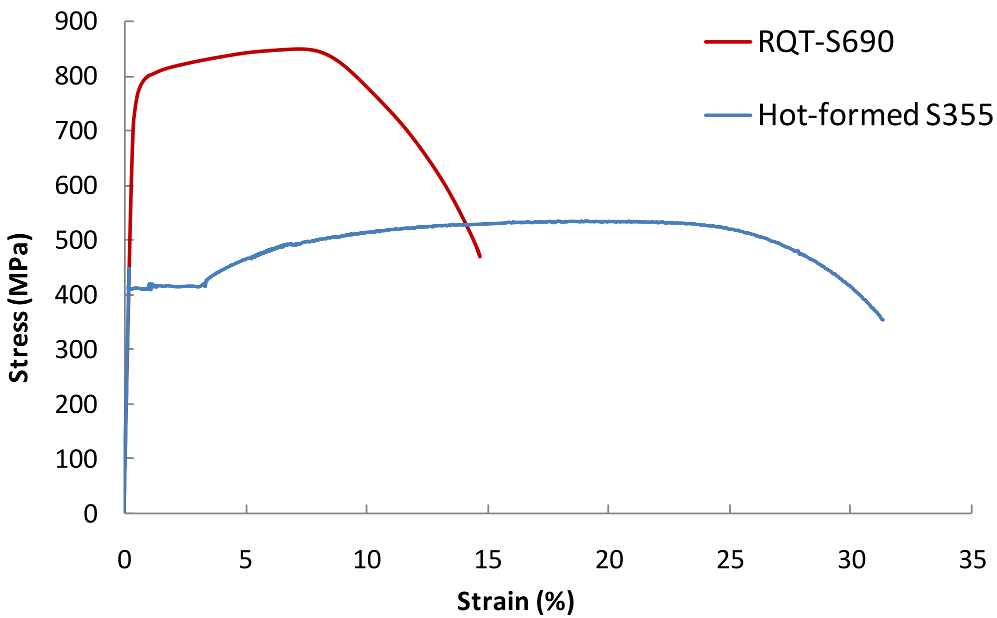
Replacing NSS with HSS



Benefit of using HSS



Structural Steel: S690 vs. S355



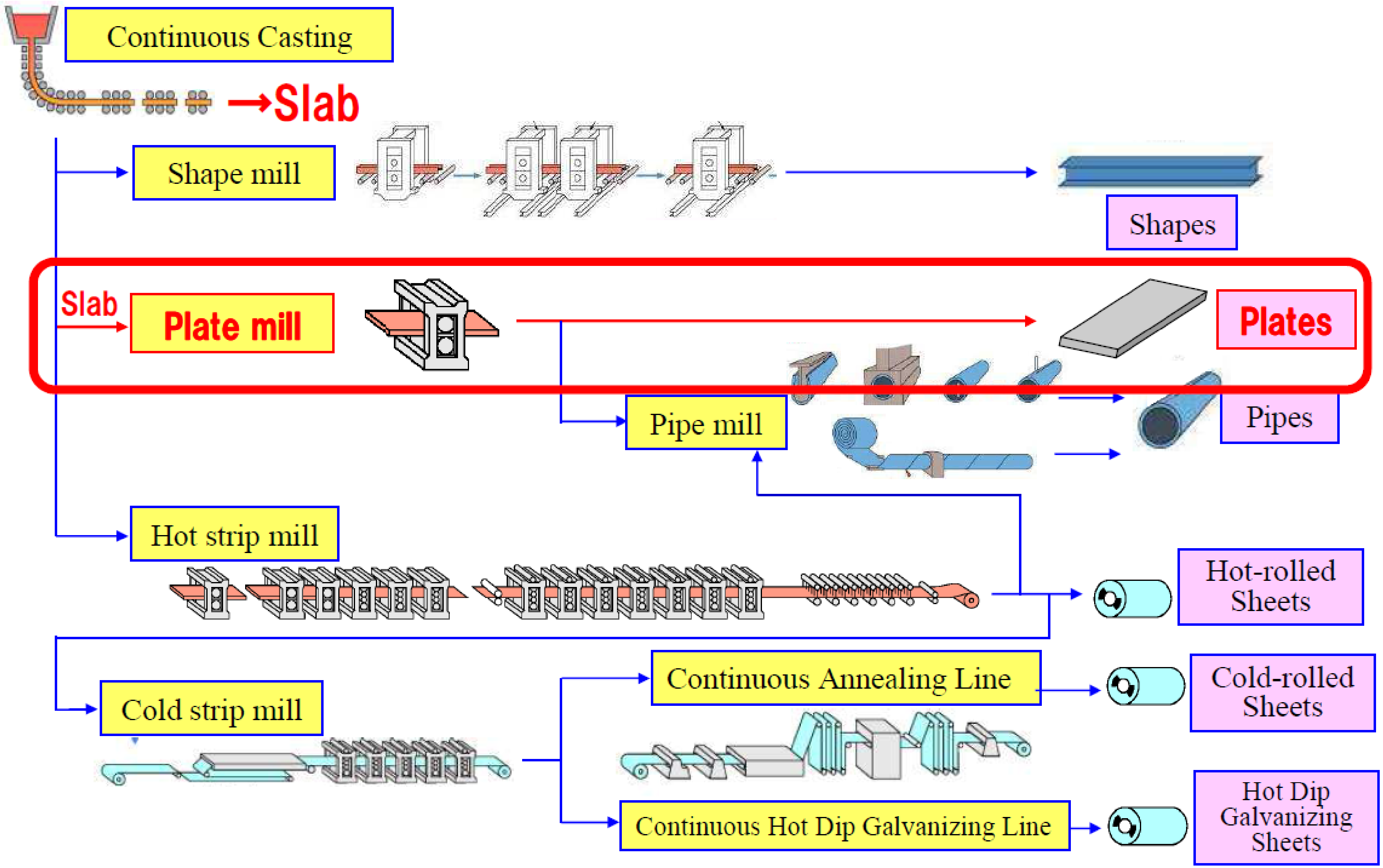
Strength ⬆ **100%**

Ductility ⬇ **100%**

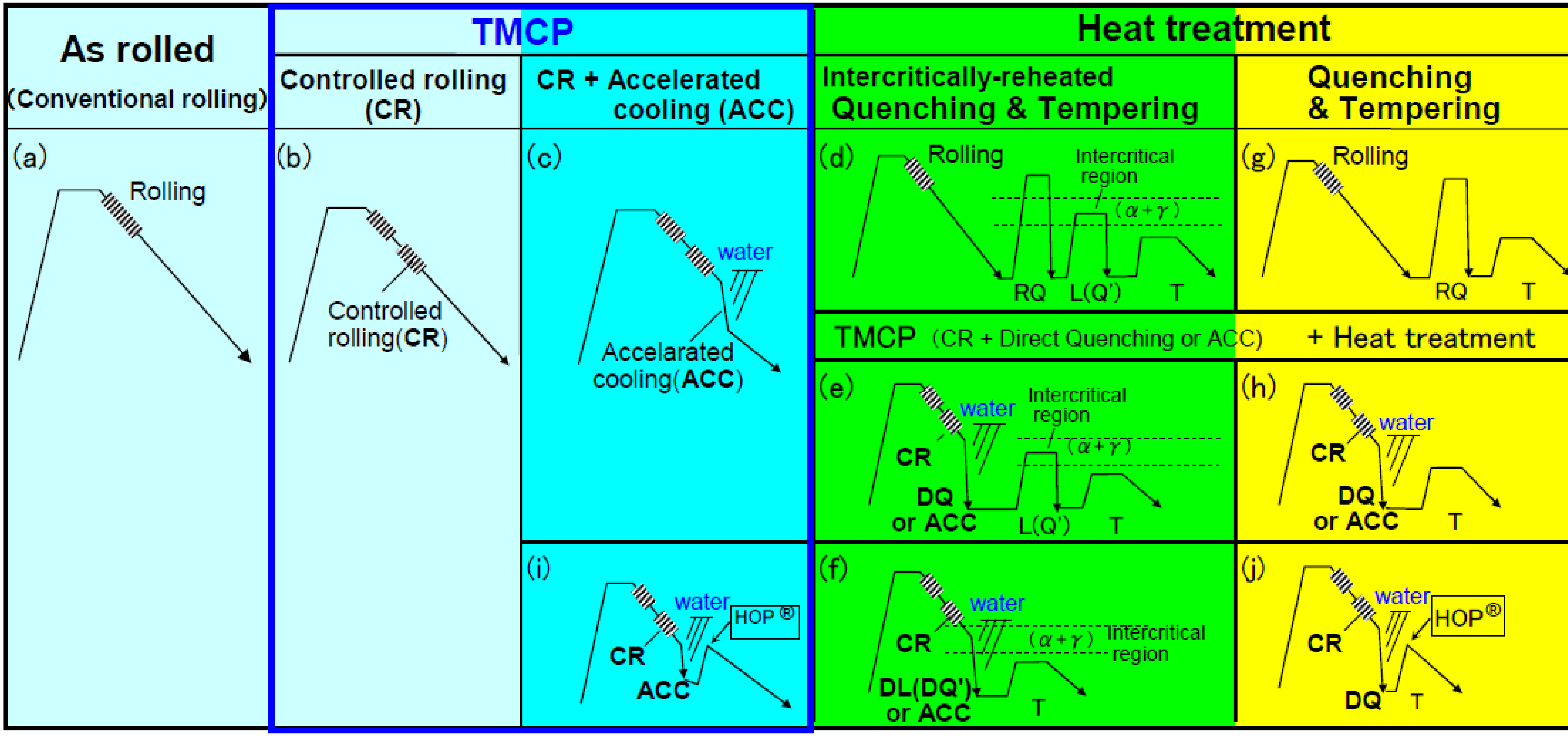
	C	Mn	Cu	P	S	Al	Ti	Si	Cr	Mo	V	Ni	B	CE
S355	0.15	1.33	0.032	0.031	0.009	0.044	<0.001	0.34	0.032	0.008	0.006	0.023	-	0.41
RQT-S690	0.14	1.35	0.01	0.012	0.003	0.035	0.025	0.4	0.01	0.12	0.05	0.01	0.002	0.4

Improve strength mainly by controlled-rolling, quenching and tempering

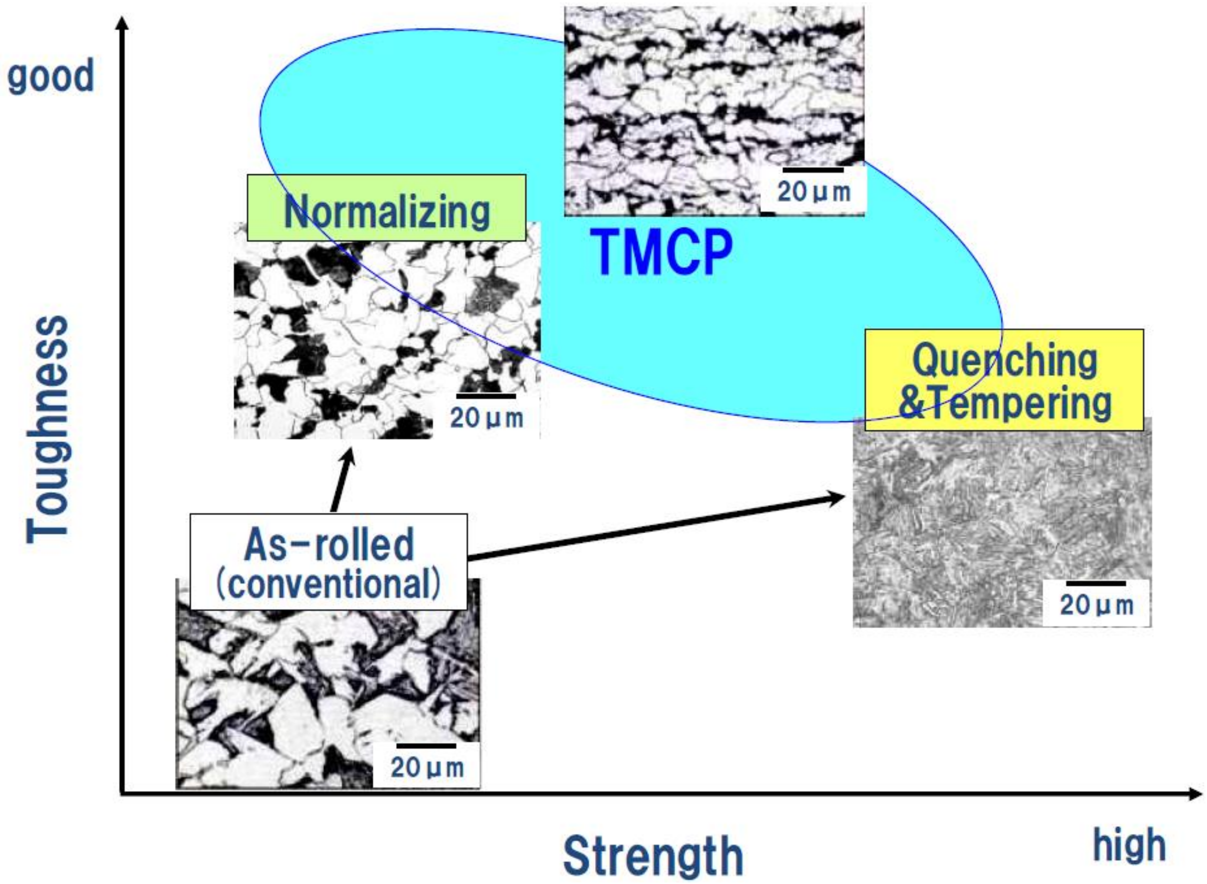
Manufacturing of Structural Steel



High Performance Steel Plates



Effect of Heat Treatment



Structural Steel vs. Reinforcing Steel

Trend is towards use of higher grade but more stringent ductility requirements in terms of tensile/yield strength ratio and elongation.

	Reinforcing Steel			Structural Steel	
	A	B	C	Normal strength	High strength
Yield strength (MPa)	400 to 600			≤ 460	≥ 460 ≤ 690
Modulus of elasticity (GPa)	200			210	
f_t/f_y or f_u/f_y	≥ 1.05	≥ 1.08	≥ 1.15 < 1.35	≥ 1.10	≥ 1.05 ≥ 1.10 (NA)
Elongation (%)	≥ 2.5	≥ 5.0	≥ 7.5	≥ 15	≥ 10
Ultimate strain				$\varepsilon_u \geq 15\varepsilon_y$	

Material Comparison in Eurocodes

		EC2	EC3	EC4
Concrete	Normal	C12/15- C90/105	–	C20/25 - C60/75
	Light weight	LC12/13 – LC80/88		LC20/22 - LC60/66
Reinforcing steel		400 - 600 N/mm^2	–	400 - 600 N/mm^2
Structural steel		–	≤ 690 N/mm^2	≤ 460 N/mm^2

Same trend towards use of higher grade concrete, leads to greater construction productivity.

However, the ranges in EC4 are more restricted than those in EC2 and EC3, **WHY?**

Current Design Issues in using High Strength Steel Reinforcing Bars in EC2

Steel Rebars (from SS2 to SS560)

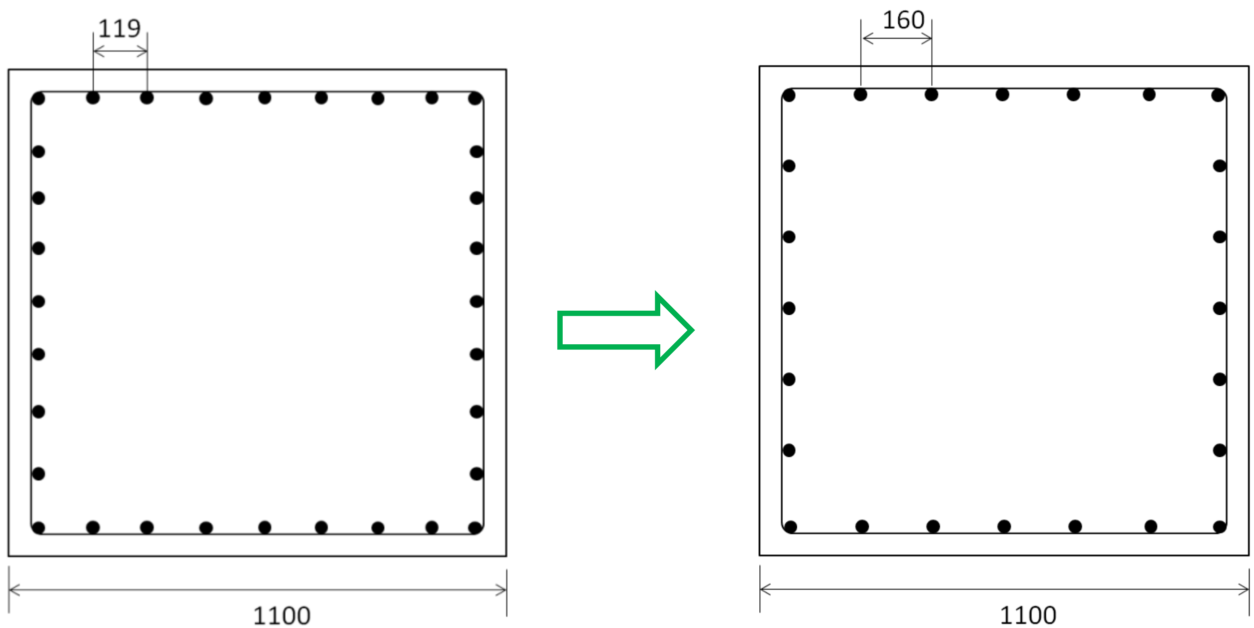
Mechanical Properties

Standard	Grade	Yield strength R_e (MPa)	Tensile/yield strength ratio, R_m/R_e	Elongation at fracture A_5 %	Elongation at maximum force, A_{gt} %
SS2: 1987	460	460	1.15	12	-
SS2: 1999	500	500	1.05	14	-
SS560: 2016	B500A	500	1.05	-	2.5
	B500B	500	1.08	-	5.0
	B500C	500	≥ 1.15, < 1.35	-	7.5
	B600A	600	1.05	-	2.5
	B600B	600	1.08	-	5.0
	B600C	600	≥ 1.15, < 1.35	-	7.5

Benefits of Grade 600 Rebar

Item	Description
Steel Saving	Potential to reduce steel reinforcement – up to 20% compared to current Grade 500 rebar
Steel Fabrication	Up to 20% less workers are needed
Logistics	Less trucks carrying steel reinforcement on the roads – up to 20% less
Site Crane	Handles up to 20% less steel and frees up crane time for other construction activities thereby speeding up construction
Concrete Saving	Reduction in structural element size is possible when used together with appropriate higher grades of concrete which will result in further overall dead weight reduction
Storage Space	Space required for site storage of steel reinforcement can be reduced by about 20%
Time Reduction	Overall time savings can be accomplished by factoring in the above items
Cost Reduction	Overall cost reduction can be achieved from reduced usage of material, manpower, construction time, etc

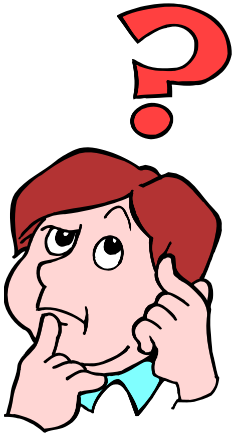
Benefits of Grade 600 Rebar ?



Cross-section: 1100mm x 1100mm
Rebar: Grade 500
Concrete: C50/60
Longitudinal rebar: 32 Φ25

Cross-section: 1100mm x 1100mm
Rebar: Grade 600
Concrete: C50/60
Longitudinal rebar: 24 Φ25

20 % saving



Benefits of Grade 600 Rebar ?

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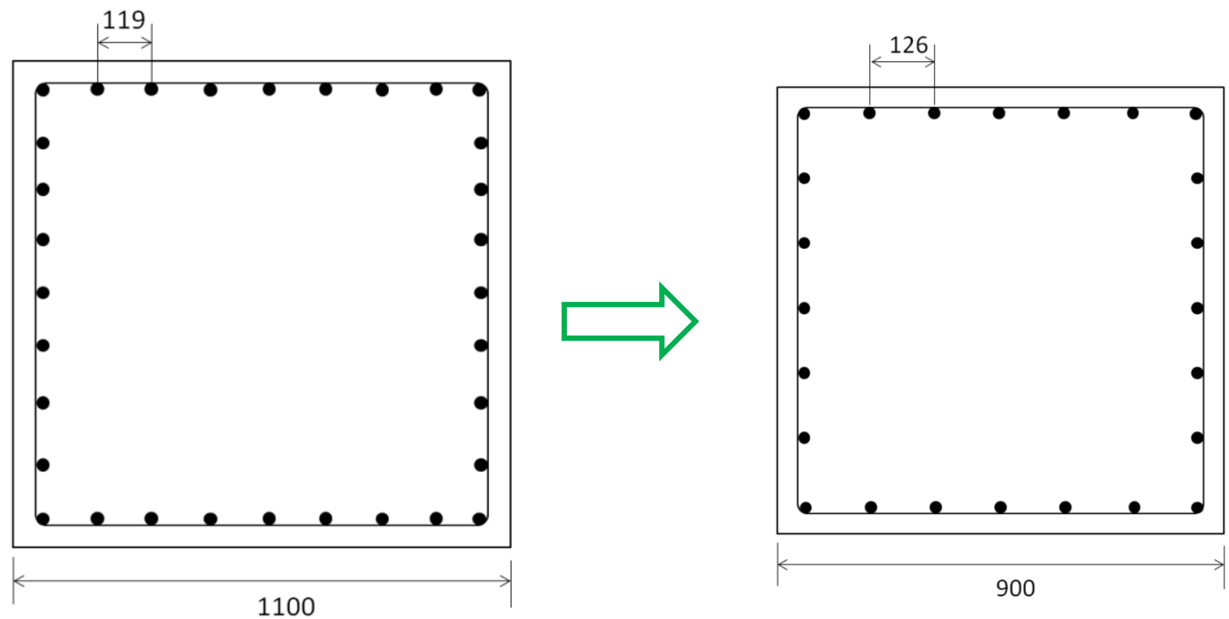
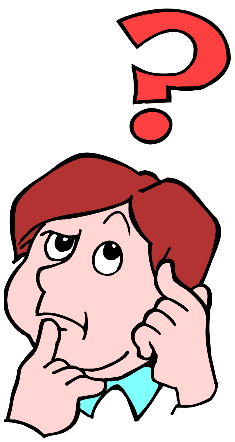


Cross-section: 1100mm x 1100mm
Rebar: Grade 500
Concrete: C50/60
Longitudinal rebar: 32 Φ25

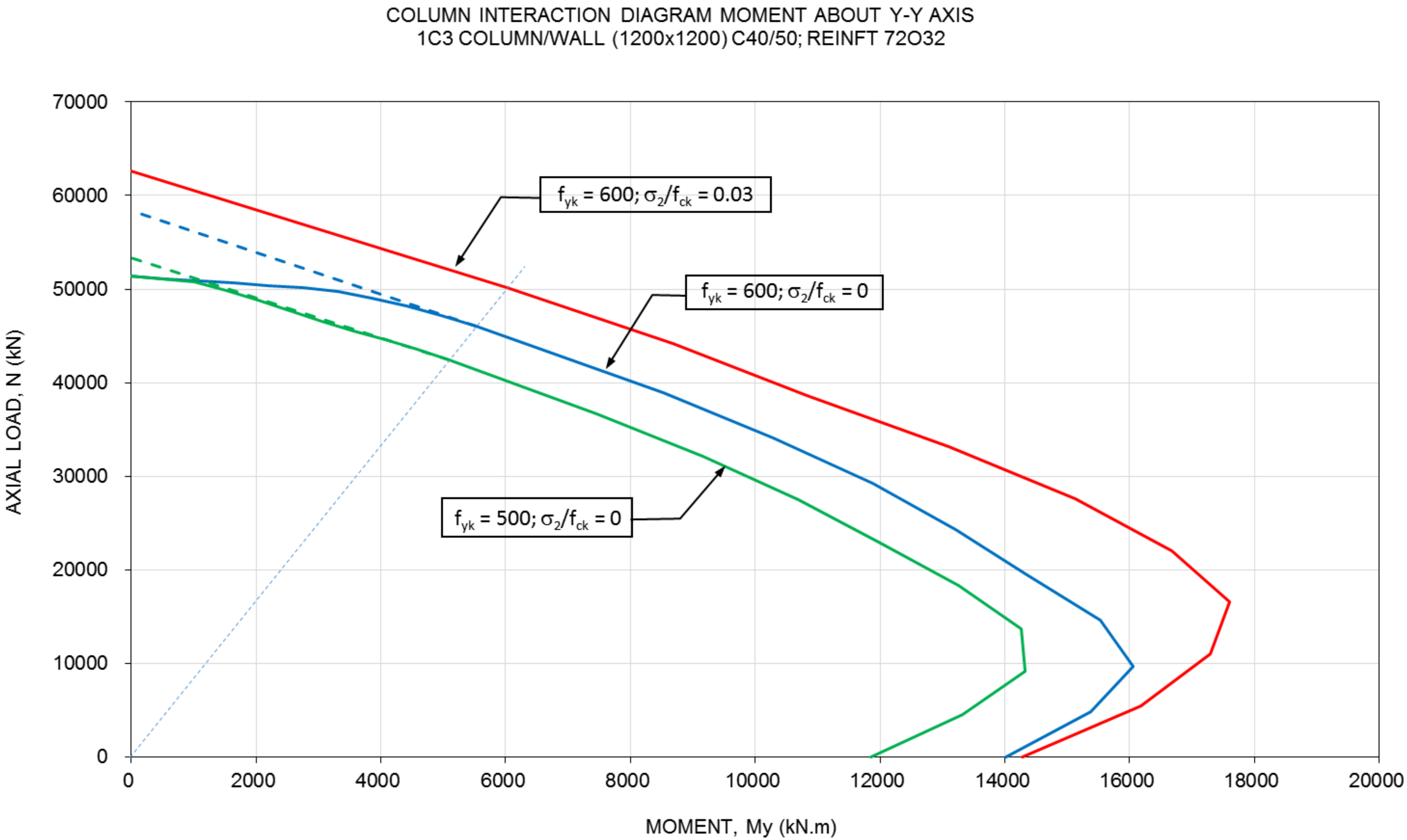
Cross-section: 900mm x 900mm
Rebar: Grade 600
Concrete: C90/105
Longitudinal rebar: 24 Φ25

Rebar:
20 % saving

Concrete:
33% saving



But how to achieve these savings ...?

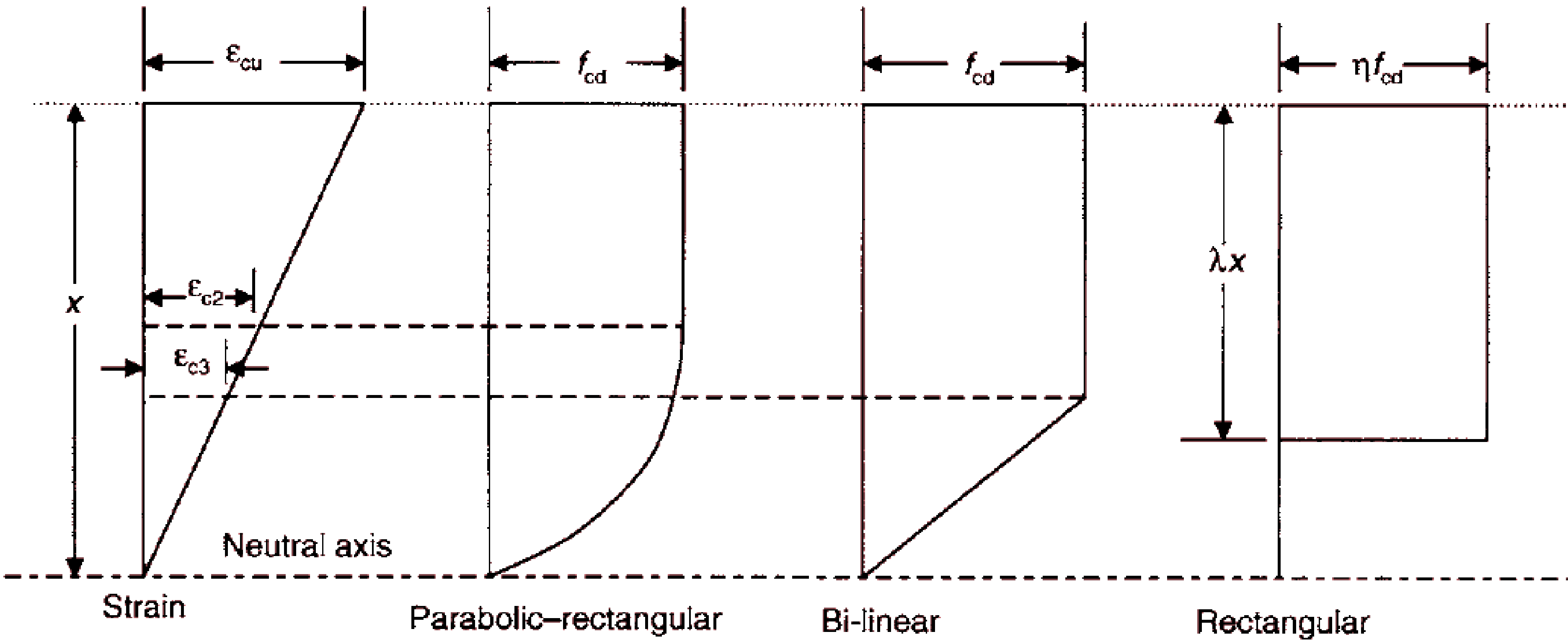


Design Provisions in EC2

Stress-strain relationships

Three types are allowed in SS EN1992-1-1, i.e.

- Parabolic-rectangular diagram
- Bi-linear stress-strain diagram
- Rectangular diagram

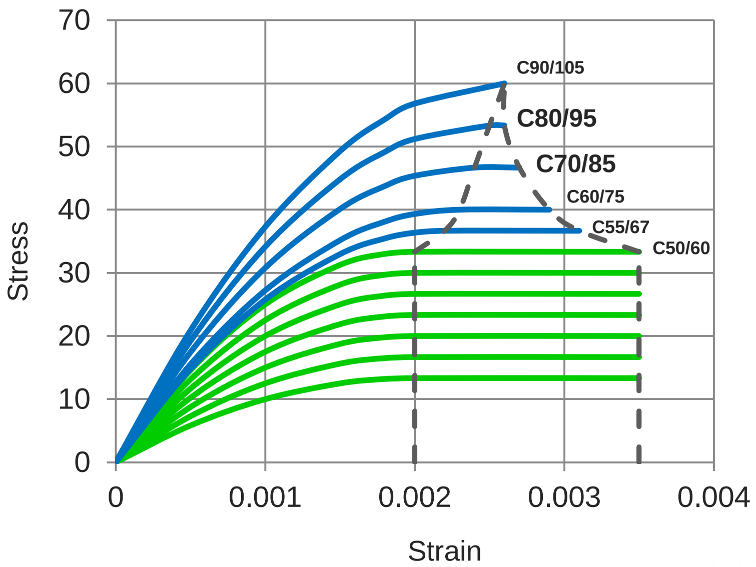


Design Provisions in EC2

Concrete compression strain

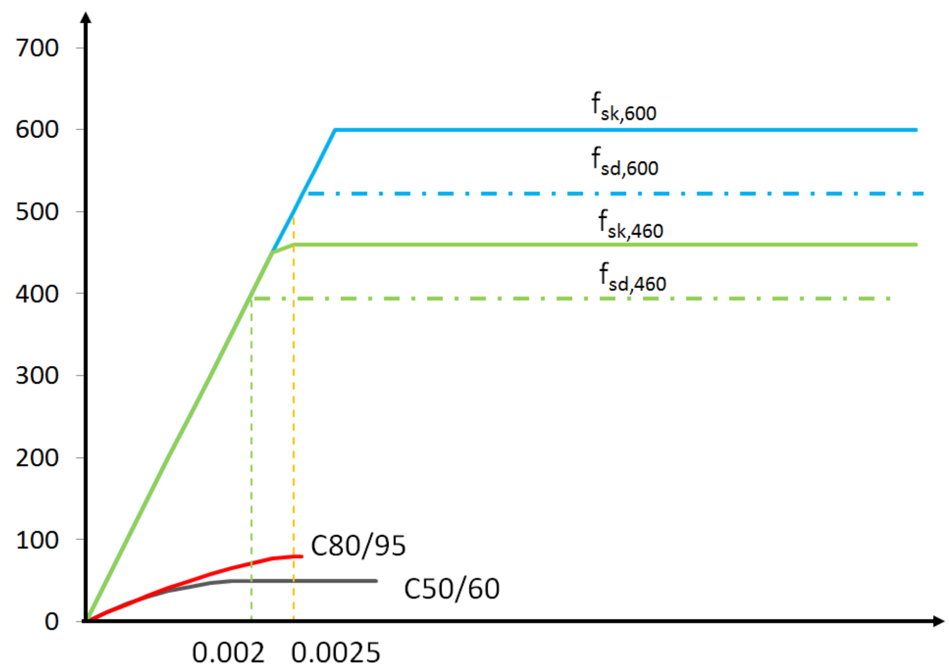
- In design, failure of concrete in compression is defined by means of limiting compressive strains.
- EN1992-1-1 adopts a limit of ε_{cu2} (or ε_{cu3} if bi-linear diagram is used) for flexural, a limit of ε_{c2} or ε_{c3} for pure axial compression, and a interpolation between the value of ε_{cu2} for flexure and ε_{c2} for axial load for combined bending and compression.

Grade	ε_{cu}	ε_{c2}	ε_{c3}
≤ C50/60	**0.0035**	**0.0020**	**0.00175**
C55/67	0.0031	0.0022	0.00180
C60/75	0.0029	0.0023	0.00190
C70/85	0.0027	0.0024	0.00200
C80/95	0.0026	0.0025	0.00220
C90/105	0.0026	0.0026	0.00230



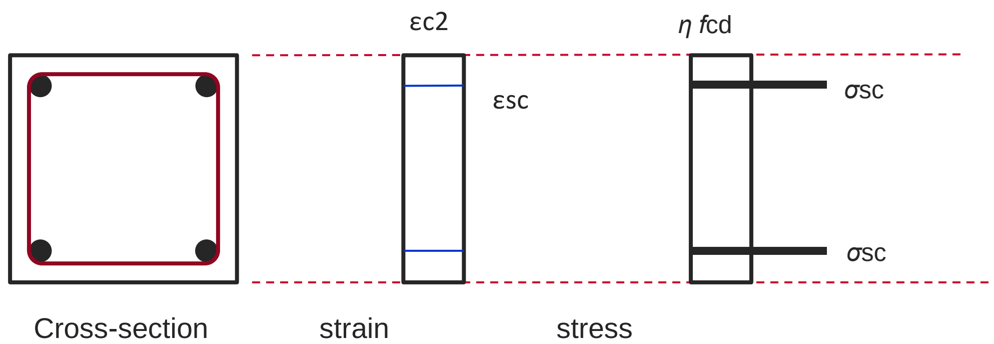
Strain Compatibility

When high strength steel rebar is used in RC column, there is much concern about early concrete crushing; when the yield strain of the steel exceeds the crushing strain of concrete (generally, ε_c = 0.002 (≤C50/60) for pure compression), concrete crushing occurs before yielding of the reinforcing steel. Thus, the high strength steel rebar cannot develop its full yield strength, and there is no benefit in using it at all.



Axially Loaded Columns

The maximum pure compressive strain is ε_{c2} or ε_{c3} when the whole section is under pure compression.



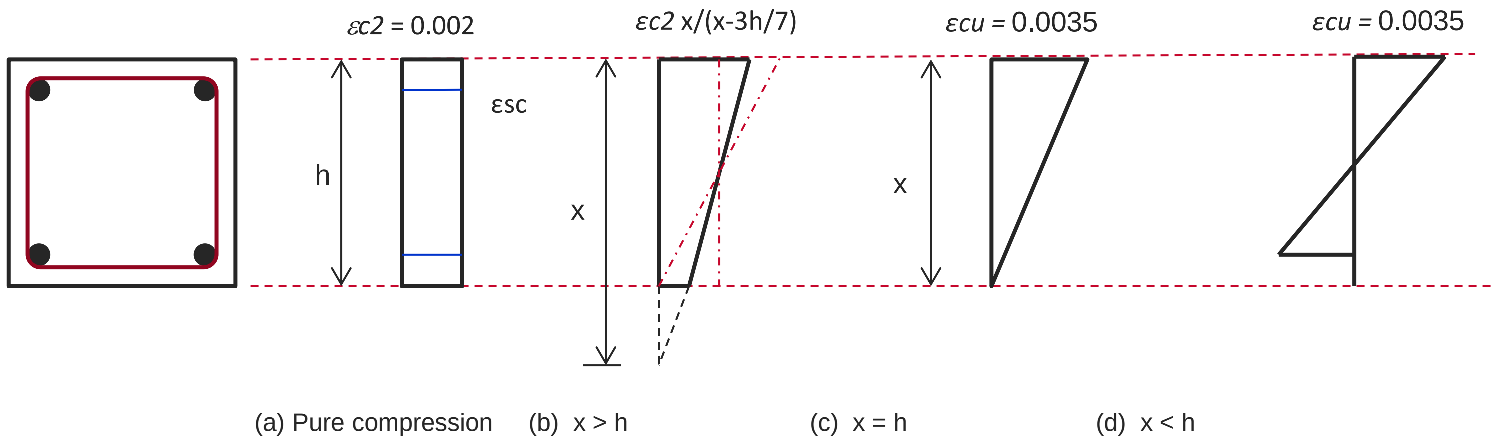
Limiting Concrete Strain & Maximum Strength of Grade 600 Rebar

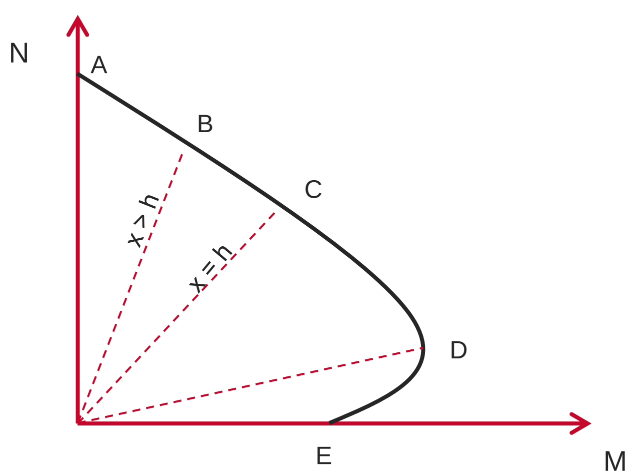
Grade	ε_{c2}	ε_{c3}	$f_{y,\varepsilon_{c2}}$	$f_{y,\varepsilon_{c3}}$
≤ C50/60	**0.0020**	**0.00175**	**460**	**403**
C55/67	0.0022	0.00180	506	414
C60/75	0.0023	0.00190	529	437
C70/85	0.0024	0.00200	552	460
C80/95	0.0025	0.00220	575	506
C90/105	0.0026	0.00230	598	529

$$f_{y,\,\varepsilon_c} = E_s \varepsilon_{sc} \gamma_s \le f_{yk}$$

Columns under Compression and Bending

The maximum compressive strain is assumed to lie between ε_{c2} *(or* ε_{c3}*)* and ε_{cu} when the section is in compression and bending.



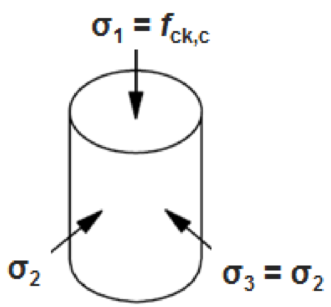


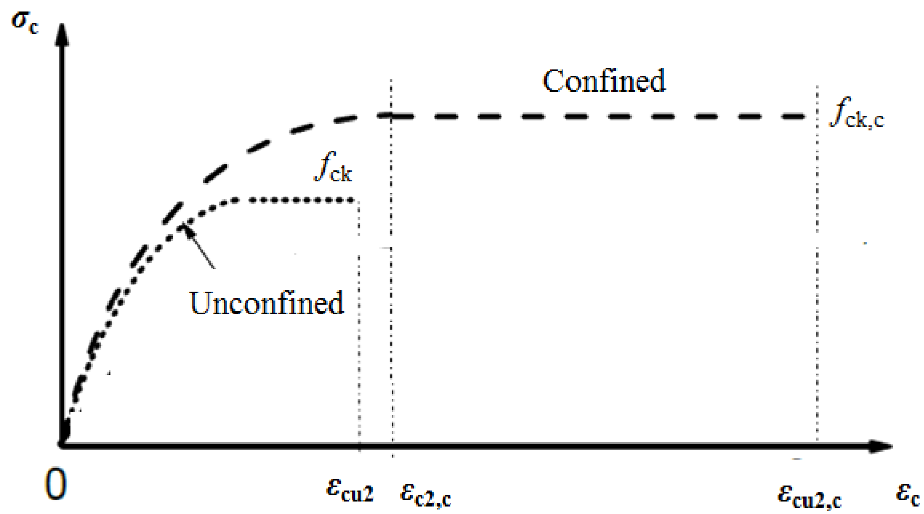
For case (a) and (b), strain compatibility issue should be considered.

Confined Concrete Strain

Confinement can be generated by adequately closed <u>hoops or links</u>. This results in higher strength and higher critical strain.

<u>SS EN1992-1-1 Clause 3.1.9</u>





$$f_{ck,c} = f_{ck}\left(1.0 + 5.0\,\sigma_2 / f_{ck}\right) \quad \text{for} \quad \sigma_2 \le 0.05 f_{ck}$$

$$f_{ck,c} = f_{ck}\left(1.125 + 2.5\,\sigma_2 / f_{ck}\right) \quad \text{for} \quad \sigma_2 > 0.05 f_{ck}$$

$$\varepsilon_{c2,c} = \varepsilon_{c2}\left(f_{ck,c} / f_{ck}\right)^2$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2\,\sigma_2 / f_{ck}$$

Confined Concrete Models

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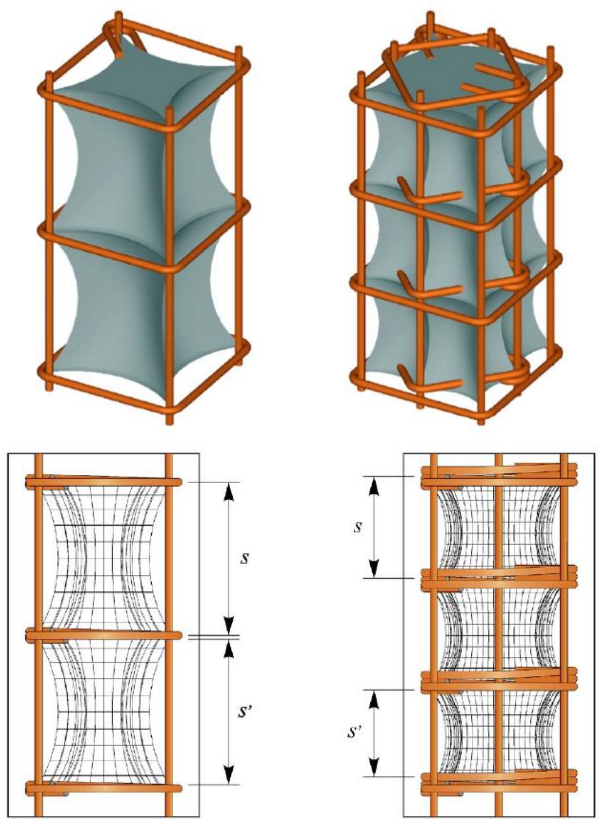
The confinement depends on many factors including

- The diameter, layout, spacing and number of the longitudinal reinforcement bars
- The diameter and spacing of the transverse reinforcement bars
- Yield stresses of the reinforcement bars
- Concrete strength



Various confinement models under study:

1.	CEB-FIP Model Code 1990
2.	FIB Model Code 2010
3.	JB Mander's Confined Concrete Model
4.	D Cusson's Confined Concrete Model
5.	F Legeron's Confined Concrete Model

Concrete Confinement

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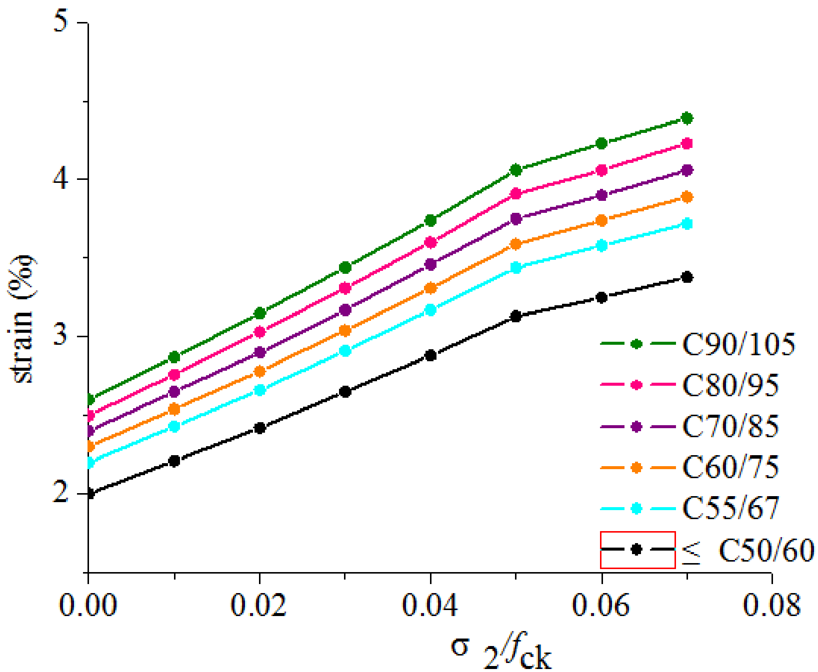
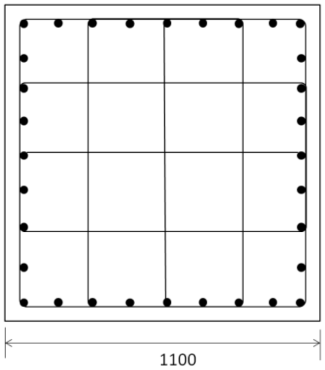
The confining pressure provided by lateral hoops or links results in an enhancement in the strength and ductility of the concrete. If the concrete is well confined, the full yield strength of the steel reinforcing bar may be developed by the increased strain of the confined concrete.

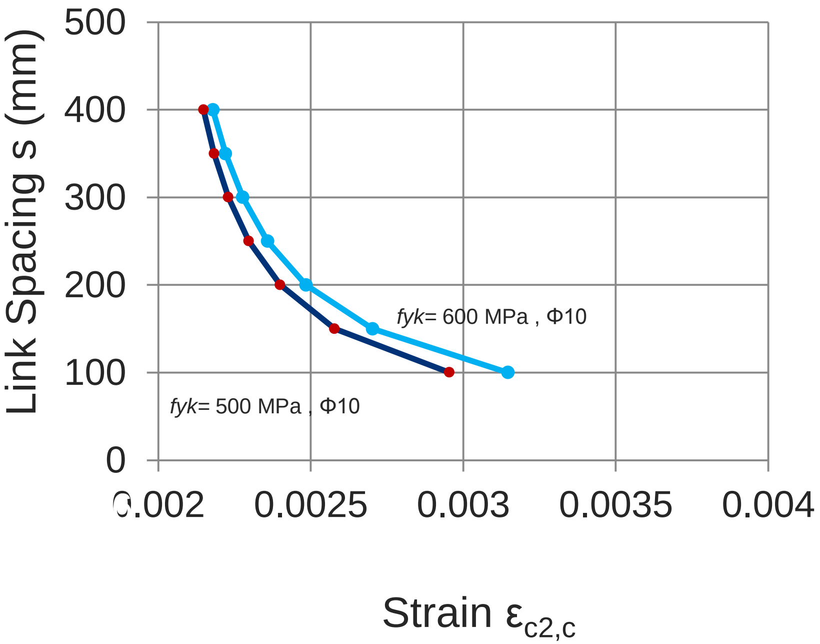


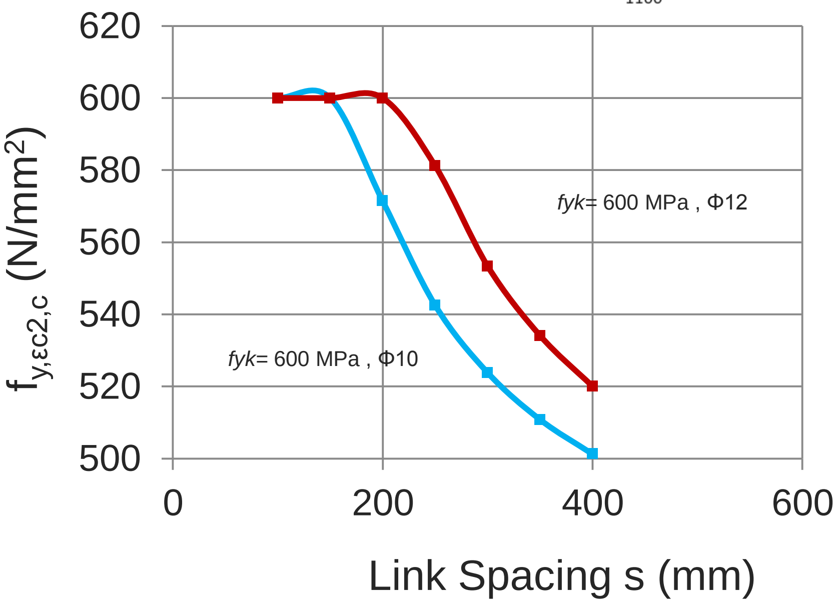
Maximum strength of Grade 600 rebar

	Unconfined	Confined Concrete	
	$f_{y,\varepsilon 2}$	σ_2/f_{ck}	$f_{y,\varepsilon 2,c}$
≤ C50/60	460	≥0.029	600
C55/67	506	≥0.018	600
C60/75	529	≥0.014	600
C70/85	552	≥0.009	600
C80/95	575	≥0.005	600
C90/105	598	≥0.001	600

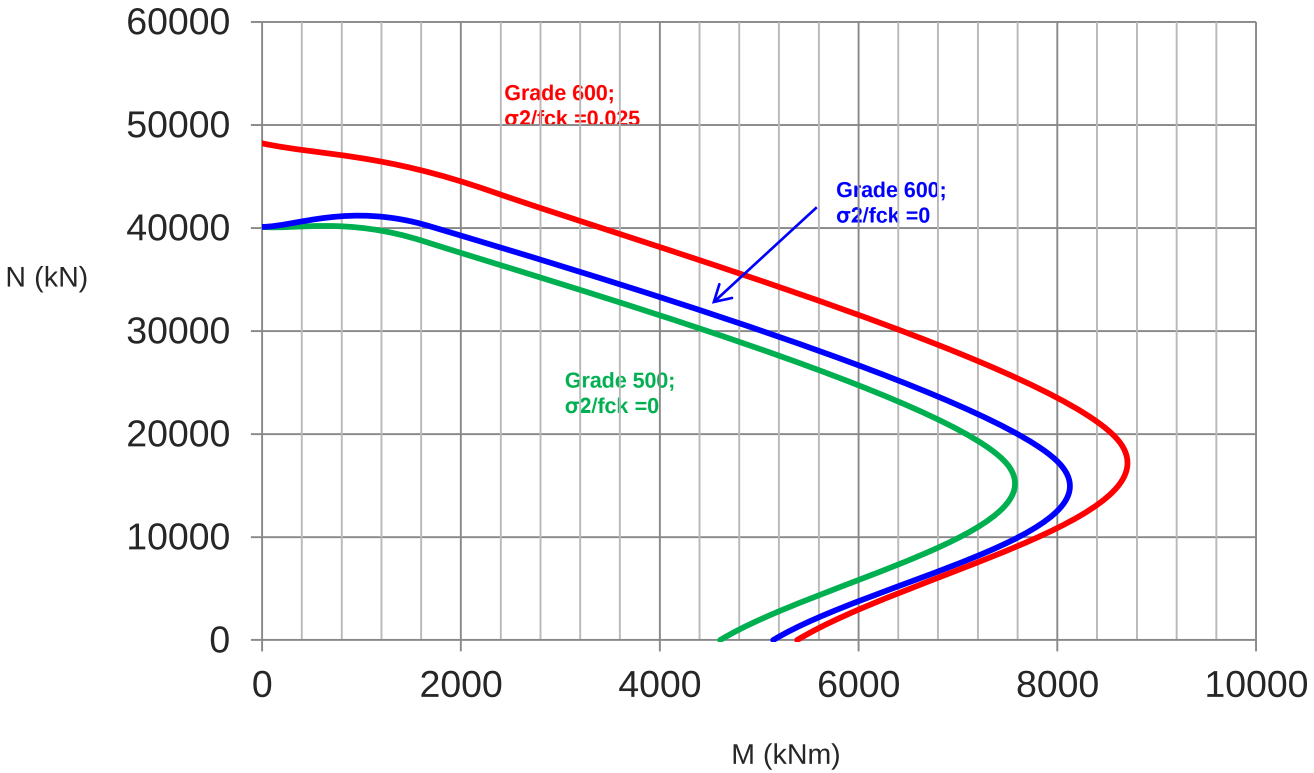
Concrete Confinement

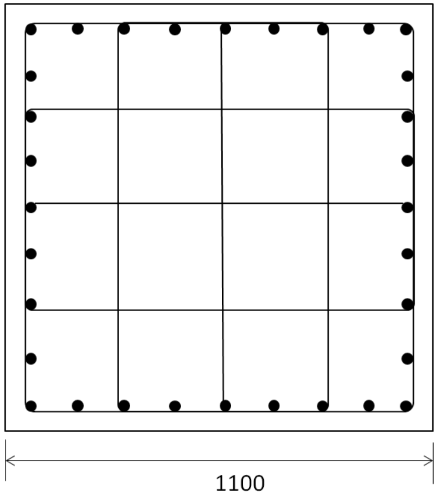






Concrete Confinement





Current Design Issues in using High Strength Steel Reinforced Concrete (SRC) Columns in EC4

Composite Columns in EC4

Steel:

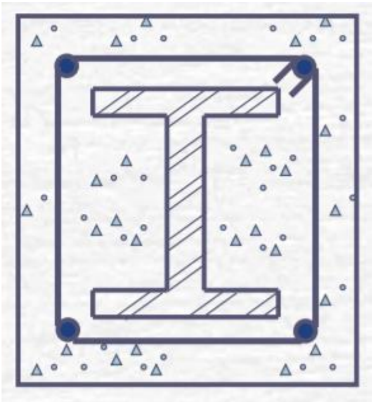
High Strength
High Ductility

Concrete:

Lower Cost
Good Fire Resistance

Composite Columns:

- Achieve overall enhancement in strength and stiffness
- Provide fire-protection and buckling resistance for steel section



SRC

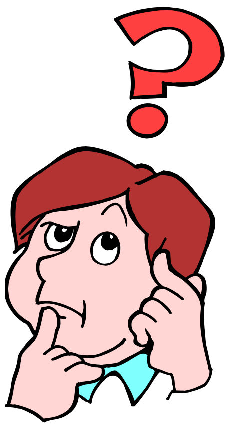
CFT

Limitation of Design Codes

Codes	Steel yield strength (N/mm^2)	Concrete cylinder strength (N/mm^2)
EC4	235 ~ 460	20 ~ 60
EC2	-	12 ~ 90
EC3	235 ~ 690	-

The ranges in EC4 are more restricted than EC2 and EC3, the question is WHY?

- Lack of test data and experience in designing composite members with high strength materials
- Strain compatibility problem



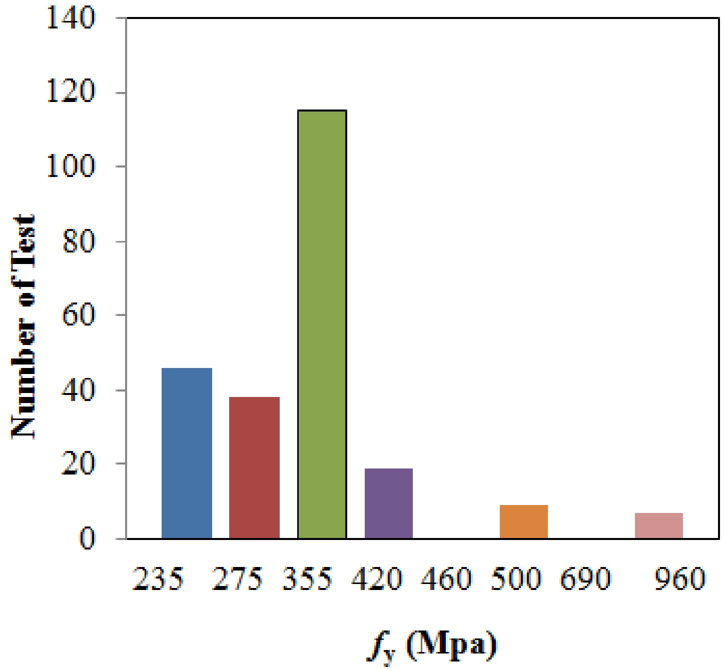
Concrete Filled Tubular (CFT) Columns

Concrete Filled Tubular Columns with High Strength Materials - An Extension of Eurocode 4 Method to C90/105 Concrete and S550 Steel

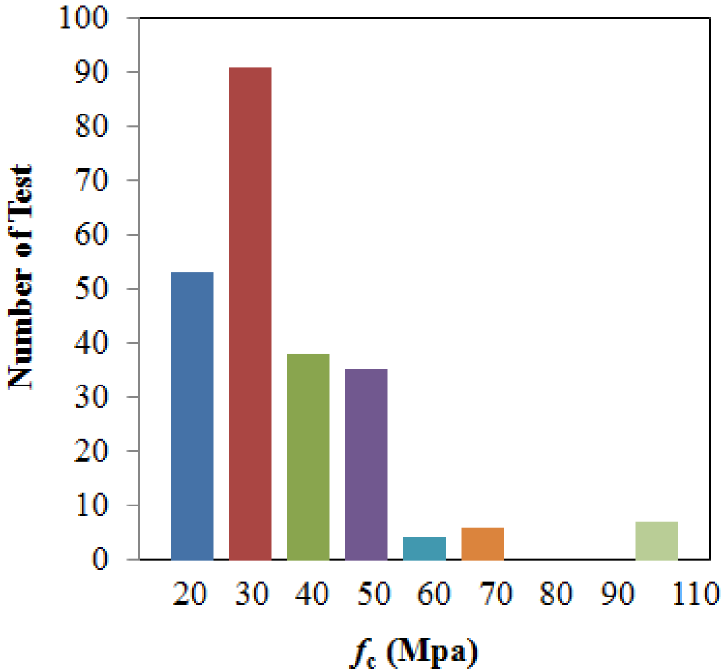
Took advantage of concrete confinement provided by the outer steel tube and validated against test database !

Question: what about SRC Columns?

Existing Test Data of SRC Columns



Distribution of f_y for SRC column tests



Distribution of f_c for SRC column tests

Test data is insufficient to establish the validity of using high strength materials in SRC columns.

Comparison Study

- **SRC columns with normal strength materials (≤ S460)**

  ($f_c \leq 50$ MPa and $f_y \leq 460$ MPa)

	Test/EC4
Mean Value	1.21

Code Conservative

Safe !

- **SRC columns with high strength materials (≥ S460)**

  ($f_c > 50$ MPa and $f_y > 460$ MPa)

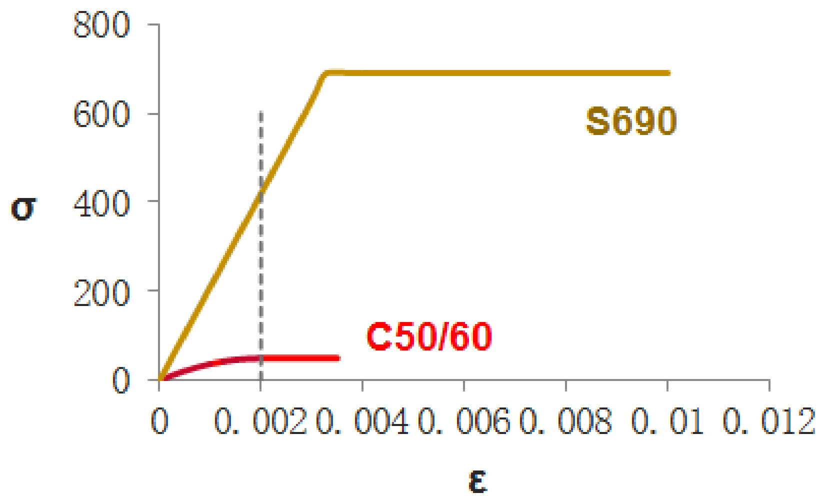
	Test/EC4
Mean Value	0.87

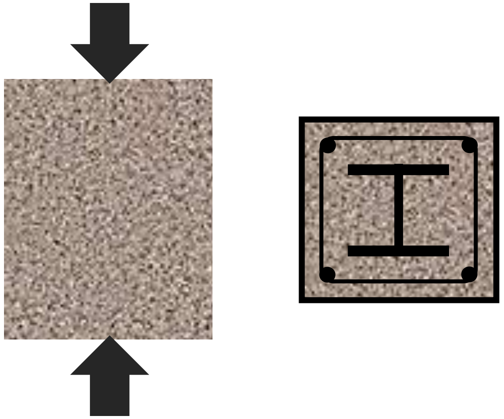
Code Overestimates Test Results

Not Safe !

Why Not Safe?

- Much concern about early concrete crushing in SRC columns with high strength steel.
- If the yield strain of the steel exceeds the crushing strain of concrete, concrete crushing occurs <u>before</u> yielding of the steel – ‘strain compatibility’.
- High strength steel cannot develop its full yield strength.
- **Full Plastic Method in EC4 is not applicable for high strength steel (f_y ≥ S460).**





Strain Compatibility Method

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Maximum Strength of Steel before Concrete Crushes

Grade	S235	S275	S355	S420	S460	S500	S550	S620	S690
≤ C50/60	235	275	355	420	420	420	420	420	420
C55/67	235	275	355	420	460	464	464	464	464
C60/75	235	275	355	420	460	483	483	483	483
C70/85	235	275	355	420	460	500	504	504	504
C80/95	235	275	355	420	460	500	525	525	525
C90/105	235	275	355	420	460	500	546	546	546

Strain of Concrete at Peak Strength

Grade	ε_{c2} (‰)
≤ C50/60	**2.0**
C55/67	2.2
C60/75	2.3
C70/85	2.4
C80/95	2.5
C90/105	2.6

$$\begin{cases} \varepsilon_\mathrm{y} = f_y / E_a \\ \varepsilon_\mathrm{y} \le \varepsilon_\mathrm{c} \end{cases}$$

$$f_{\mathrm{y,\ max}} = E_a \varepsilon_\mathrm{c} \le f_\mathrm{y}$$

Yield Strain of Steel

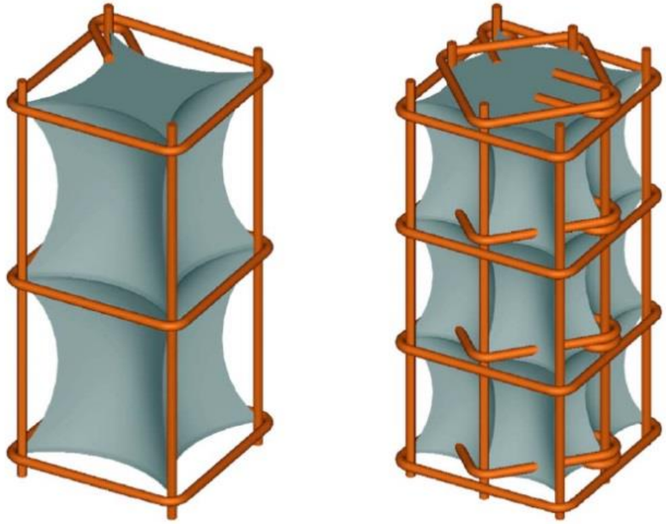
Steel grade	ε_y (‰)
S235	1.12
S275	1.31
S355	1.69
S420	2.00
S460	2.19
S500	2.38
S550	2.62
S620	2.95
S690	3.29

Confined Concrete Method

Confinement can be provided by closed hoops or lateral links. The confining pressure results in higher strength and higher strain of the concrete.

The confinement depends on many factors

- The diameter, layout, spacing and number of the longitudinal reinforcement bars
- The diameter and spacing of the transverse reinforcement bars
- Yield stresses of the reinforcement bars
- Concrete strength

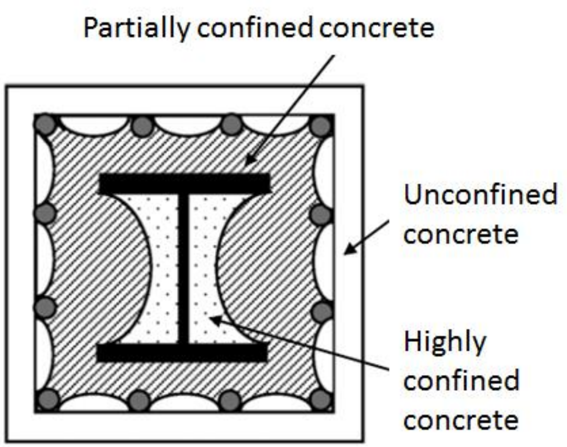


Various possible confinement models:

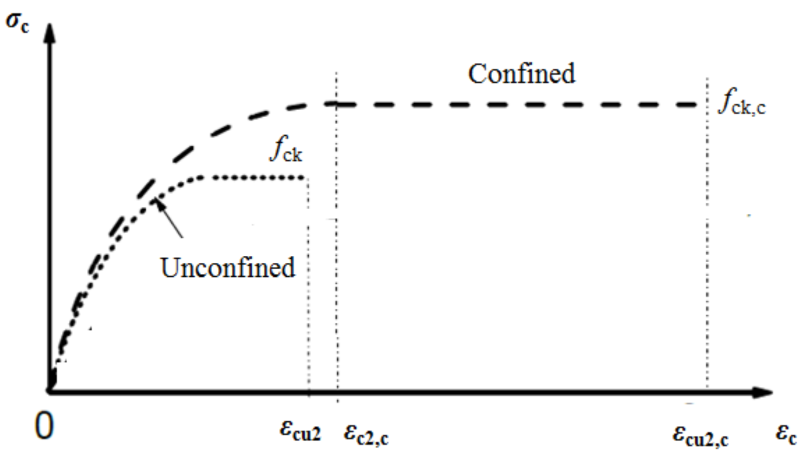
1.	CEB-FIP Model Code 1990
2.	FIB Model Code 2010
3.	JB Mander's Confined Concrete Model
4.	D Cusson's Confined Concrete Model
5.	F Legeron's Confined Concrete Model

Confined Concrete Method

- For SRC columns, confinement can be provided by the lateral links and steel section.
- Eurocodes only provides the stress-strain relationship of confined concrete in RC columns.
- How to predict the stress-strain behaviour of confined concrete in SRC columns?



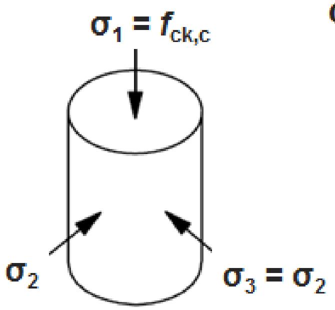
Confinement zones in SRC columns

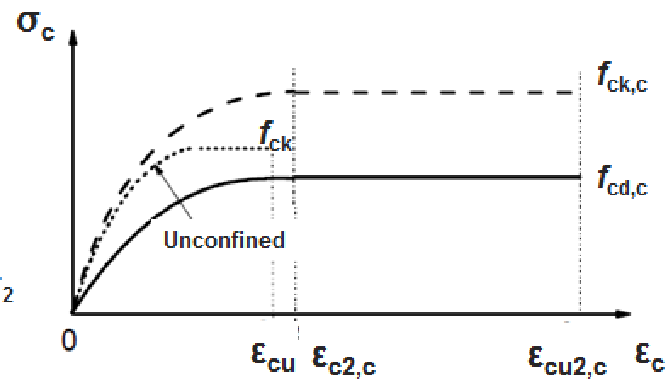


Confined concrete Model for RC Members (EC2)

Confined Concrete Method

EN1992-1-1 Clause 3.1.9





$$f_{ck,c} = f_{ck}(1.0 + 5.0\sigma_2 / f_{ck}) \quad \text{for} \quad \sigma_2 \le 0.05 f_{ck}$$

$$f_{ck,c} = f_{ck}(1.125 + 2.5\sigma_2 / f_{ck}) \quad \text{for} \quad \sigma_2 > 0.05 f_{ck}$$

$$\varepsilon_{c2,c} = \varepsilon_{c2}(f_{ck,c} / f_{ck})^2$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2\sigma_2 / f_{ck}$$

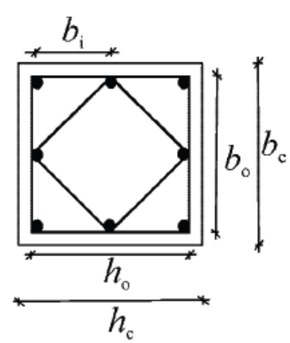
The effective lateral compressive strength due to confinement σ_2:

$$\sigma_2 / f_{ck} = 0.5\alpha\omega_{wd}$$

$$\omega_{wd} = \frac{\text{Volume of hoops}}{\text{Volume of confined concrete}} \frac{f_{yd}}{f_{cd}}$$

$$\alpha = \alpha_n \alpha_s$$

$$\alpha_n = 1 - \frac{\sum b_i^2 / 6}{b_0 h_0} \qquad \alpha_s = \left(1 - \frac{s}{2b_0}\right)\left(1 - \frac{s}{2h_0}\right)$$



EN 1998-1

Comparison Study

Comparison of test data and codes on SRC columns with high strength materials

	Test/EC4	*Test/EC4,u*	*Test/EC4,c*
Mean	0.87	1.11	0.94

EC4 is the resistance by EC4 plastic method;

EC4,u is the resistance by EC4 considering strain-compatibility;

EC4,c is the resistance by EC4 considering strain-compatibility & confinement effect according to EC2;

Conclusions:

- Full Plastic Method in EC4 is unsafe and cannot be used.
- Strain Compatibility Method is too conservative, uneconomical.
- Confinement model in EC2 cannot be directly applied to SRC columns with high strength materials, unsafe to use.
- Requires some modifications for SRC columns.

Current Restrictions in Extension of Existing Design Rules up to Grade S690 in EC3-1-12

Intensification of Low-Density Developments: Functional Bridging Buildings

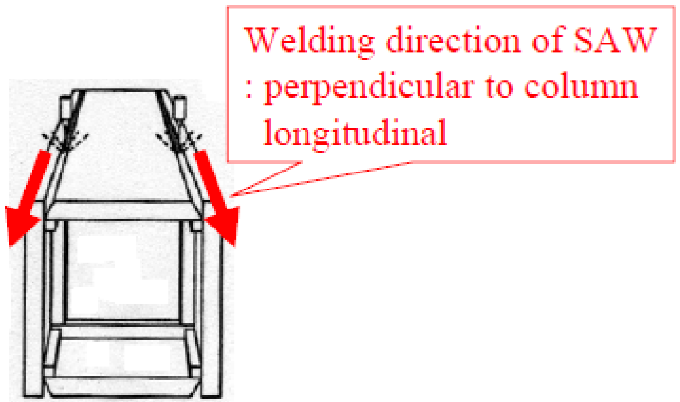
Combined S690 + S355

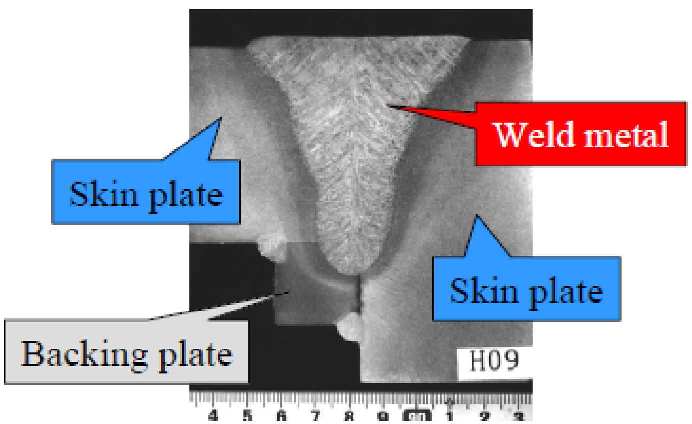
Weight/m^2= 311 kg/m^2 (Grade S690)
+ 150 kg/m^2 (Grade S355)
+ 22 kg/m^2 (Y1860S7 ϕ15mm)

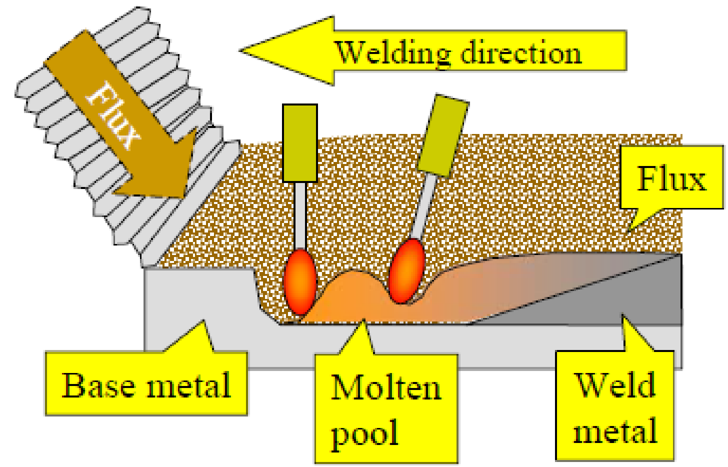
Only S355

Weight/m^2= -
+ 896 kg/m^2 (Grade S355)
+ 22 kg/m^2 (Y1860S7 ϕ15mm)

Submerged Arc Welding (SAW)

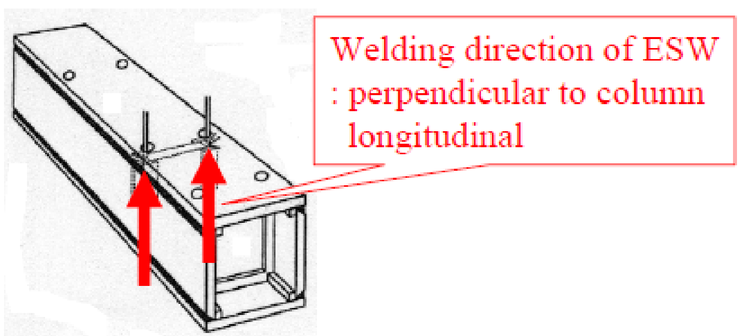


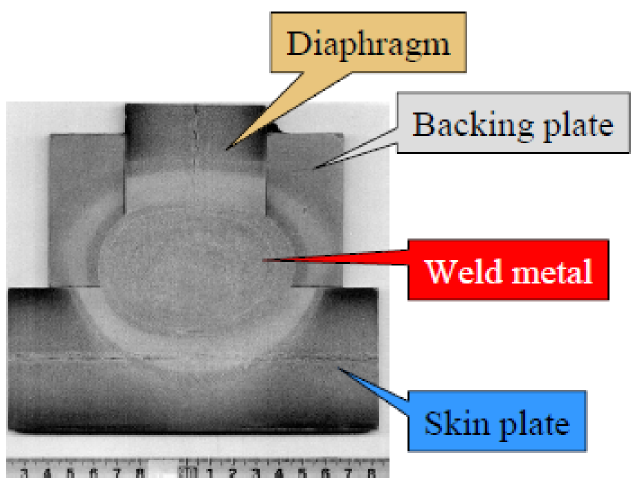


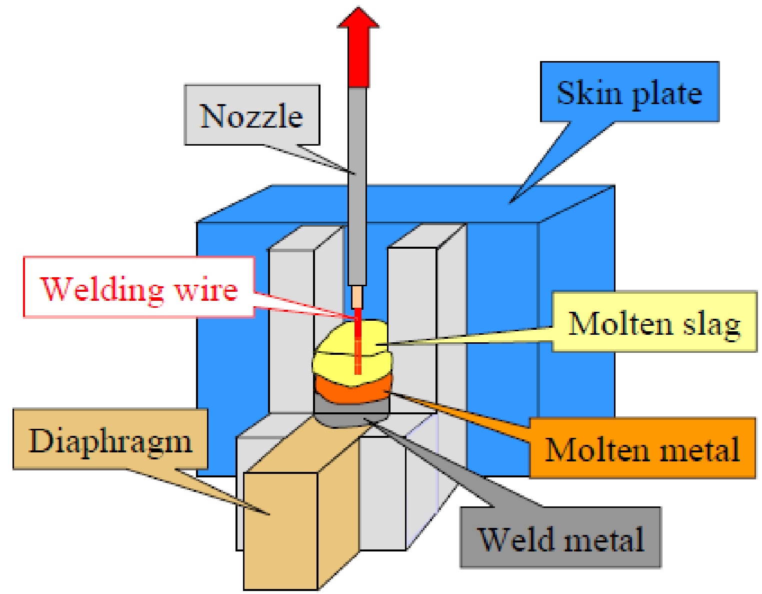


- Feed the welding wire into the flux supplied ahead of welding wires.
- Large current, multiple electrodes > High work efficiency

Electro-Slag Welding (ESW)





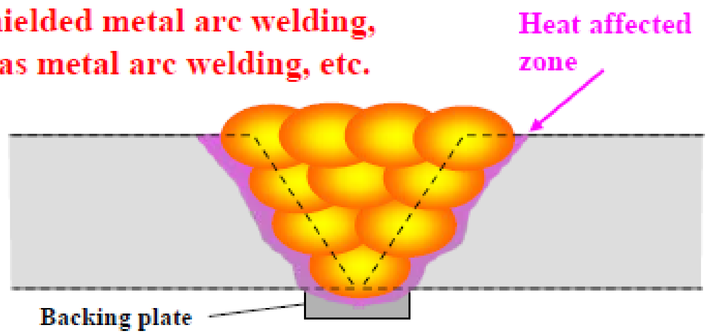


- Feed the welding wire into molten slag
  > Welding wire and base metal melt
  due to resistance heating of slag
- Upward welding

Heat Input & Size of HAZ – Productivity

Low heat input welding

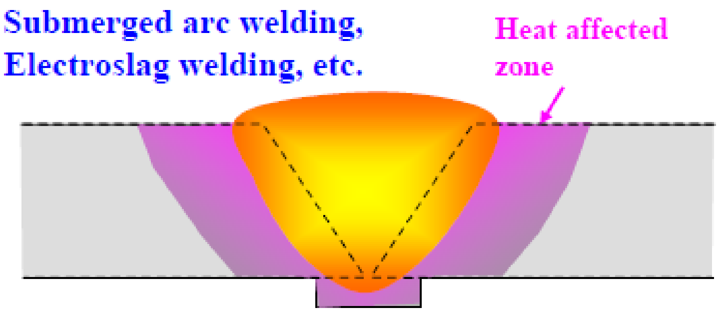
Shielded metal arc welding, Gas metal arc welding, etc.



- Weld heat input : 10～100kJ/cm
- Multi pass welding for heavy gauge plate
  > Low work efficiency
- Minor degradation of material properties in weld metal and heat affected zone

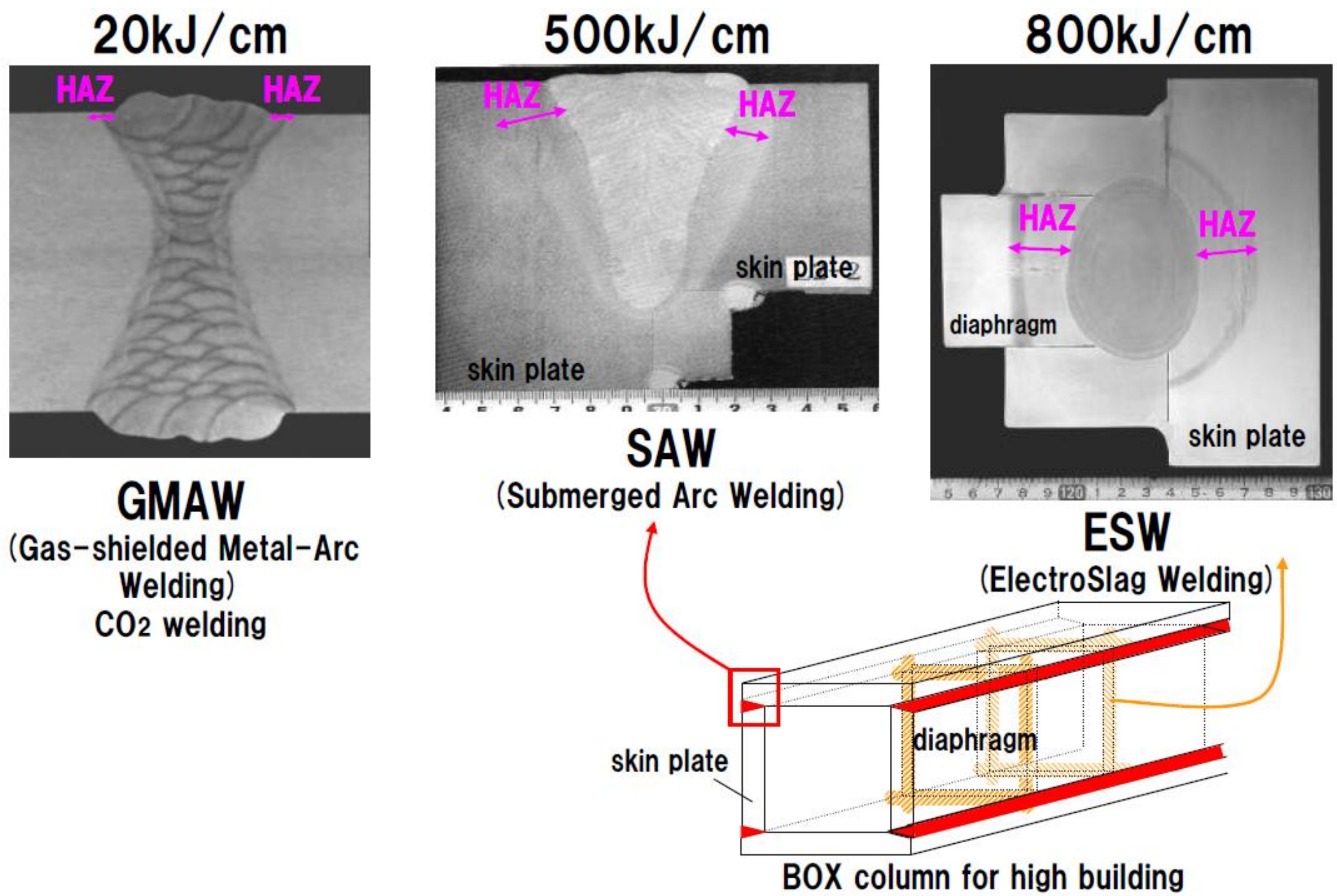
High heat input welding

Submerged arc welding, Electroslag welding, etc.



- Weld heat input : 150～1000kJ/cm
- Single pass welding for heavy gauge plate
  > High work efficiency
- Significant degradation of material properties in weld metal and heat affected zone

Heat Input for Various Welding Process



Possible Effects of Welding

1. HAZ: Mechanical property alteration including strength, toughness and ductility

2. High residual stress caused by thermal expansion & contraction and phase transformation

3. Distortion

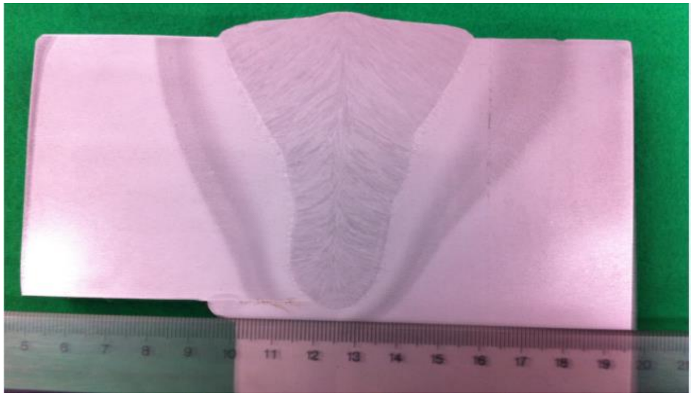
Influences of residual stress

Compression: Failures due to instability or buckling

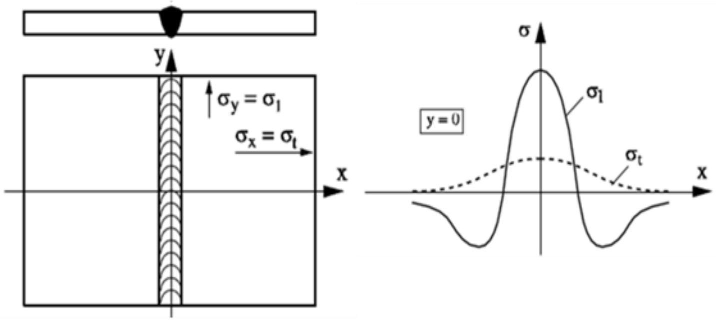
Tension: Premature failures and fatigue

"The effect on the performance of welded structure is significant only on phenomena that occur under low applied stress. The phenomena include Brittle fracture, fatigue and stress corrosion cracking."

– AWS Welding Handbook



HAZ by high heat input SAW



Residual stress distribution in butt welding

EN1993-1-12: Extension to S690

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ADDITIONAL RULES TO EN 1993-1-8

- Angles connected by one leg and other unsymmetrically connected members in tension: *Eccentricity must be considered*
- Rules for Lug angles are not applicable
- Rules for Resistance of welded connections for under matched welding electrodes
- Rigid-plastic and elastic-plastic global analyses are not applicable
- Rules for semi-rigid joints are not applicable
- Rules for design resistance of basic components – valid only for bolt failure mode: *concrete in compression, base plate in bending under compression and tension, anchor bolt in tension*
- Rules for hollow section: static resistance should be reduced by a factor of 0.8 (0.9 for S355-S460)

ADDITIONAL RULES TO EN 1993-1-10: Rules for maximum permissible element thickness

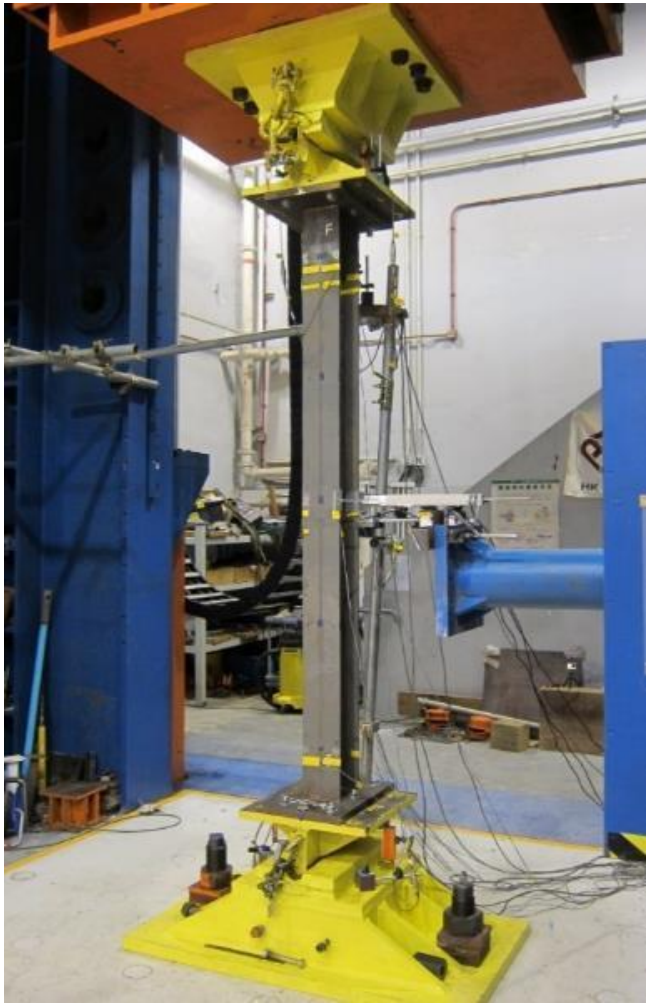
Reduction Factor of 0.8 for S690

Compression Tests on S690 Columns

Concentrically loaded columns: Back analysis to EN 1993-1

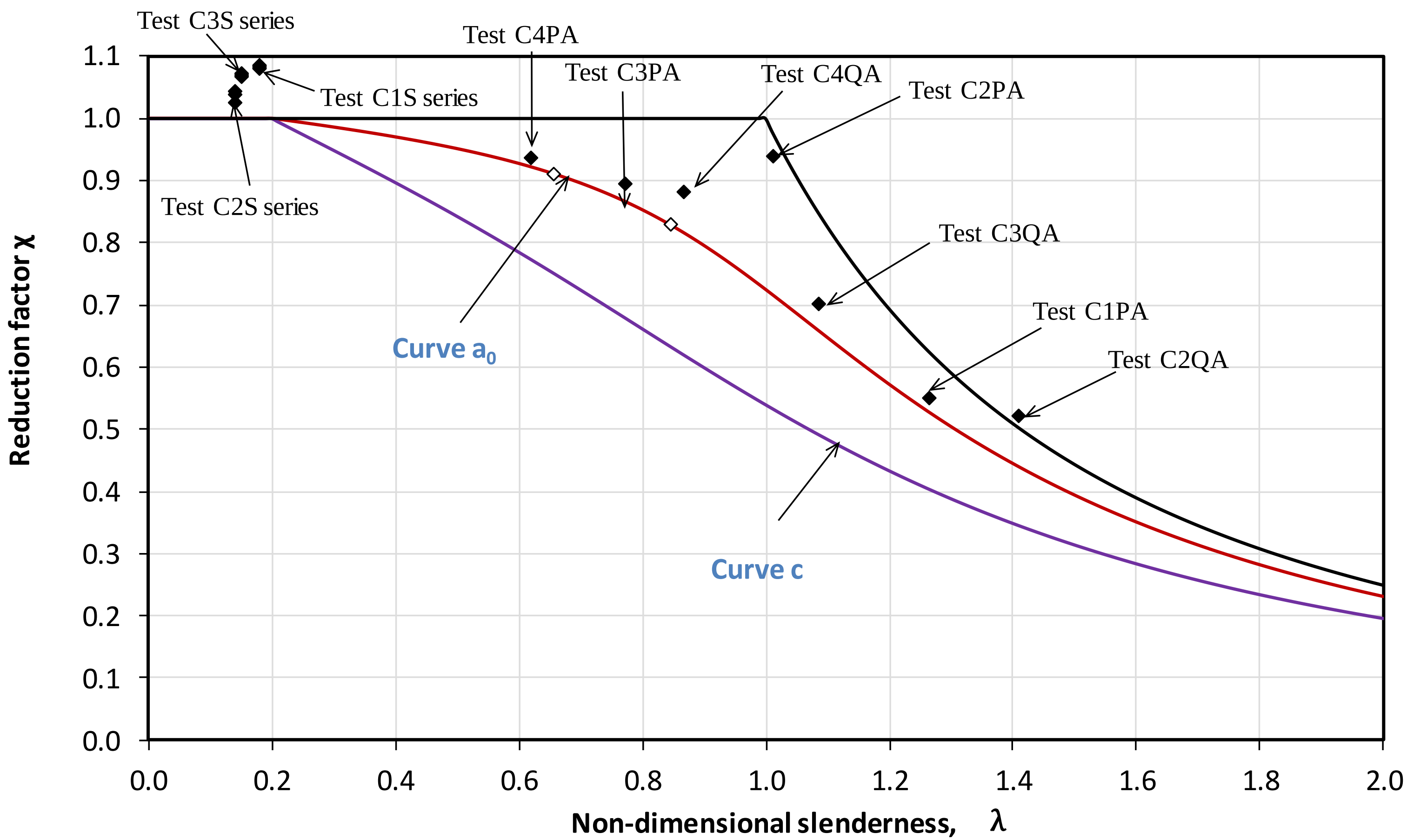
Loading procedure:
Pre-loading: up to 10% predicted resistance

Loading application:
1. Initially, use load control:
   ≈ **10 N/mm² per minute**
2. When approaching 80% of the predicted resistance, use displacement control:
≈ **0.2 mm per minute**



Tests done @HKPolyU

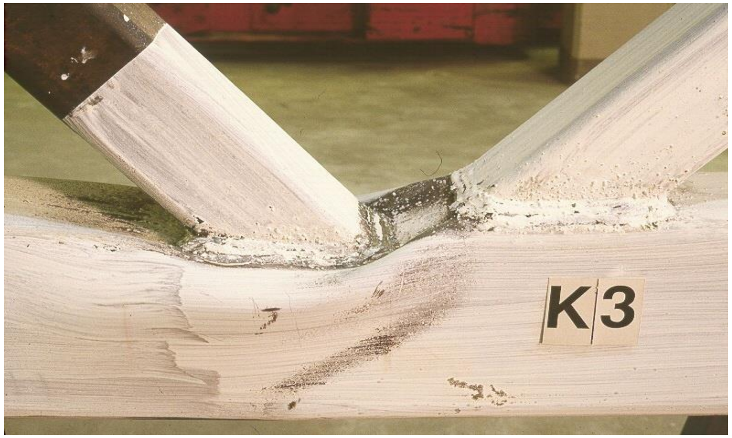
Column Test Results



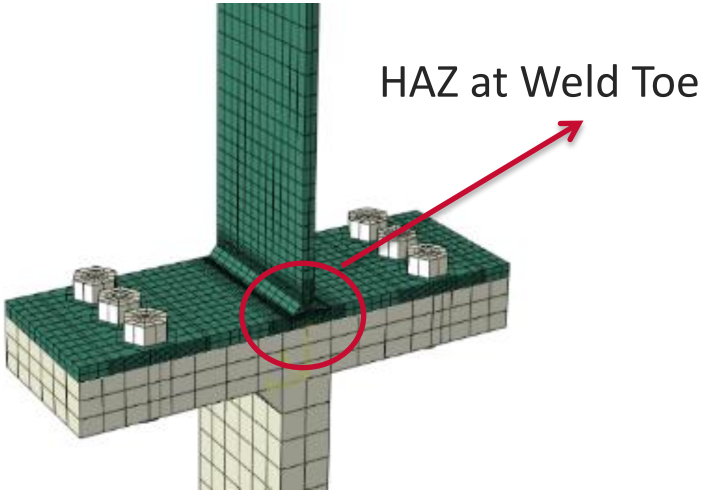
- ✓ **Current design rules for cross-section resistances are considered to be applicable to S690 steel columns.**
- ✓ **Section classification for determination of cross-section resistances may be enhanced.**

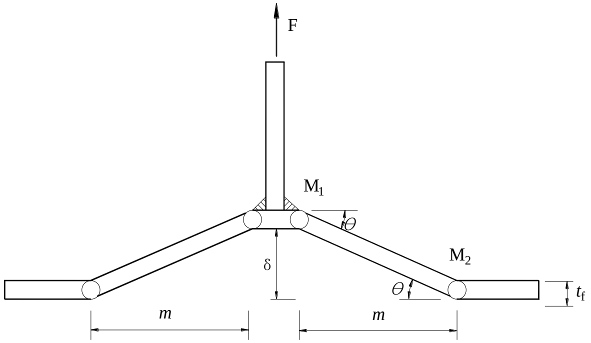
Equivalent T-Stub Joint

Why model and test simple T-Stub Joint?



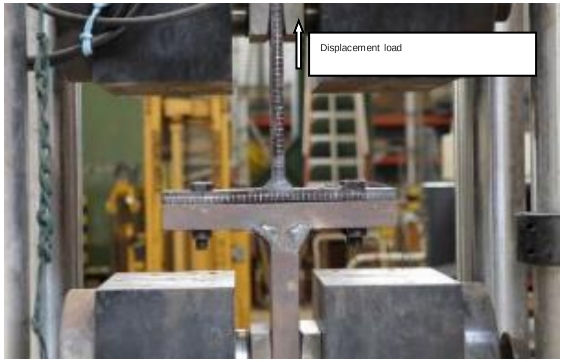
Plastification failure – Formation of Plastic Hinge at Weld Toe

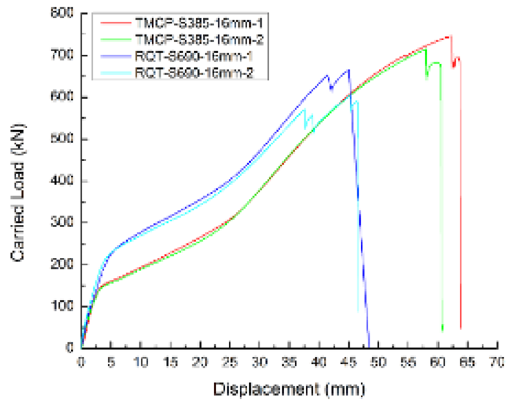




Deformation of the T-stub joint (Mode 1)

Effect of HAZ



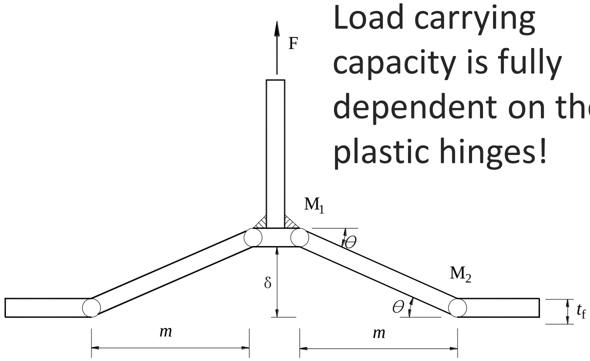


EN1993-1-8
Design plastic resistance
(*Component/Yield line Method)*

$$F = \frac{4M_{pl,1,Rd}}{m} \qquad \text{Eqn. (1)}$$

$$F = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)} \qquad \text{Eqn. (2)}$$

Note: Eqn. (2) takes the effect of washer and bolt heat into consideration



Load carrying capacity is fully dependent on the plastic hinges!

Deformation of the T-stub joint (Mode 1)

$$M_{pl,1,Rd} = l_{eff}\,(\frac{t}{2})^2 f_y$$

Design moment resistance of the section

Thickness (mm)		Test Results (kN)			EC3 Eqns. (kN)		Diff 3 (%)	Diff 4 (%)
		Test1	Test2	Test Average	Eqn. 1	Eqn. 2		
RQT-S690	16	217.8	219.5	218.7	210.3	225.6	4.0	-3.1
TMCP-S385	16	146.5	142.0	144.3	117.3	125.9	23.0	14.6

Safe for NSS but not safe for HSS!

In EC3, three failure modes, namely (1) complete yielding of the flange, (2) bolt failure with yielding of the flange and (3) bolt failure are identified. - Mode 2 and 3 depend on the strength of the bolts, while mode 1 depends on the strength of the steel.

Reduction Factor for Plastic Resistance

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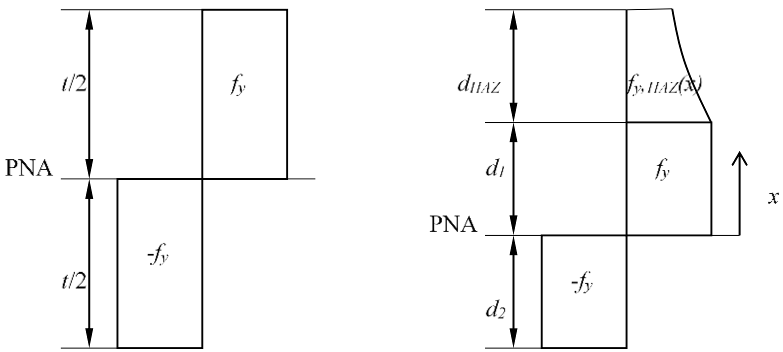


(a) Without HAZ

(b) With HAZ
stress distribution with $f_{y,HAZ}$ varies with x

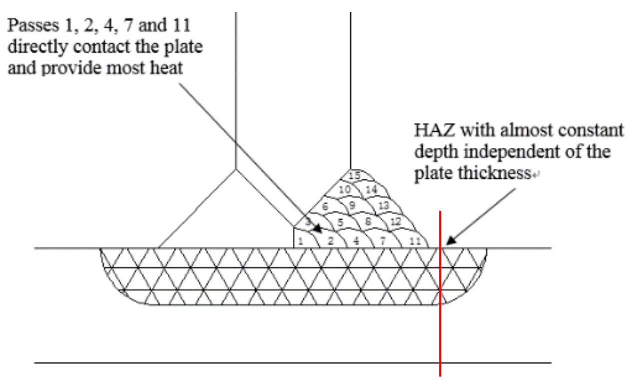
Modified first yield resistance

$$F = \frac{2M^{wt}_{pl,1,Rd} + 2M^{bh}_{pl,1,Rd}}{m} = \frac{4(\chi_{HAZ})M_{pl,1,Rd}}{m} \quad \text{Eqn. (3)}$$

$$F = \frac{(8n - 2e_w)(2M^{wt}_{pl,1,Rd} + 2M^{bh}_{pl,1,Rd})/4}{2mn - e_w(m+n)} = \frac{(8n - 2e_w)(\chi_{HAZ})M_{pl,1,Rd}}{2mn - e_w(m+n)} \quad \text{Eqn. (4)}$$

$$\chi_{HAZ} = \frac{2M^{wt}_{pl,1,Rd} + 2M^{bh}_{pl,1,Rd}}{4M_{pl,1,Rd}} = \frac{M^{wt}_{pl,1,Rd}}{2M_{pl,1,Rd}} + 0.5$$

χ_{HAZ} is in fact a moment resistance ratio: weld toe over the normal cross section



Cross-section at the weld toe

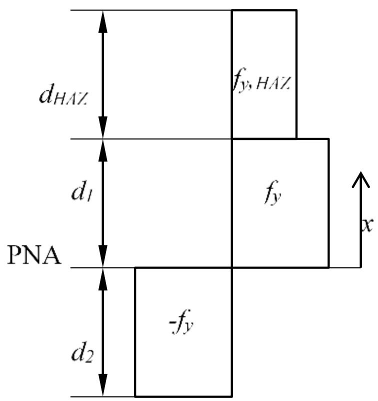
Simply multiply χ_{HAZ} to the EC3 equations!

χ_{HAZ}

$$F = \frac{4M_{pl,1,Rd}}{m} \quad \text{Eqn. (1)}$$

$$F = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)} \quad \text{Eqn. (2)}$$

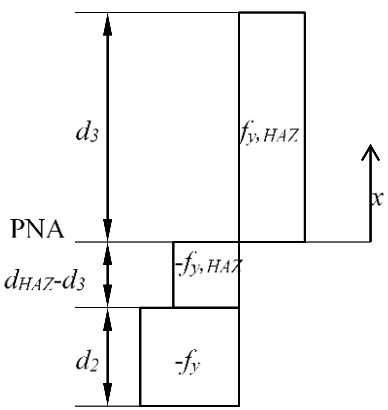
How to obtain Reduction Factor χ_{HAZ}



Case 1: d_{HAZ} is smaller than the depth of the PNA (usual case)

$$\begin{cases} d_{HAZ} + d_1 + d_2 = t \\ \int_{-t/2}^{t/2} \sigma dy = f_{y,HAZ} d_{HAZ} + f_y d_1 - f_y d_2 = 0 \\ M_{pl,1,Rd}^{wt} = \int_{-t/2}^{t/2} \sigma y dy = l_{eff} [\frac{f_y (d_2^2 + d_1^2)}{2} + f_{y,HAZ} d_{HAZ} (\frac{d_{HAZ}}{2} + d_1)] \end{cases}$$

$$\Rightarrow \chi_{HAZ} = \frac{\rho_{HAZ}^2}{2}(1 - \varepsilon_{HAZ}^2) - \rho_{HAZ}(1 - \varepsilon_{HAZ}) + 1 \qquad \text{Eqn. (5)}$$



Case 2: d_{HAZ} is larger than the depth of the PNA (extreme case)

$$\begin{cases} d_{HAZ} + d_2 = t \\ \int_{-t/2}^{t/2} \sigma dy = f_{y,HAZ} d_3 - f_{y,HAZ}(d_{HAZ} - d_3) - f_y d_2 = 0 \\ M_{pl,1,Rd}^{wt} = \int_{-t/2}^{t/2} \sigma y dy = l_{eff} [f_y d_2 (d_{HAZ} - d_3 + \frac{d_2}{2}) + f_{y,HAZ} \frac{(d_{HAZ} - d_3)^2}{2} + f_y \frac{d_3^2}{2}] \end{cases}$$

$$\Rightarrow \chi_{HAZ} = \frac{\varepsilon_{HAZ} \rho_{HAZ}^2}{2} - \frac{(1 - \rho_{HAZ})^2}{2\varepsilon_{HAZ}} - \rho_{HAZ} + 1.5 \qquad \text{Eqn. (6)}$$

Reduction factor is dependent on relative yield strength and relative depth of the assumed HAZ only!

$$\begin{cases} \varepsilon_{HAZ} = f_{y,HAZ}/f_y \\ \rho_{HAZ} = d_{HAZ}/t \end{cases}$$

In particular, if $\rho_{HAZ} = 1$, that is the whole weld toe thickness is fully affected by welding,

$$\Rightarrow \chi_{HAZ} = \frac{(\varepsilon_{HAZ} + 1)}{2}$$

Results and Parametric Study

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Test average (kN)	χ_{HAZ}	Eqn. 3 (kN)	Eqn. 4 (kN)	Diff 3 (%)	Diff 4 (%)
218.7	0.957	201.3	215.9	8.6	1.3
	0.842	177.2	190.0	23.4	15.1

Note: Better results can be obtained if the evaluation of the mechanical properties of HAZ is more precise

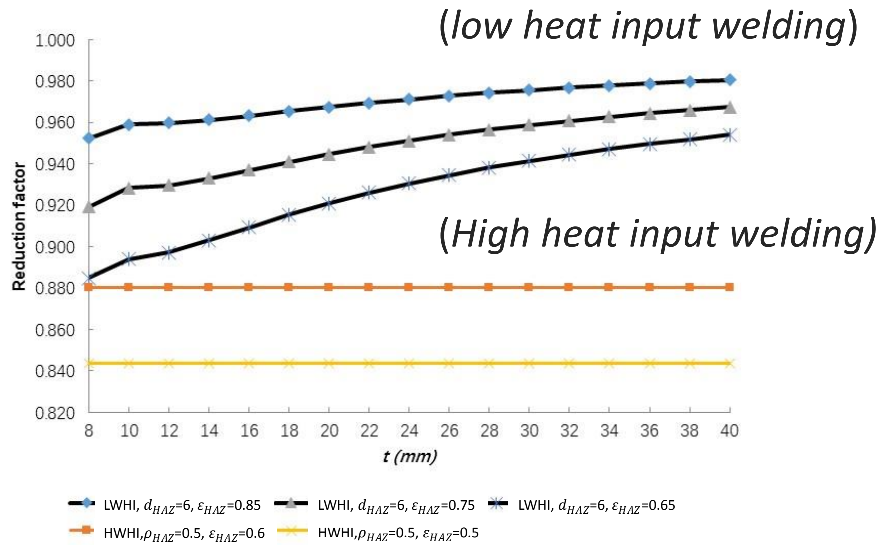
LOW HEAT INPUT WELDING (e.g. SMAW, FCAW)

- d_{HAZ} will almost be constant and independent of the thickness
- strength ratio ε_{HAZ} will be relatively large (e.g. 0.65 to 0.85 for RQT-S690)

HIGH HEAT INPUT WELDING (e.g. ESW, SAW)

- ρ_{HAZ} will be a constant with respect to the thickness
- ε_{HAZ} will be relatively low (e.g. 0.5 to 0.6 for RQT-S690)

Parametric Study



Reference: M.S. Zhao, C.K. Lee, T.C Fung and S.P. Chiew, Impact of welding on the strength of high performance steel T-stub joints, to be published.

To be precise, eqns. 5 and 6 should be used to calculate χ_{HAZ}.
For design simplicity, reduction factor χ_{HAZ} = 0.85 is recommended.

Conclusions

		EC2	EC3	EC4
Concrete	Normal	C12/15- C90/105	–	C20/25 - C60/75
	Light weight	LC12/13 – LC80/88		LC20/22 - LC60/66
Reinforcing steel		400 - 600 N/mm²	–	400 - 600 N/mm²
Structural steel		–	≤ 690 N/mm²	≤ 460 N/mm²

1. Work is in progress to extend the current limitation of high strength GR600 steel rebars in RC columns in EC2.
2. Work is in progress to extend the current limitation of high strength steel of S460 in SRC columns in EC4.
3. Finally, we should use more GR600 and S460 steel to improve productivity.

Thank you !