Towards The Design of Super Columns

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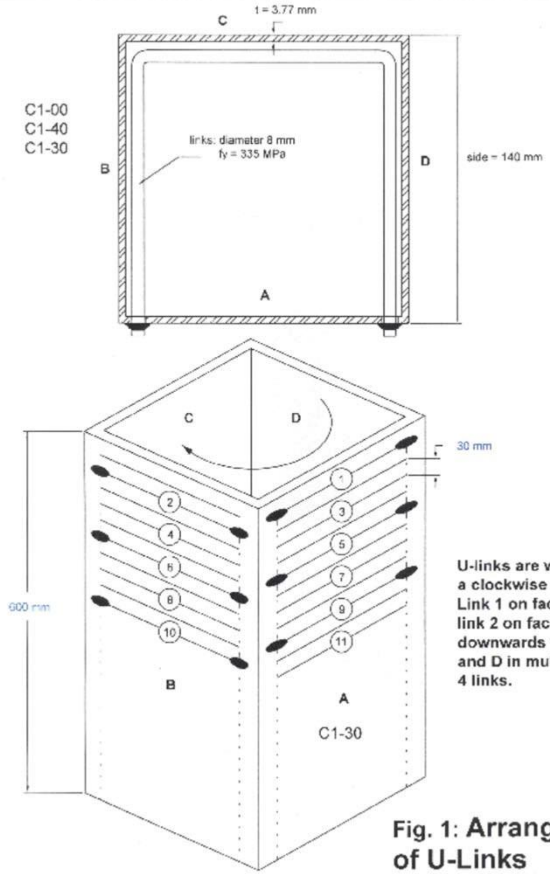
Description:

- Tubular Column – Square or Round
- Filled with Concrete
- Provided with U-Links welded to its Walls as shown in Figure 1

Compression Specimen

U-Links are used to **Confine** Concrete



U-links are welded in a clockwise manner. Link 1 on face A, then link 2 on face B, downwards to faces C and D in multiples of 4 links.

Fig. 1: **Arrangement of U-Links**

What is a Super Column?

- Large Forces
- Sustains Large Axial Strains
- Has A Unique Type of Failure

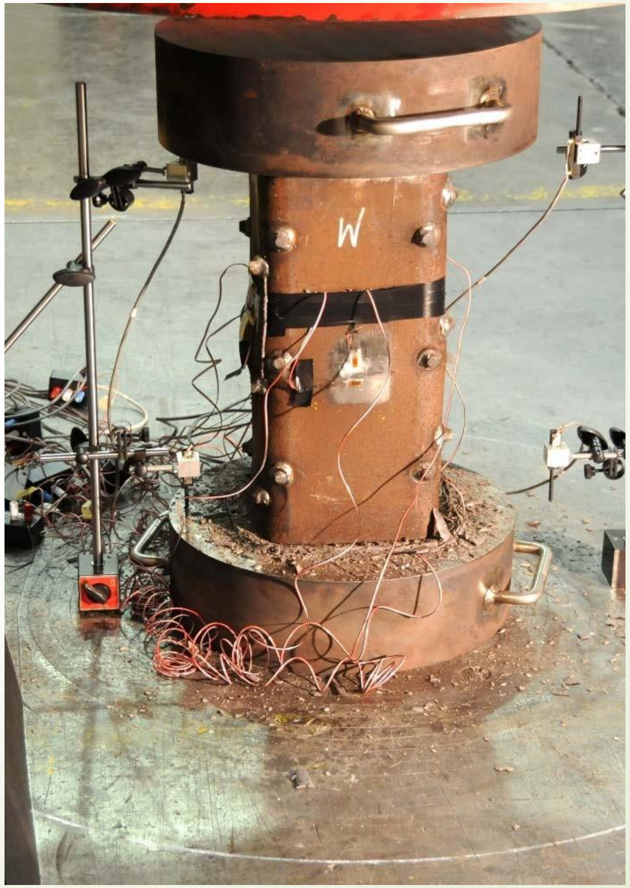
Super Column Failure:

- By Plastic Buckling of the steel Tube.
- Not by the Crushing of Concrete.
- Concrete does not fail
- In fact: Concrete deforms inside the buckled tube shape depicting its exact inside shape with no signs of cracking!!!

3 Million pounds Apparatus – Newmark Lab - University of Illinois at Urbana Champaign

***Author** among professors from Civil and Mechanical Engineering Departments.*

Test Specimen fitted with all sorts of measuring devices



Concrete reshaped – No signs of cracks

Summary of Test Results

	Square Specimens **Group-1**			**Circular Specimens** **Group-2**				**Circular Specimens** **Group-3**		**Rectangular Specimens** **group-4**		
Specimen	C1-000	C1-040	C1-030	cc-000	cc-040	cc-030	cc-020	C2C-000	C2C-020	REC-000	REC-045	REC-035
Spacing of U-Links (mm)	-	40	30	-	40	30	20	-	20	-	45	35
Ultimate Load (kN)	1104	1568	1620	1230	1693	1744	1947	2270	3290	1900	2280	2306
$f_{max.} / f_c'$	0.85	1.73	1.83	0.86	1.87	1.98	2.43	0.78	1.77	0.85	1.58	1.63
	f_c' = 30 MPa $A_c = 17540\ mm^2$ f_y (shell) =320 MPa Dia. of U-link = 8 mm f_y (link) = 335 MPa			f_c' = 23.7 MPa $A_c = 19265\ mm^2$ f_y (shell) = 355 MPa Dia. of U-link = 8 mm f_y (link) = 335 MPa				f_c' = 27.1 MPa $A_c = 37840\ mm^2$ f_y (shell) = 454 MPa Dia. of U-link = 10mm f_y (link) = 486 MPa		f_c' = 23.4 MPa $A_c = 22220\ mm^2$ f_y (shell) = 367 MPa Dia. of U-link = 10 mm f_y (link) = 486 MPa		

Summary of the results.

Test Results at Newmark Laboratories Control Specimen 1222 kips

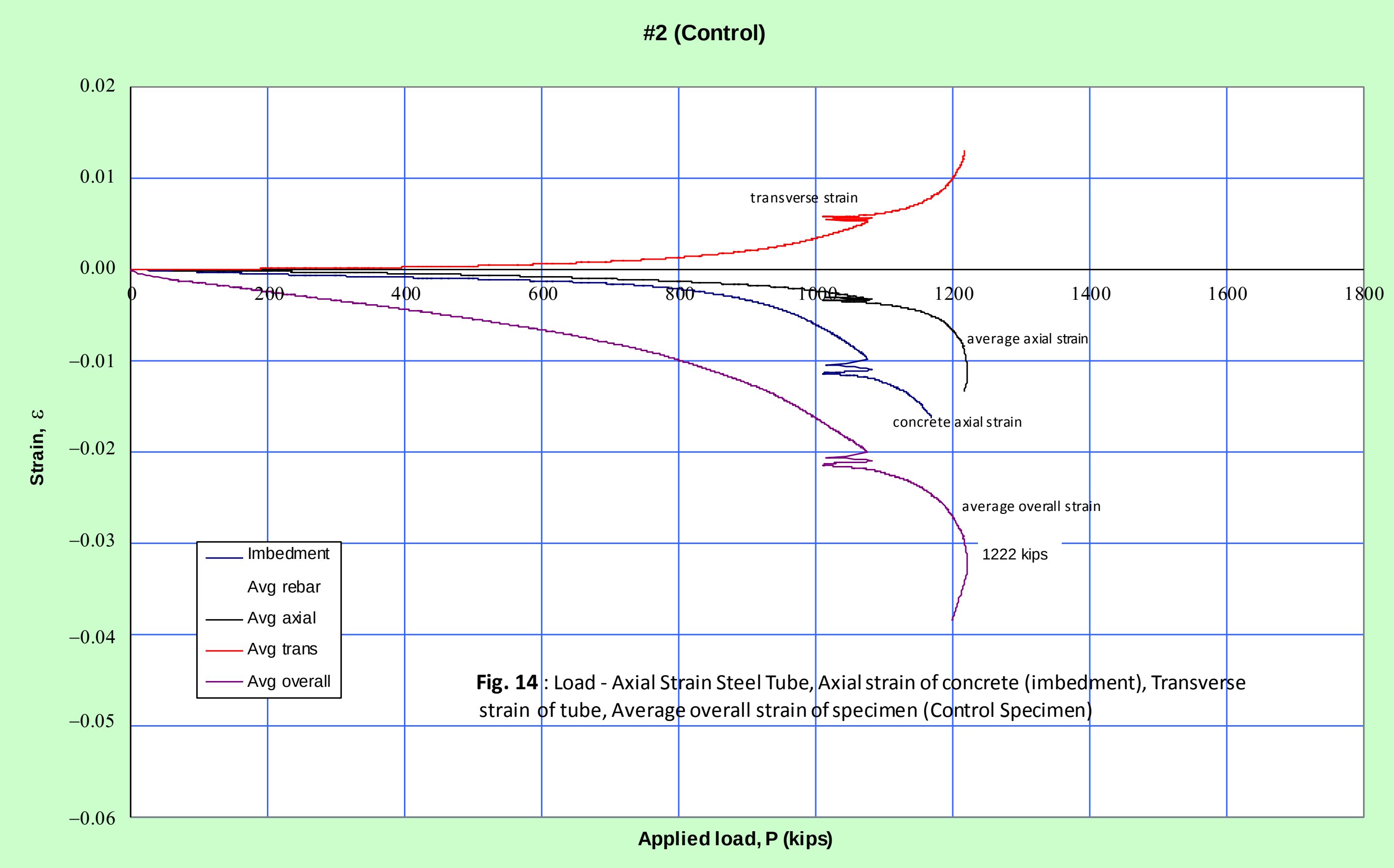


Fig. 14 : Load - Axial Strain Steel Tube, Axial strain of concrete (imbedment), Transverse strain of tube, Average overall strain of specimen (Control Specimen)

3/8” U-Links
250 kips ----5.1 ksi added =20%

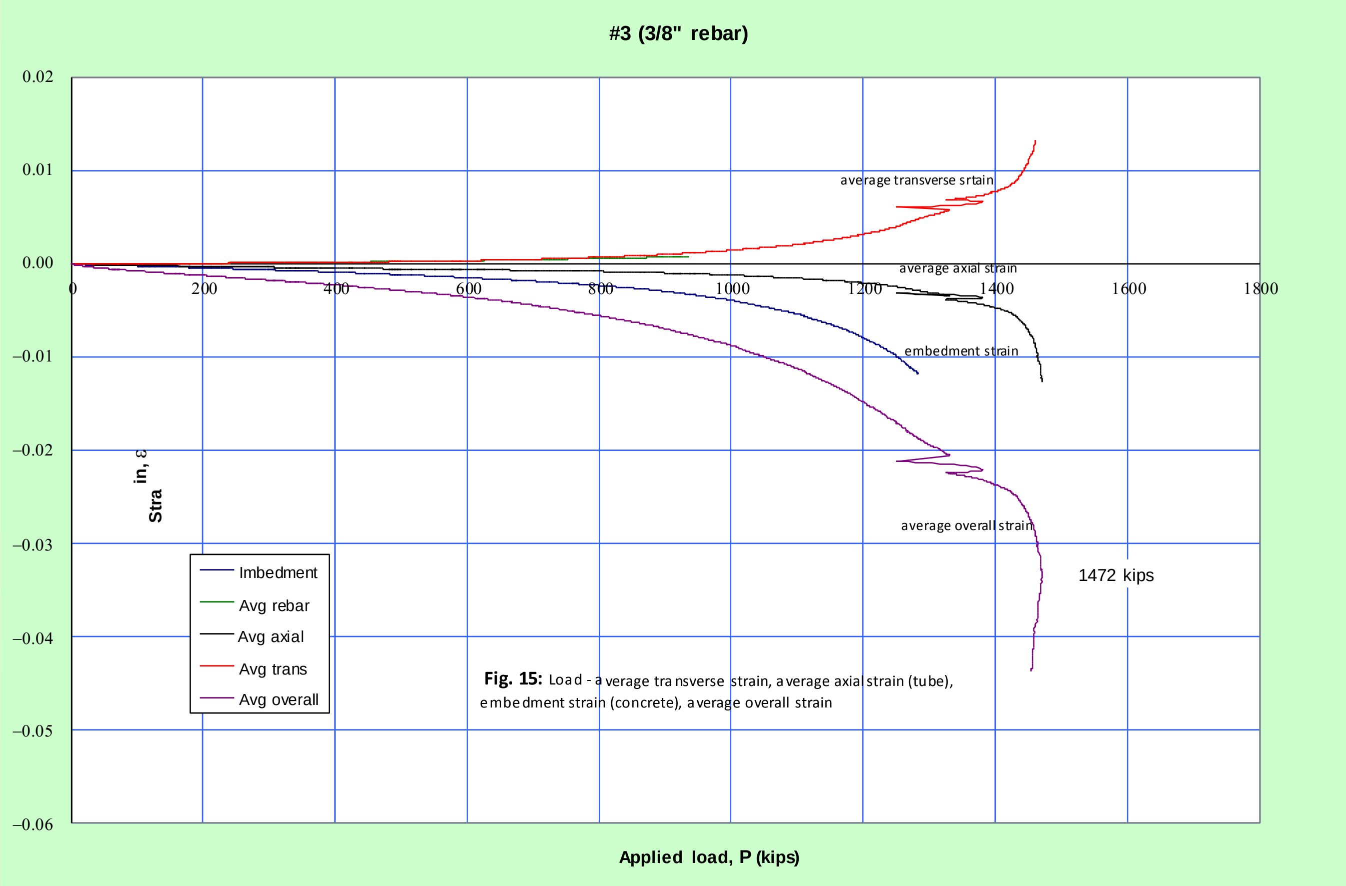
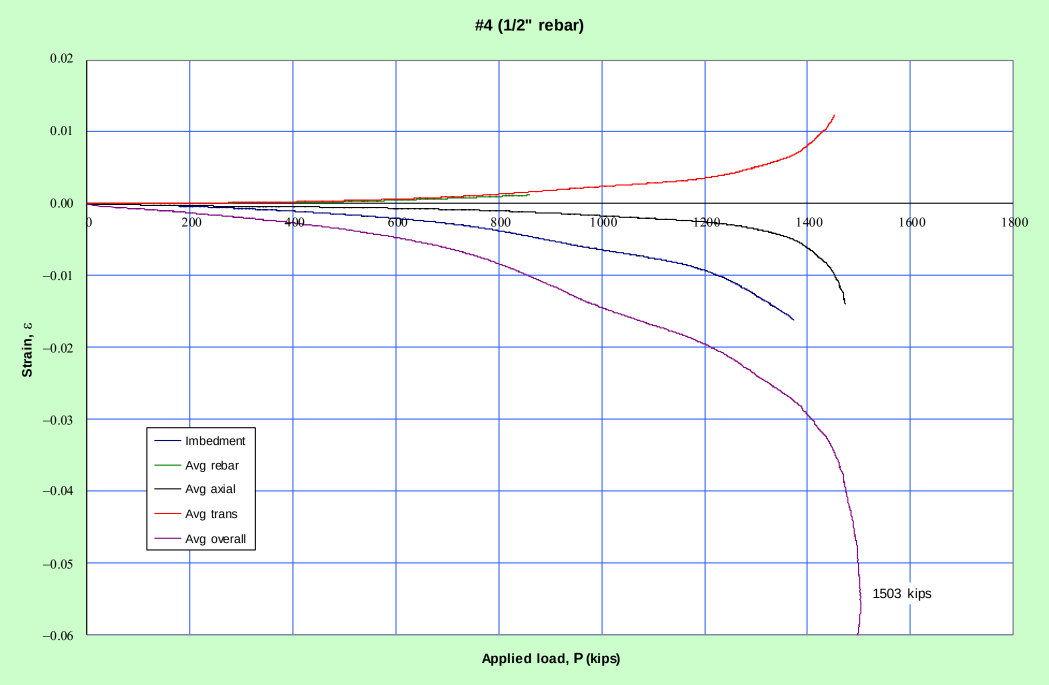
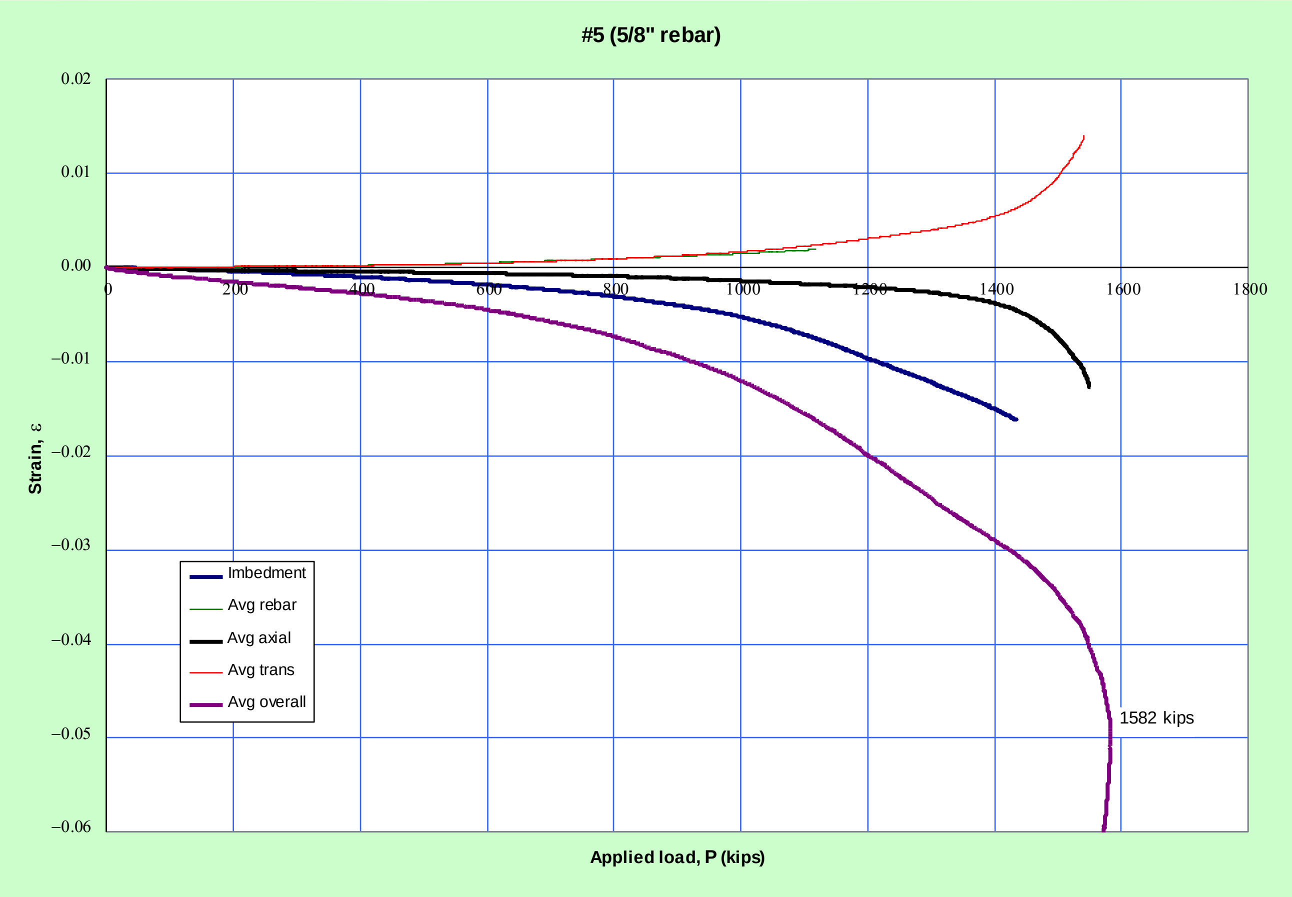


Fig. 15: Load - average transverse strain, average axial strain (tube), embedment strain (concrete), average overall strain

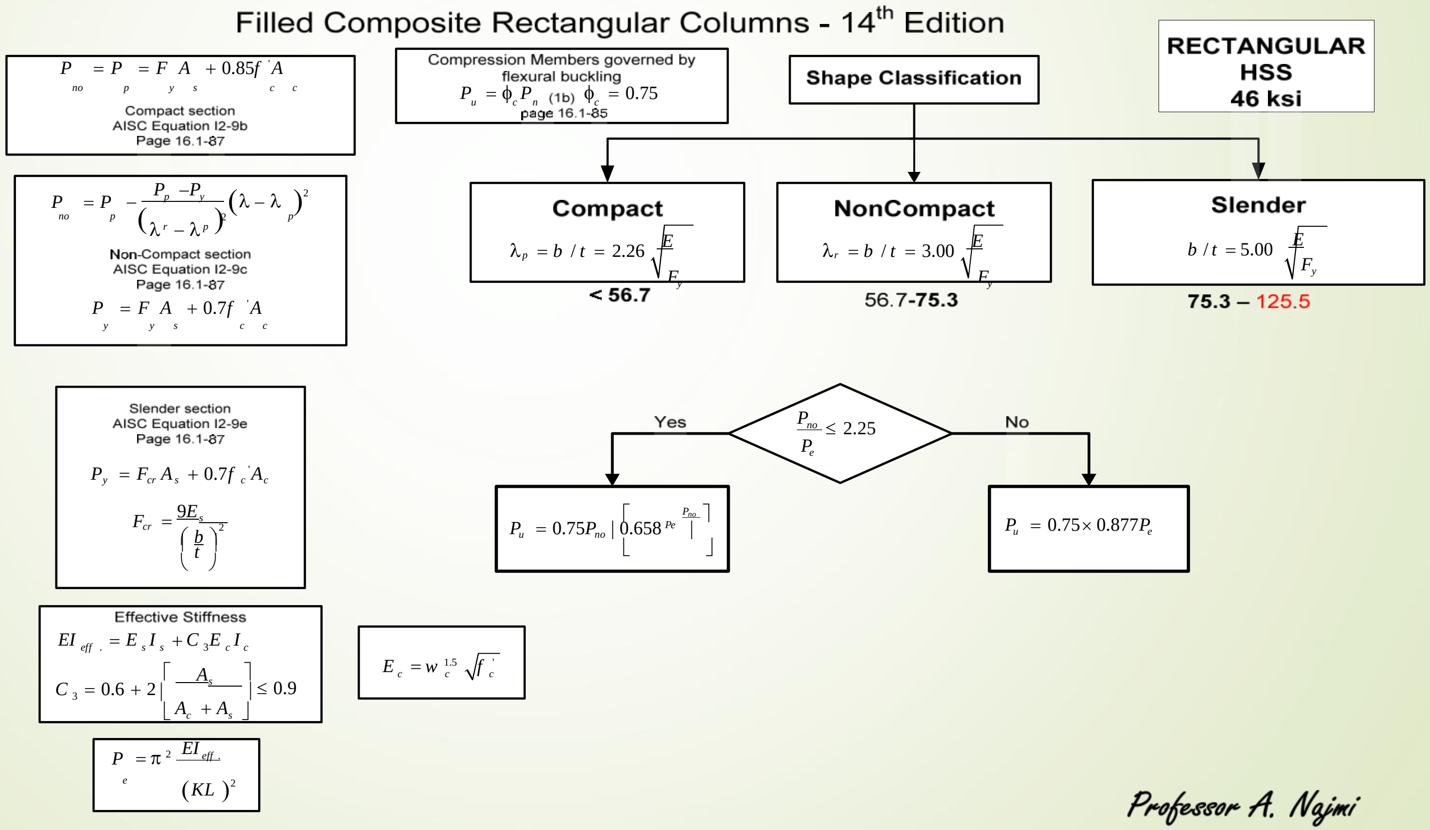
½ “ U-Links
281 kips ----5.7 ksi added =23%



5/8” U-Links
360 kips ---- 7.3 ksi added =30%



Filled Composite Rectangular Columns AISC Specifications



Filled Composite Rectangular Columns - 14th Edition

RECTANGULAR
HSS
46 ksi

Compression Members governed by flexural buckling
$P_u = \phi_c P_n$ (1b) $\phi_c = 0.75$
page 16.1-85

Shape Classification

Compact
$\lambda_p = b/t = 2.26\sqrt{\frac{E}{F_y}}$
< 56.7

NonCompact
$\lambda_r = b/t = 3.00\sqrt{\frac{E}{F_y}}$
56.7-**75.3**

Slender
$b/t = 5.00\sqrt{\frac{E}{F_y}}$
75.3 – 125.5

$$P_{no} = P_p = F_y A_s + 0.85 f'_c A_c$$

Compact section
AISC Equation I2-9b
Page 16.1-87

$$P_{no} = P_p - \frac{P_p - P_y}{(\lambda_r - \lambda_p)^2}(\lambda - \lambda_p)^2$$

Non-Compact section
AISC Equation I2-9c
Page 16.1-87

$$P_y = F_y A_s + 0.7 f'_c A_c$$

Slender section
AISC Equation I2-9e
Page 16.1-87

$$P_y = F_{cr} A_s + 0.7 f'_c A_c$$

$$F_{cr} = \frac{9E_s}{\left(\frac{b}{t}\right)^2}$$

Effective Stiffness

$$EI_{eff.} = E_s I_s + C_3 E_c I_c$$

$$C_3 = 0.6 + 2\left[\frac{A_s}{A_c + A_s}\right] \le 0.9$$

$$P_e = \pi^2 \frac{EI_{eff.}}{(KL)^2}$$

$$E_c = w_c^{1.5}\sqrt{f'_c}$$

$\frac{P_{no}}{P_e} \le 2.25$

Yes

$$P_u = 0.75 P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right]$$

No

$$P_u = 0.75 \times 0.877 P_e$$

Professor A. Najmi

Where:

$$P_u = 0.75P_{no}\left[0.658^{\frac{P_{no}}{Pe}}\right] \qquad (1)$$

$$P_{no} = P_p = F_y A_s + 0.85 f_c' A_c \qquad (2)$$

$$P_e = \pi^2 \frac{EI_{eff.}}{(KL)^2} \qquad (3)$$

$$EI_{eff.} = E_s I_s + C_3 E_c I_c$$

$$C_3 = 0.6 + 2\left[\frac{A_s}{A_c + A_s}\right] \le 0.9$$

P_{no} = nominal strength

P_e = elastic buckling load

A_c = area of concrete

E_c = modulus of elasticity of concrete

EI_{eff} = effective stiffness of composite section

E_s = modulus of elasticity of steel

F_y = minimum yield sress of steel

I_s = moment of inertia of steel shape

K = effective length factor

L = laterally unbraced length

f_c' = concrete grade

w_c = weight of concrete per unit volume

C_3 = coefficient related to filled composite compression member

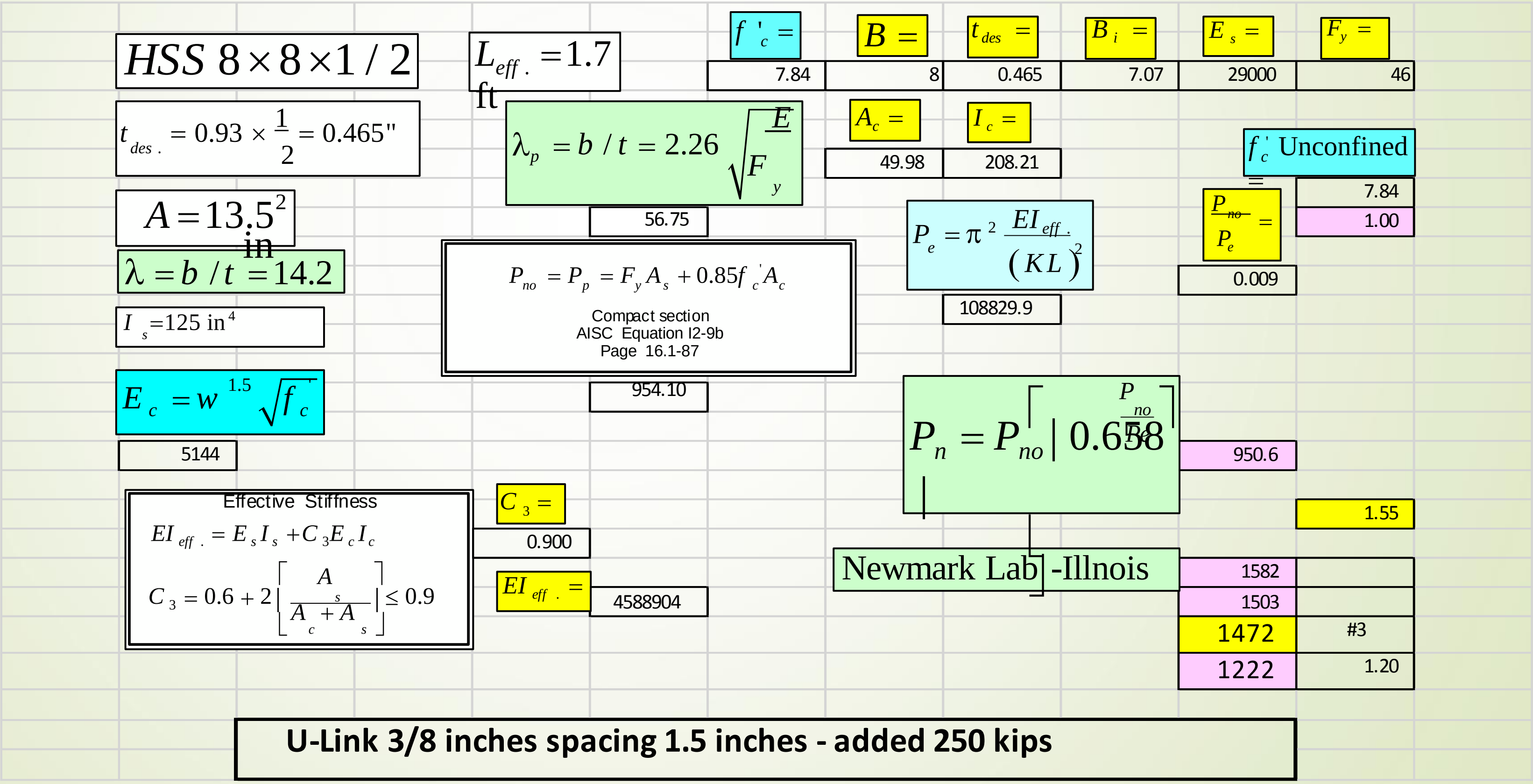
P_p = P_{no} for compact sections

A_s = area of steel shape

ϕ_c = 0.75

AISC Code

Calculations – AISC Eqn. 12-9b



$HSS\ 8 \times 8 \times 1/2$

$L_{eff.} = 1.7$ ft

$t_{des.} = 0.93 \times \frac{1}{2} = 0.465"$

$A = 13.5$ in^2

$\lambda = b/t = 14.2$

$I_s = 125$ in^4

$E_c = w^{1.5}\sqrt{f'_c}$

5144

$\lambda_p = b/t = 2.26\sqrt{\frac{E}{F_y}}$

56.75

$P_{no} = P_p = F_y A_s + 0.85 f'_c A_c$

Compact section
AISC Equation I2-9b
Page 16.1-87

954.10

Effective Stiffness

$EI_{eff.} = E_s I_s + C_3 E_c I_c$

$C_3 = 0.6 + 2\left[\frac{A_s}{A_c + A_s}\right] \leq 0.9$

$C_3 =$ 0.900

$EI_{eff.} =$ 4588904

$f'_c =$	$B =$	$t_{des} =$	$B_i =$	$E_s =$	$F_y =$
7.84	8	0.465	7.07	29000	46

$A_c =$ 49.98

$I_c =$ 208.21

$P_e = \pi^2 \frac{EI_{eff.}}{(KL)^2}$

108829.9

f'_c Unconfined = 7.84

1.00

$\frac{P_{no}}{P_e} =$ 0.009

$P_n = P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right]$

950.6

1.55

Newmark Lab -Illnois

1582	
1503	
1472	#3
1222	1.20

U-Link 3/8 inches spacing 1.5 inches - added 250 kips

$\varepsilon_y = \dfrac{F_y}{E_{s1}}$ □ $E_{s1} =$ 1st secant modulus of steel (E_{steel})

$\lambda = \dfrac{f_c''}{f_c'} \Rightarrow f_c'' =$ enhanced stress attained of concrete

$n = \dfrac{E_{s1}}{E_{s2}} \Rightarrow E_{s2} =$ 2nd secant modulus of steel at $n\varepsilon_y$

$E_{c2} = \dfrac{\lambda f_c'}{n} \Rightarrow$ 2nd secant modulus of concrete at $n\varepsilon_y$

$E_{s2} = \dfrac{E_{s1}}{n}, \quad N = \dfrac{E_{s1}}{E_{c2}},$

$A_{cT} = \dfrac{A_c}{N}$ ... (steel units), $A_{s2} = \dfrac{A_{s1}}{n}$ .... (steel units),

$A_T = A_{cT} + A_{s2}$ (steel units)

$I_{s2} = \dfrac{I_s}{n}$ ....(steel units), $I_{cT} = \dfrac{(b-2t)}{N} \times \dfrac{(b-2t)^3}{12}$ ...(steel units)

$I_T = \dfrac{I_s}{n} + \dfrac{(b-2t)^4}{12N}$ ...(steel units)

$A_T = \dfrac{A_s}{n} + \dfrac{(b-2t)^2}{N}$ ....(steel units)

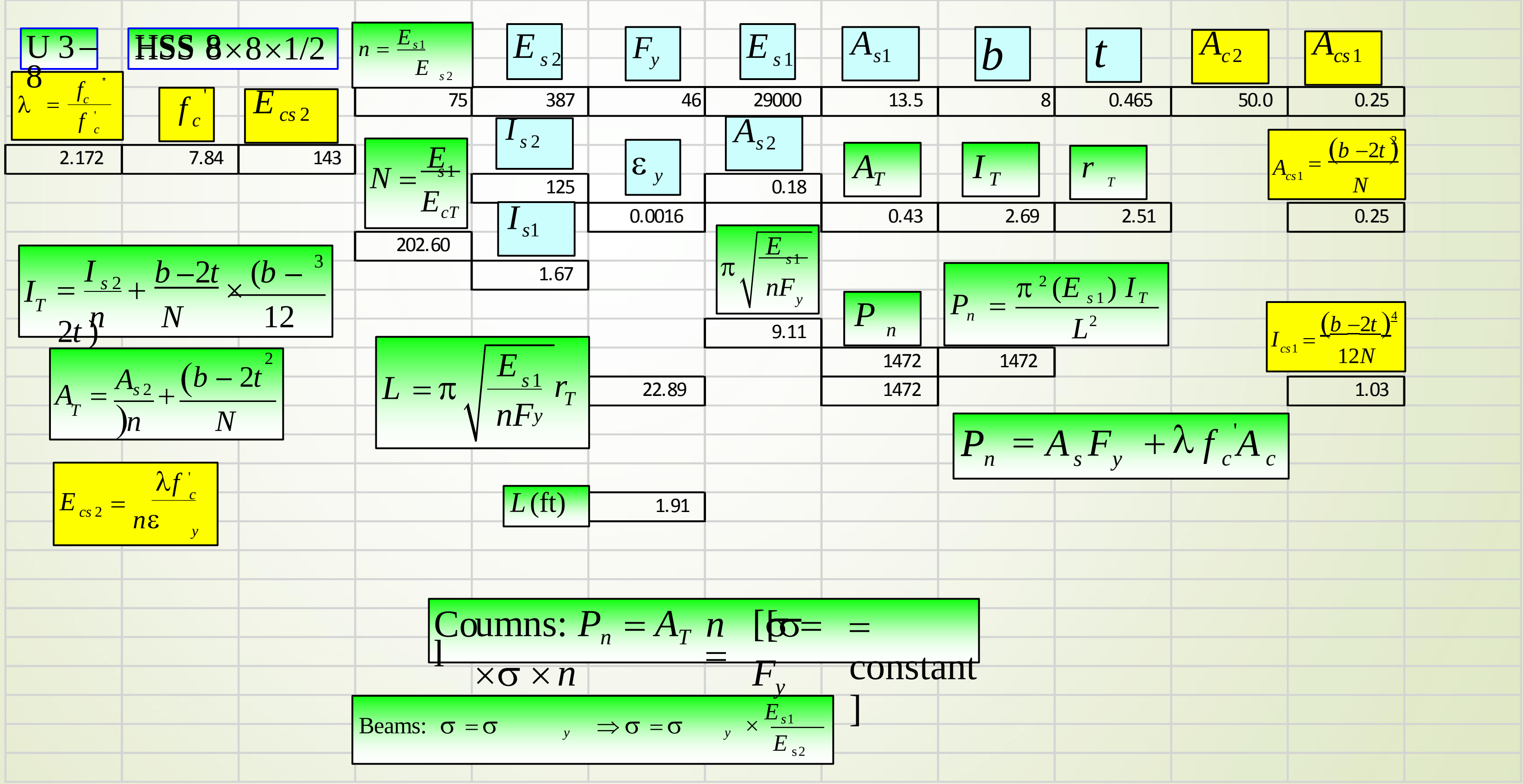
$r_T = \sqrt{\dfrac{I_T}{A_T}}$ ....(steel units)

$$\mathrm{P}_n \le \begin{cases} A_T F_y n \\ A_s F_y + \lambda f_c' A_c \\ \dfrac{\pi^2 E_{s2} I_T}{L^2} \\ \dfrac{\pi^2 E_s}{\left(\dfrac{L}{r_T}\right)^2} A_T \end{cases}$$

Non-Linear Transformation

Calculations

Applies to: Linear, plastic and Strain Hardening Stages



U 3–8

HSS 8×8×1/2

$\lambda = \frac{f_c''}{f_c'}$ = 2.172

f_c' = 7.84

E_{cs2} = 143

$n = \frac{E_{s1}}{E_{s2}}$ = 75

E_{s2} = 387

F_y = 46

E_{s1} = 29000

A_{s1} = 13.5

b = 8

t = 0.465

A_{c2} = 50.0

A_{cs1} = 0.25

$N = \frac{E_{s1}}{E_{cT}}$ = 202.60

I_{s2} = 125

I_{s1} = 1.67

ε_y = 0.0016

A_{s2} = 0.18

A_T = 0.43

I_T = 2.69

r_T = 2.51

$A_{cs1} = \frac{(b-2t)^2}{N}$ = 0.25

$\pi \sqrt{\frac{E_{s1}}{nF_y}}$ = 9.11

P_n = 1472

$P_n = \frac{\pi^2 (E_{s1}) I_T}{L^2}$ = 1472

1472

$I_{cs1} = \frac{(b-2t)^4}{12N}$ = 1.03

$$I_T = \frac{I_{s2}}{n} + \frac{b-2t}{N} \times \frac{(b-2t)^3}{12}$$

$$A_T = \frac{A_{s2}}{n} + \frac{(b-2t)^2}{N}$$

$$E_{cs2} = \frac{\lambda f_c'}{n\varepsilon_y}$$

$L = \pi \sqrt{\frac{E_{s1}}{nF_y}}\, r_T$ = 22.89

$L\,(\mathrm{ft})$ = 1.91

$$P_n = A_s F_y + \lambda f_c' A_c$$

Coumns: $P_n = A_T \times \sigma \times n$ [$\sigma = F_y$ = constant]

Beams: $\sigma = \sigma_y \Rightarrow \sigma = \sigma_y \times \frac{E_{s1}}{E_{s2}}$

Multi-Cell Column Cross-sections

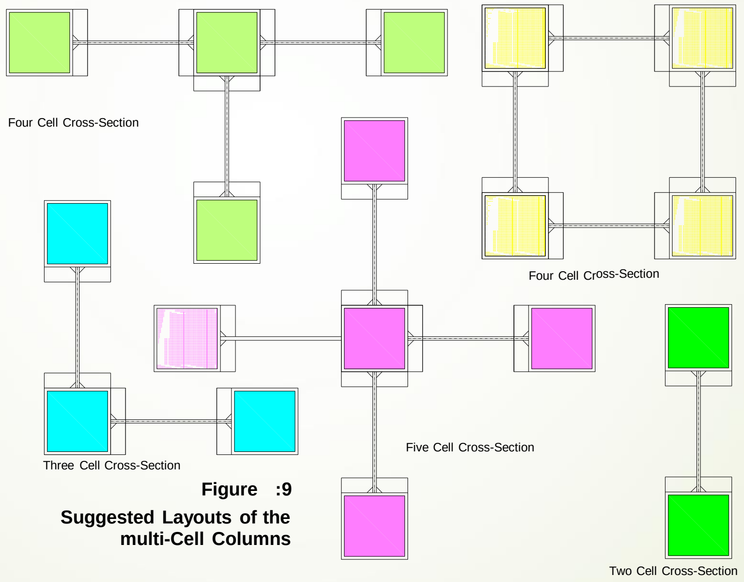


Figure :9
Suggested Layouts of the multi-Cell Columns

CONCLUSION

- The strength of a compression cell is linked to the U-Links provided, the use of square tubes ensures a uniform confinement of concrete.
- The ultimate strains attained in the compression cells together with the large inertia properties of multi-cell cross-sections results in high design moments close to plastic moments when considering large unsupported lengths.
- The failure of such columns will tend to be linked to plastic buckling of steel tubes rather than of crushing in concrete.