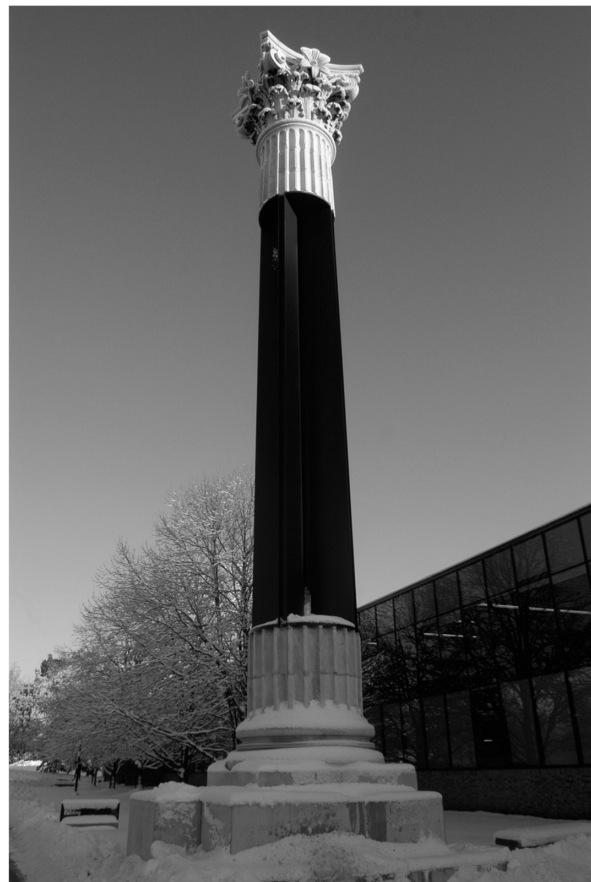


Steel Column Design Steel Connections

- Capacity Analysis of Steel Columns
- Design of Steel Columns
- Connection Types
- Connection Analysis



Design of Steel Columns with AISC Strength Tables

Data:

- Column – length
- Support conditions
- Material properties – F_y
- Applied design load - P_u

Required:

- Column Size

1. Enter table with height, $KL = L_c$
2. Read allowable load for each section to find the smallest adequate size.
3. **Tables assume weak axis buckling. If the strong axis controls the length must be divided by the ratio r_x/r_y**
4. Values stop in table (black line) at slenderness limit, $KL/r = 200$

4-24

DESIGN OF COMPRESSION MEMBERS

Table 4-1a (continued)													
Available Strength in													
Axial Compression, kips													
$F_y = 50 \text{ ksi}$													
W-Shapes													
Shape		W8x											
lb/ft													
Design		67		58		48		40		35		31	
		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_e (ft), with respect to least radius of gyration, r_y	0	590	886	512	769	422	634	350	526	308	463	273	411
	6	542	815	470	706	387	581	320	481	281	423	249	374
	7	526	790	455	685	375	563	309	465	272	409	241	362
	8	508	763	439	660	361	543	298	448	262	394	232	348
	9	488	733	422	634	347	521	285	429	251	377	222	333
	10	467	701	403	606	331	497	272	409	239	359	211	317
	11	444	668	384	576	314	473	258	388	226	340	200	301
	12	421	633	363	546	297	447	243	366	213	321	189	283
	13	397	597	342	514	280	421	228	343	200	301	177	266
	14	373	560	321	482	262	394	213	321	187	281	165	248
	15	348	523	299	450	244	367	198	298	174	261	153	230
	16	324	487	278	418	226	340	183	275	160	241	141	212
	17	300	450	257	386	209	314	169	253	147	221	130	195
	18	276	415	236	355	192	288	154	232	135	203	118	178
	19	253	381	216	325	175	264	141	211	123	184	108	162
	20	231	347	197	296	159	239	127	191	111	166	97.2	146
	22	191	287	163	244	132	198	105	158	91.5	138	80.3	121
	24	160	241	137	205	111	166	88.2	133	76.9	116	67.5	101
	26	137	205	116	175	94.2	142	75.2	113	65.5	98.5	57.5	86.5
	28	118	177	100	151	81.2	122	64.8	97.4	56.5	84.9	49.8	74.5
	30	103	154	87.5	131	70.7	106	56.5	84.9	49.2	74.0	43.2	64.9
	32	90.3	136	76.9	116	62.2	93.5	49.6	74.6	43.3	65.0	38.0	57.1
	34	79.9	120	68.1	102	55.1	82.8	44.0	66.1				
Properties													
P_{max} , kips		126	190	102	153	72.0	108	57.2	85.9	45.9	68.9	39.4	59.1
P_{max} , kip/in.		19.0	28.5	17.0	25.5	13.3	20.0	12.0	18.0	10.3	15.5	9.50	14.3
P_{max} , kips/in.		507	761	363	546	174	262	127	192	81.1	122	63.0	94.7
P_{max} , kips		164	246	123	185	87.8	132	58.7	88.2	45.9	68.9	35.4	53.2
L_c , ft		7.49		7.42		7.35		7.21		7.17		7.18	
L_c , ft		47.6		41.6		35.2		29.9		27.0		24.8	
A_g , in. ²		19.7		17.1		14.1		11.7		10.3		9.13	
I_x , in. ⁴		272		228		184		146		127		110	
I_y , in. ⁴		88.6		75.1		60.9		49.1		42.6		37.1	
r_x , in.		2.12		2.10		2.08		2.04		2.03		2.02	
r_y/r_x		1.75		1.74		1.74		1.73		1.78		1.72	
$r_{xx}/L_c^2 \times 10^4$, k-in. ²		7790		6530		5270		4180		3630		3150	
$r_{yy}/L_c^2 \times 10^4$, k-in. ²		2540		2150		1740		1410		1220		1060	
ASD		LRFD											
$\Omega_c = 1.67$		$\phi_c = 0.90$											
Note: Heavy line indicates L_c/r_y equal to or greater than 200.													

Design Example 1

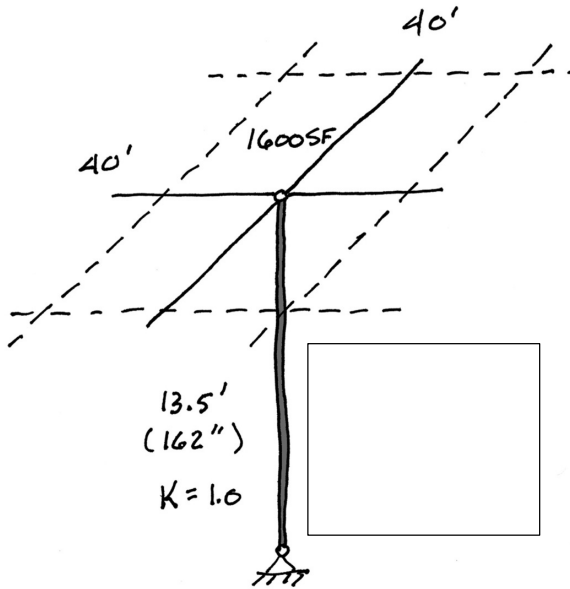
Free standing column

Third floor studio space

Supports roof load = 20 psf DL + 30 psf SL

$P_u = 1.2(20) + 1.6(30) = 72 \text{ psf}$

$\phi P_n = 1600 (72) = 115200 \text{ lbs} = 115.2 \text{ k}$



$\phi P_n = 1600 (72) = 115200 \text{ lbs} = 115.2 \text{ k}$

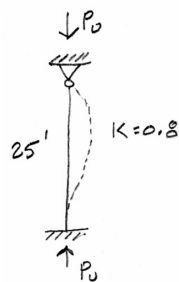
Table 4-1a (continued) Available Strength in Axial Compression, kips														$F_y = 50$ ksi
W-Shapes														
Shape		WBx												
lb/ft		67		58		48		40		35		31		
Design		P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_e (ft), with respect to least radius of gyration, r_y	0	590	886	512	769	422	634	350	526	308	463	273	411	
	6	542	815	470	706	387	581	320	481	281	423	249	374	
	7	526	790	455	685	375	563	309	465	272	409	241	362	
	8	508	763	439	660	361	543	298	448	262	394	232	348	
	9	488	733	422	634	347	521	285	429	251	377	222	333	
	10	467	701	403	606	331	497	272	409	239	359	211	317	
	11	444	668	384	576	314	473	258	388	226	340	200	300	
	12	421	633	363	546	297	447	243	366	213	321	189	283	
	13	397	597	342	514	280	421	228	343	200	301	177	266	
	14	373	560	321	482	262	394	213	321	187	281	165	248	
	15	348	523	299	450	244	367	198	298	174	261	153	230	
	16	324	487	278	418	226	340	183	275	160	241	141	212	
	17	300	450	257	386	209	314	169	253	147	221	130	195	
	18	276	415	236	355	192	288	154	232	135	203	118	178	
	19	253	381	216	325	175	264	141	211	123	184	108	162	
	20	231	347	197	296	159	239	127	191	111	166	97.2	146	
	22	191	287	163	244	132	198	105	158	91.5	138	80.3	121	
	24	160	241	137	205	111	166	88.2	133	76.9	116	67.5	101	
	26	137	205	116	175	94.2	142	75.2	113	65.5	98.5	57.5	86.5	
	28	118	177	100	151	81.2	122	64.8	97.4	56.5	84.9	49.6	74.5	
30	103	154	87.5	131	70.7	106	56.5	84.9	49.2	74.0	43.2	64.9		
32	90.3	136	76.9	116	62.2	93.5	49.6	74.6	43.3	65.0	38.0	57.1		
34	79.9	120	68.1	102	55.1	82.8	44.0	66.1						
Properties														
P_{n0} , kips	126	190	102	153	72.0	108	57.2	85.9	45.9	68.9	39.4	59.1		
P_{n1} , kip/in.	19.0	28.5	17.0	25.5	13.3	20.0	12.0	18.0	10.3	15.5	9.50	14.3		
P_{n2} , kips	507	761	363	546	174	262	127	192	81.1	122	63.0	94.7		
P_{n3} , kips	164	246	123	185	87.8	132	58.7	88.2	45.9	68.9	35.4	53.2		
L_{p0} , ft	7.49		7.42		7.35		7.21		7.17		7.18			
L_{p1} , ft	47.6		41.6		35.2		29.9		27.0		24.8			
A_g , in. ²	19.7		17.1		14.1		11.7		10.3		9.13			
I_y , in. ⁴	272		228		184		146		127		110			
I_x , in. ⁴	88.6		75.1		60.9		49.1		42.6		37.1			
r_{yx} , in.	2.12		2.10		2.08		2.04		2.03		2.02			
r_{xy} , in.	1.75		1.74		1.74		1.73		1.78		1.72			
$P_{n4}/L^2/10^4$, k-in. ²	7790		6530		5270		4180		3630		3150			
$P_{n5}/L^2/10^4$, k-in. ²	2540		2150		1740		1410		1220		1080			
ASD		LRFD		Note: Heavy line indicates L_{e/r_y} equal to or greater than 200.										
$\Omega_c = 1.67$		$\phi_c = 0.90$												

Shape	HSS8x8						HSS7x7						
	1/4		3/16		1/2		3/8		1/2		3/4		
	ϕ_t	ϕ_c	ϕ_t	ϕ_c	ϕ_t	ϕ_c	ϕ_t	ϕ_c	ϕ_t	ϕ_c	ϕ_t	ϕ_c	
lb/ft	25.82	18.63	13.26	50.81	42.05	32.58							
Design	P_u/ϕ_t	P_u/ϕ_c	P_u/ϕ_t	P_u/ϕ_c	P_u/ϕ_t	P_u/ϕ_c	P_u/ϕ_t	P_u/ϕ_c	P_u/ϕ_t	P_u/ϕ_c	P_u/ϕ_t	P_u/ϕ_c	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_e (ft), with respect to least radius of gyration, r_y	0	213	319	137	206	66.6	100	419	630	347	522	269	404
	6	205	308	134	201	65.2	98.0	396	595	329	494	255	383
	7	202	303	133	199	64.7	97.2	388	583	322	484	250	376
	8	199	299	131	197	64.1	96.3	379	569	315	474	245	368
	9	195	293	130	195	63.4	95.3	369	554	307	461	239	359
	10	191	287	128	193	62.7	94.2	358	538	298	440	232	349
	11	187	281	126	190	61.9	93.0	346	520	289	434	225	338
	12	182	274	124	187	61.0	91.7	334	502	279	419	218	327
	13	178	267	122	184	60.1	90.3	321	482	269	404	210	316
	14	173	260	120	180	59.1	88.8	307	462	258	387	202	303
	15	167	252	118	177	58.0	87.2	294	441	247	371	194	291
	16	162	243	115	173	56.9	85.6	280	420	235	354	185	278
	17	156	235	112	169	55.8	83.8	265	399	224	336	176	265
	18	151	227	110	165	54.6	82.0	251	377	212	319	168	252
	19	145	218	107	160	53.3	80.1	237	356	200	301	159	239
	20	139	209	104	156	52.0	78.2	223	335	189	284	150	226
	21	133	200	101	151	50.7	76.2	209	314	177	267	141	212
	22	127	191	97.1	146	49.3	74.2	195	293	166	250	133	200
	23	121	182	92.7	139	47.9	72.1	182	273	155	233	124	187
	24	115	173	88.3	133	46.5	69.9	169	253	145	217	116	175
25	110	165	83.9	126	45.1	67.8	156	234	134	201	108	163	
26	104	156	79.5	120	43.6	65.6	144	216	124	186	100	151	
27	98.1	147	75.3	113	42.2	63.4	133	201	115	173	92.9	140	
28	92.5	139	71.1	107	40.7	61.1	124	186	107	161	86.4	130	
29	87.1	131	67.0	101	39.2	58.9	116	174	99.6	150	80.6	121	
30	81.7	123	63.0	94.7	37.7	56.6	108	162	93.1	140	75.3	113	
32	71.8	108	55.4	83.2	34.6	52.0	95.0	143	81.8	123	66.2	99.4	
34	63.6	95.6	49.0	73.7	31.9	48.0	84.1	126	72.4	109	58.6	88.1	
36	56.7	85.3	43.7	65.7	29.5	44.4	75.1	113	64.6	97.1	52.3	78.6	
38	50.9	76.5	39.3	59.0	27.0	40.5	67.4	101	58.0	87.2	46.9	70.5	
40	46.0	69.1	35.4	53.2	24.3	36.6	60.8	91.4	52.3	78.7	42.3	63.6	
Properties													
A_g , in. ²	7.10		5.37		3.62		14.0		11.6		8.97		
I_y , in. ⁴	70.7		54.4		37.4		93.4		80.5		65.0		
r_y , in.	3.15		3.18		3.21		2.58		2.63		2.69		
ASD			LRFD										
$\Omega_c = 1.67$			$\phi_c = 0.90$										

AISC Critical Stress Table

for previous example $Kl/r_y = 118.2$

$W8 \times 35$
 $F_y = 50 \text{ ksi}$
 $E = 29,000 \text{ ksi}$
 $L = 25' \text{ (NO BRACING)}$



Slenderness $\gamma\text{-y}$

$$\frac{Kl_y}{r_y} = \frac{0.8(25')}{2.03} = 118.2$$

TO FIND CAPACITY:

$$\phi F_{cr} = 16.2 \text{ ksi}$$

$$\phi P_n = P_u = \phi F_{cr} A_g$$

$$P_u = 16.2(10.3) = \underline{166.8 \text{ k}}$$

Table 4-22 (continued)
Available Critical Stress for
Compression Members

$F_y = 35 \text{ ksi}$			$F_y = 36 \text{ ksi}$			$F_y = 42 \text{ ksi}$			$F_y = 46 \text{ ksi}$			$F_y = 50 \text{ ksi}$		
$\frac{KL}{r}$	$\frac{F_{cr}}{\Omega_c}$	ϕF_{cr}	$\frac{KL}{r}$	$\frac{F_{cr}}{\Omega_c}$	ϕF_{cr}	$\frac{KL}{r}$	$\frac{F_{cr}}{\Omega_c}$	ϕF_{cr}	$\frac{KL}{r}$	$\frac{F_{cr}}{\Omega_c}$	ϕF_{cr}	$\frac{KL}{r}$	$\frac{F_{cr}}{\Omega_c}$	ϕF_{cr}
ASD	LRFD		ASD	LRFD		ASD	LRFD		ASD	LRFD		ASD	LRFD	
81	15.0	22.5	81	15.3	22.9	81	16.8	25.3	81	17.7	26.6	81	18.5	27.9
82	14.9	22.3	82	15.1	22.7	82	16.6	25.0	82	17.5	26.3	82	18.3	27.5
83	14.7	22.1	83	15.0	22.5	83	16.5	24.8	83	17.3	26.0	83	18.1	27.2
84	14.6	22.0	84	14.9	22.3	84	16.3	24.5	84	17.1	25.8	84	17.9	26.9
85	14.5	21.8	85	14.7	22.1	85	16.1	24.3	85	16.9	25.5	85	17.7	26.5
86	14.4	21.6	86	14.6	22.0	86	16.0	24.0	86	16.7	25.2	86	17.4	26.2
87	14.2	21.4	87	14.5	21.8	87	15.8	23.7	87	16.6	24.9	87	17.2	25.9
88	14.1	21.2	88	14.3	21.6	88	15.6	23.5	88	16.4	24.6	88	17.0	25.5
89	14.0	21.0	89	14.2	21.4	89	15.5	23.2	89	16.2	24.3	89	16.8	25.2
90	13.8	20.8	90	14.1	21.2	90	15.3	23.0	90	16.0	24.0	90	16.6	24.9
91	13.7	20.6	91	13.9	21.0	91	15.1	22.7	91	15.8	23.7	91	16.3	24.6
92	13.6	20.4	92	13.8	20.8	92	15.0	22.5	92	15.6	23.4	92	16.1	24.2
93	13.5	20.2	93	13.7	20.5	93	14.8	22.2	93	15.4	23.1	93	15.9	23.9
94	13.3	20.0	94	13.5	20.3	94	14.6	22.0	94	15.2	22.8	94	15.7	23.6
95	13.2	19.9	95	13.4	20.1	95	14.4	21.7	95	15.0	22.6	95	15.5	23.3
96	13.1	19.7	96	13.3	19.9	96	14.3	21.5	96	14.8	22.3	96	15.3	22.9
97	13.0	19.5	97	13.1	19.7	97	14.1	21.2	97	14.6	22.0	97	15.0	22.6
98	12.8	19.3	98	13.0	19.5	98	13.9	21.0	98	14.4	21.7	98	14.8	22.3
99	12.7	19.1	99	12.9	19.3	99	13.8	20.7	99	14.2	21.4	99	14.6	22.0
100	12.6	18.9	100	12.7	19.1	100	13.6	20.5	100	14.1	21.1	100	14.4	21.7
101	12.4	18.7	101	12.6	18.9	101	13.4	20.2	101	13.9	20.8	101	14.2	21.3
102	12.3	18.5	102	12.5	18.7	102	13.3	20.0	102	13.7	20.6	102	14.0	21.0
103	12.2	18.3	103	12.3	18.5	103	13.1	19.7	103	13.5	20.3	103	13.8	20.7
104	12.1	18.1	104	12.2	18.3	104	12.9	19.5	104	13.3	20.0	104	13.6	20.4
105	11.9	17.9	105	12.1	18.1	105	12.8	19.2	105	13.1	19.7	105	13.4	20.1
106	11.8	17.7	106	11.9	17.9	106	12.6	19.0	106	12.9	19.4	106	13.2	19.8
107	11.7	17.5	107	11.8	17.7	107	12.4	18.7	107	12.8	19.2	107	13.0	19.5
108	11.5	17.3	108	11.7	17.5	108	12.3	18.5	108	12.6	18.9	108	12.8	19.2
109	11.4	17.2	109	11.5	17.3	109	12.1	18.2	109	12.4	18.6	109	12.6	18.9
110	11.3	17.0	110	11.4	17.1	110	12.0	18.0	110	12.2	18.3	110	12.4	18.6
111	11.2	16.8	111	11.3	16.9	111	11.8	17.7	111	12.0	18.1	111	12.2	18.3
112	11.0	16.6	112	11.1	16.7	112	11.6	17.5	112	11.8	17.8	112	12.0	18.0
113	10.9	16.4	113	11.0	16.5	113	11.5	17.3	113	11.7	17.5	113	11.8	17.7
114	10.8	16.2	114	10.9	16.3	114	11.3	17.0	114	11.5	17.3	114	11.6	17.4
115	10.7	16.0	115	10.7	16.2	115	11.2	16.8	115	11.3	17.0	115	11.4	17.1
116	10.5	15.8	116	10.6	16.0	116	11.0	16.5	116	11.1	16.7	116	11.2	16.8
117	10.4	15.6	117	10.5	15.8	117	10.8	16.3	117	11.0	16.5	117	11.0	16.5
118	10.3	15.5	118	10.4	15.6	118	10.7	16.1	118	10.8	16.2	118	10.8	16.2
119	10.2	15.3	119	10.2	15.4	119	10.5	15.8	119	10.6	16.0	119	10.6	16.0
120	10.0	15.1	120	10.1	15.2	120	10.4	15.6	120	10.4	15.7	120	10.4	15.7
ASD	LRFD		ASD	LRFD		ASD	LRFD		ASD	LRFD		ASD	LRFD	
$\Omega_c = 1.67$			$\phi_c = 0.90$											

Steel Connections

Methods of Connections

Bolted



Welded



Steel Connections

Shop vs. Field Connections

Shop Connections:

- Welding preferably performed in the shop as opposed to the field due to controlled environment
- Members can be positioned for more economical welding (welding upside down is difficult)
- Welding may have an equipment advantage in the shop
- Shops use both welding and bolting



Field Connections:

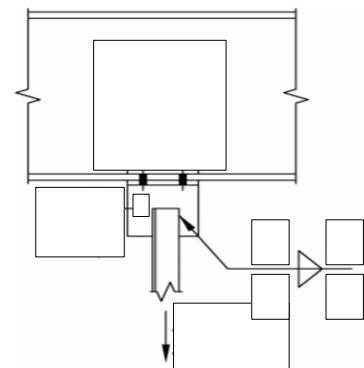
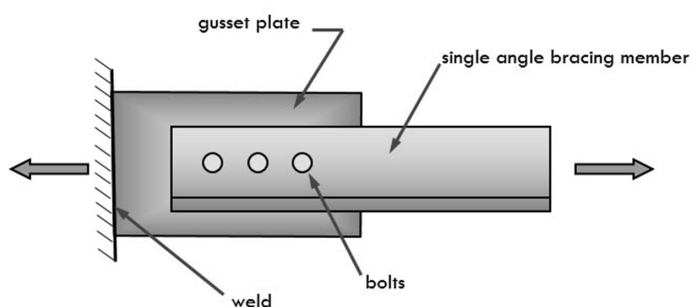
- Bolting easily performed in the field and generally preferred when possible
- Bolting provides a method to erect the members and release the crane hook quickly



Steel Connections

Failure modes – Limit States

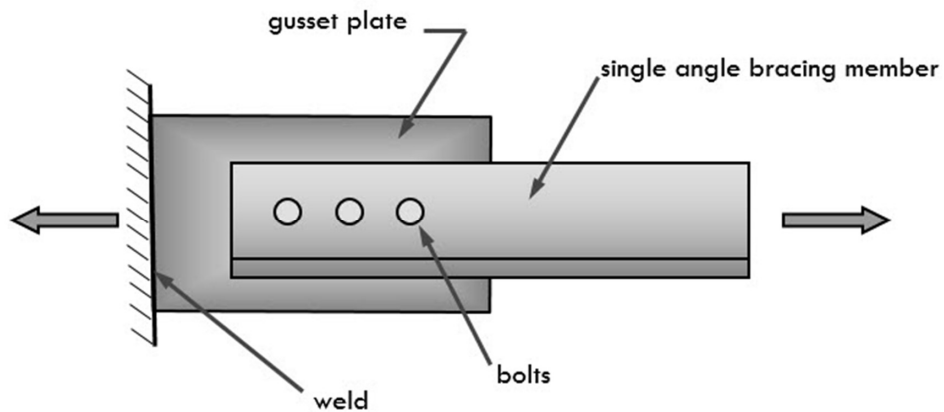
- Fasteners (bolts or welds)
 - shear
 - tension
 - bearing
- Connecting elements (plates or tees)
 - tension
 - block shear
 - tear out
- Supporting or supported members



Tension Connection: Example Angle – Bolts – Gusset Plate

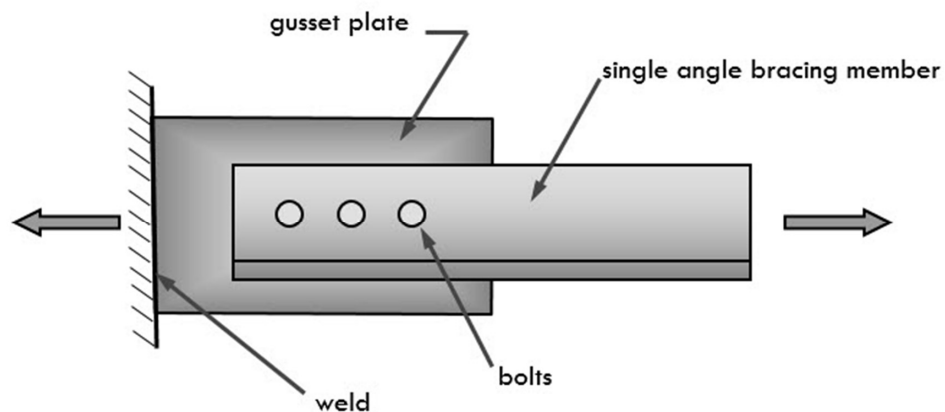
Load Path

Angle → Bolts → Gusset plate → Weld → Support



Tension Connection – Angle Failure example

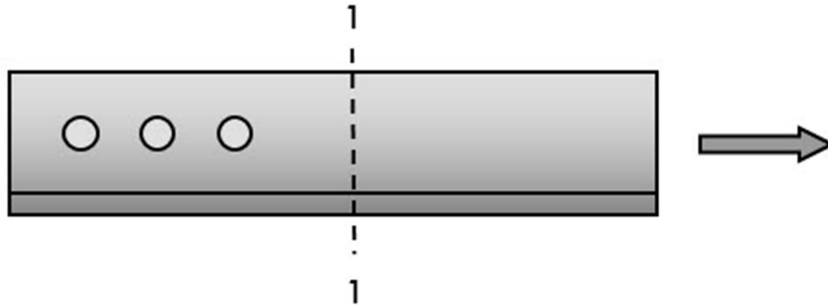
1. Tensile Yielding
2. Tensile Rupture
3. Block Shear
4. Bearing and Tearout at Bolt Holes
5. Bolt Shear
6. Bearing and Tearout at Bolt Holes
7. Block Shear
8. Tensile Rupture
9. Tensile Yielding
10. Tension Rupture in Weld



Tension Connection – Angle Failure

1. Tensile Yielding

- at gross section $R_n = F_y A_g$ $\phi = 0.9$
- F_y = minimum yield stress, ksi
- A_g = gross area of member, in²



Tension Connection – Angle Failure

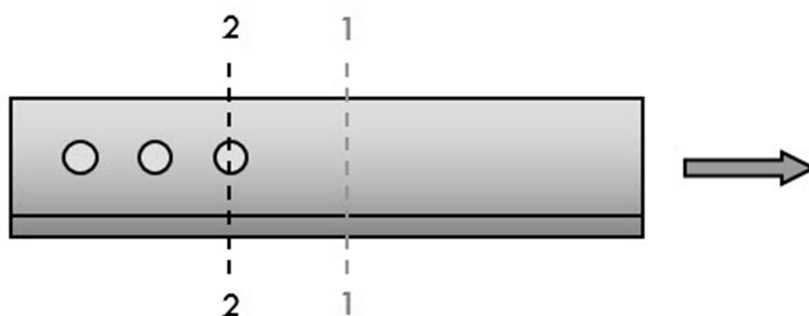
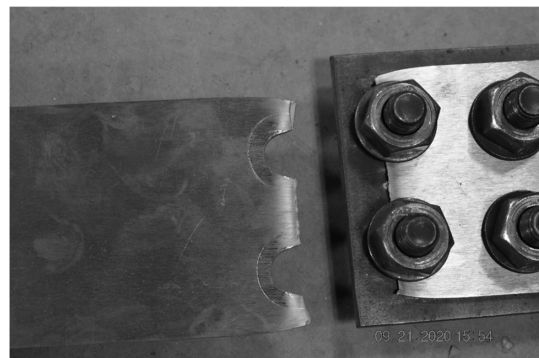
2. Tensile Rupture

Flat Bar

- $R_n = F_u A_e$ $\phi = 0.75$
- F_u = minimum tensile strength, ksi
- A_e = effective net area, in²

Section (not flat)

- $A_e = A_n U$
- A_n = net area
- U = shear lag factor (Table D3.1)



Tension Connection

Angle Failure

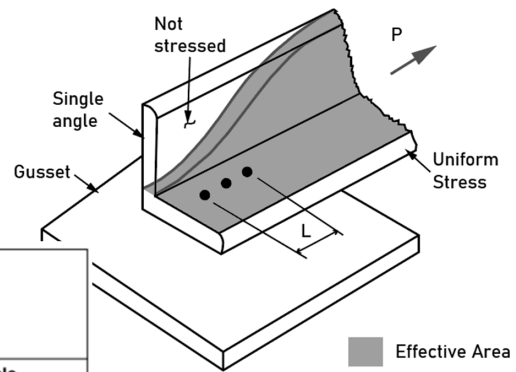


TABLE D3.1 Shear Lag Factors for Connections to Tension Members			
Case	Description of Element	Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).	$U = 1.0$	—
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)	$U = 1 - \frac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.	$U = 1.0$ and $A_n = \text{area of the directly connected elements}$	—
4 ^[a]	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.	$U = \frac{3l^2}{3l^2 + w^2} \left(1 - \frac{\bar{x}}{l} \right)$	
5	Round HSS with a single concentric gusset plate through slots in the HSS.	$l \geq 1.3D, U = 1.0$ $D \leq l < 1.3D, U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{D}{\pi}$	

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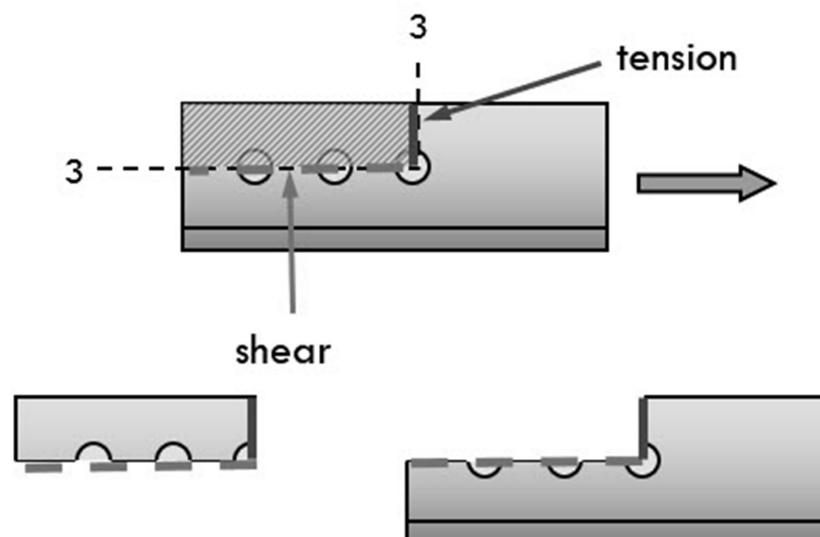
Structures II

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Tension Connection – Angle Failure

3. Block Shear

- $R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt}$ $\phi = 0.75$
- A_{nv} = net area in shear
- A_{nt} = net area in tension
- $U_{bs} = 1.0$ (uniform stress) $U_{bs} = 0.5$ (non-uniform stress)



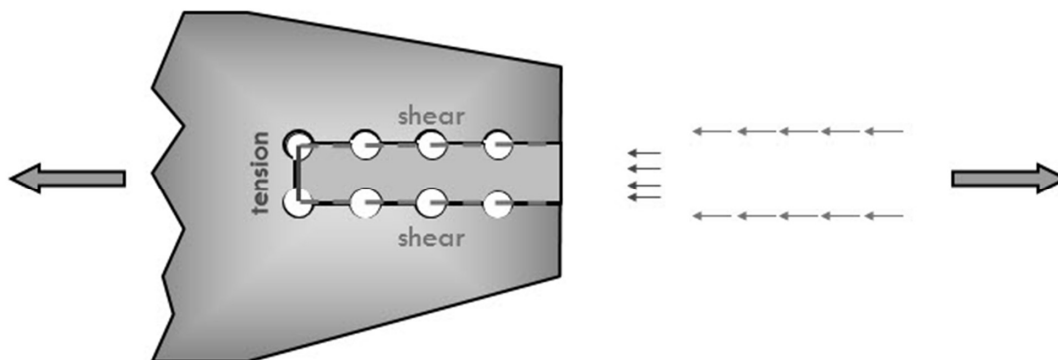
University of Michigan, TCAUP

Structures II

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Tension Connection

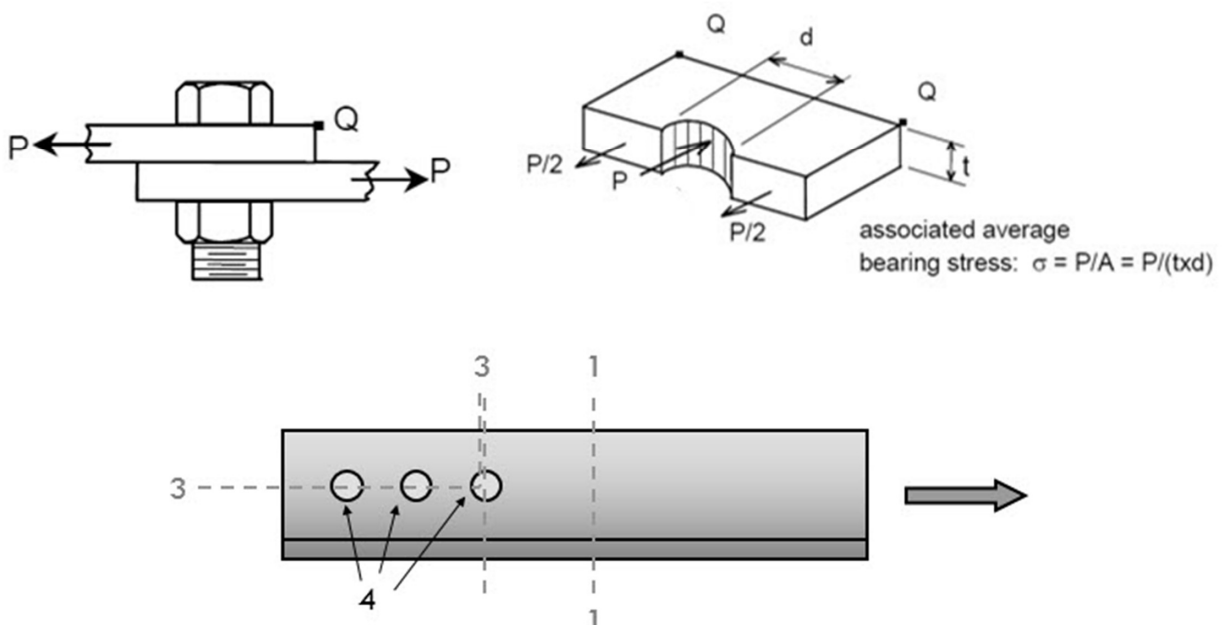
Block Shear Example



Tension Connection - Bolt Failure

4. Bearing and Tearout at Bolt Holes

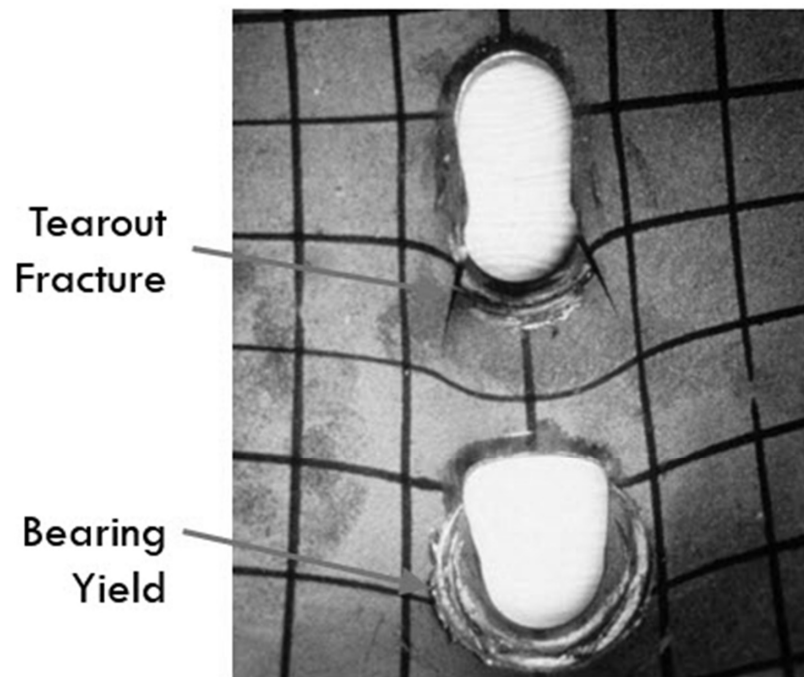
- **Bearing:** deformation of material at the loaded edge of the bolt holes
- **Tearout:** block shear rupture between bolts or at the edge due to bearing



Tension Connection - Bolt Failure

4. Bearing and Tearout at Bolt Holes

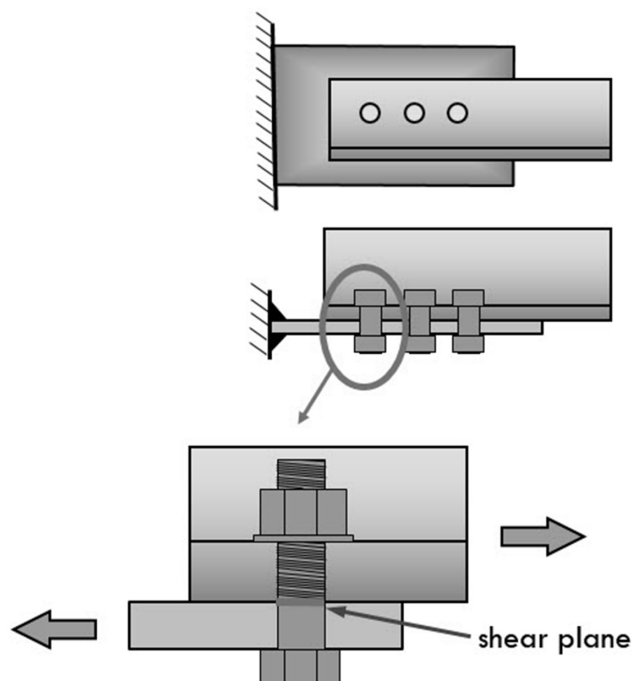
- **Bearing:** deformation of material at the loaded edge of the bolt holes
- **Tearout:** block shear rupture between bolts or at the edge due to bearing



Tension Connection - Bolt Failure

5. Bolt Shear

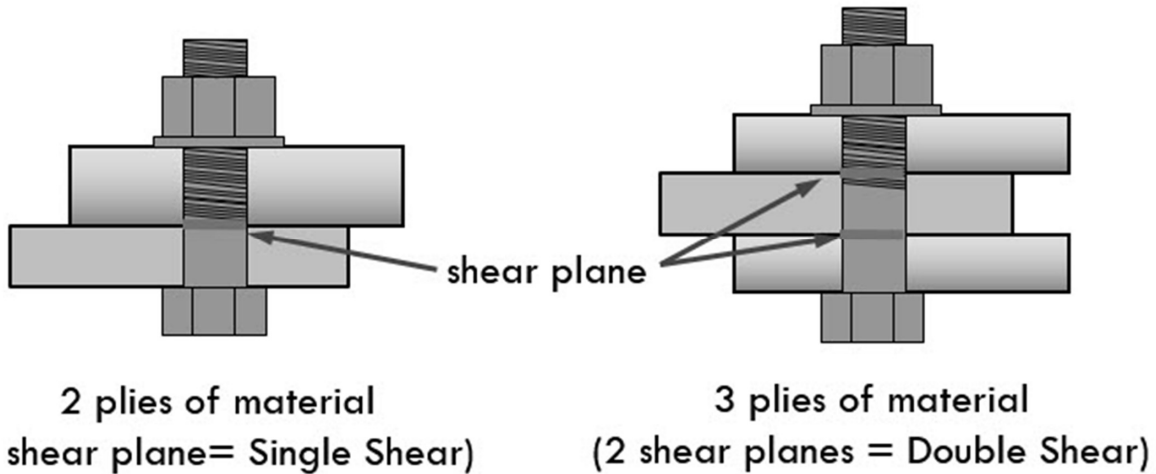
- Shear failure of the bolts along the shear plane (interface)



Tension Connection - Bolt Failure

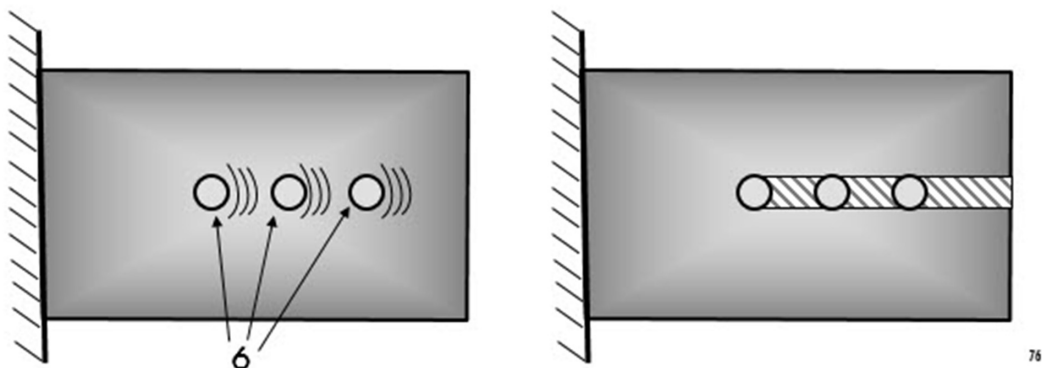
5. Bolt Shear

- Shear failure of the bolts along the shear plane (interface)
- Single shear vs. double shear
- $R_n = F_n A_b \quad \phi = 0.75$
- F_n = nominal shear stress, F_{nv} (or tensile stress F_{nt})
- A_b = nominal bolt area (threaded or unthreaded)



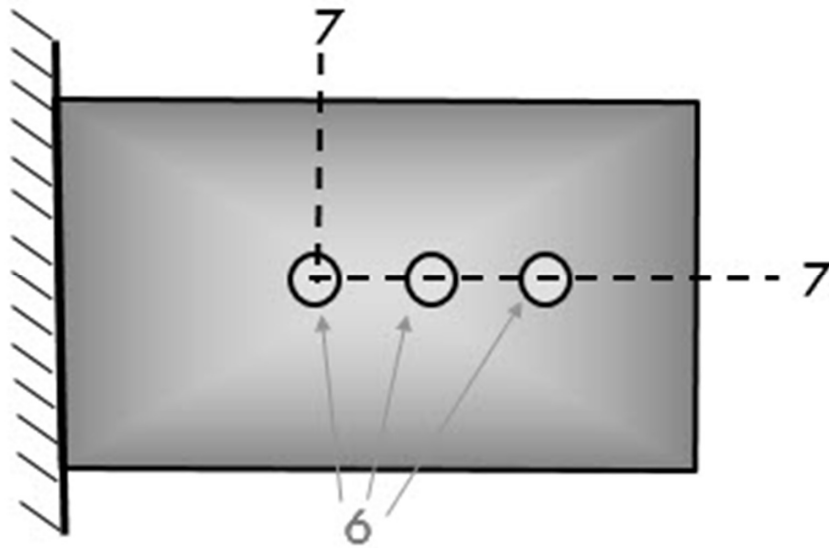
Tension Connection – Gusset Plate Failure

6. Bearing and Tearout at Bolt Holes



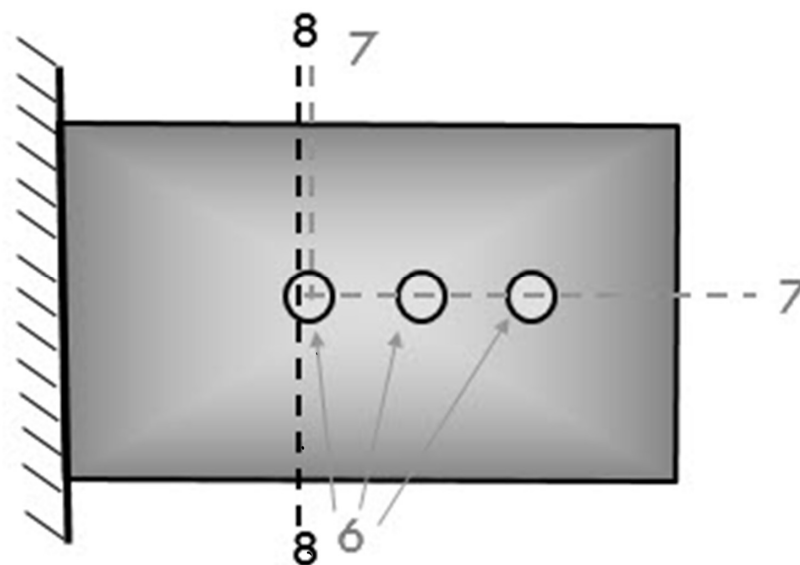
Tension Connection – Gusset Plate Failure

7. Block Shear



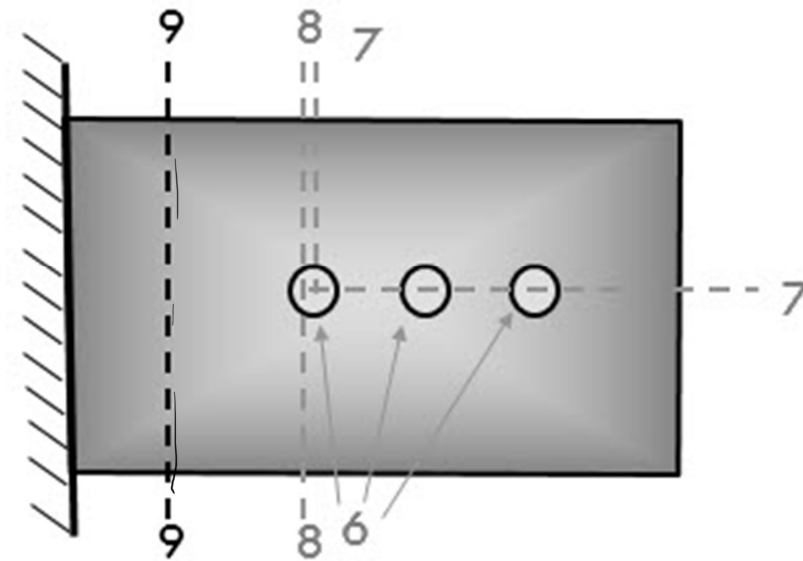
Tension Connection – Gusset Plate Failure

8. Tensile Rupture



Tension Connection – Gusset Plate Failure

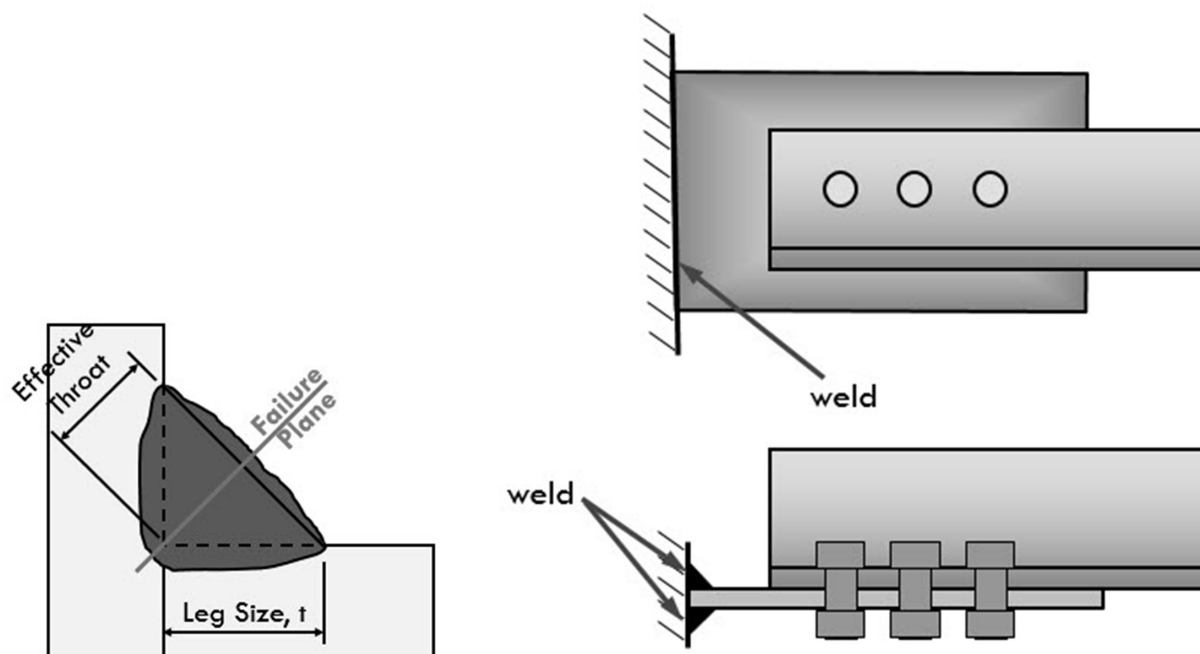
9. Tensile Yielding



Tension Connection – Gusset Plate Failure

10. Tension Rupture in Weld

- Shear failure on the effective throat of the weld



Steel Frame Construction



University of Michigan – North Quad

Steel Frame Construction Messe Leipzig – 1996

Congress Centre – Gerkan, Marg und Partner
Glass Hall – Ian Ritchie Architects
Tower - Schlaich, Bergermann und Partner



Messe Leipzig - Glass Hall - Ian Ritchie Architects



Messe Leipzig – Cable braced tower. Jörg Schlaich

Steel Frame Construction



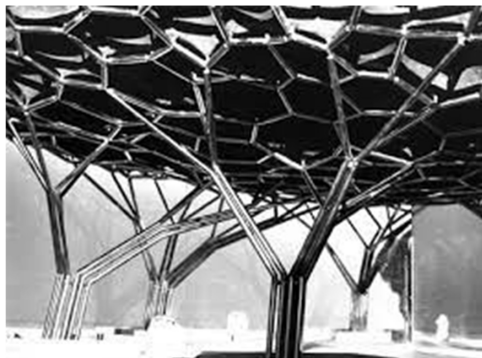
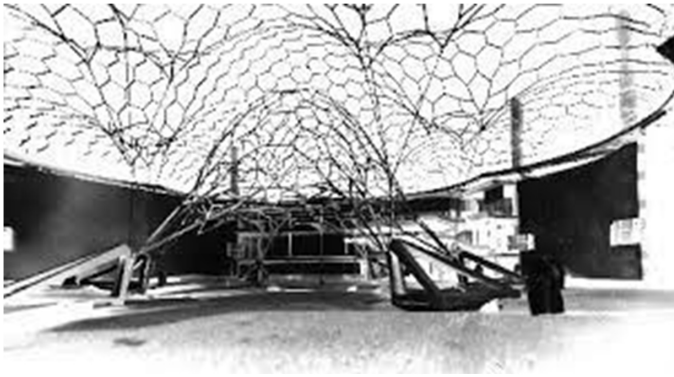
Messe Leipzig Glass Hall - Ian Ritchie Architects

Steel Frame Construction



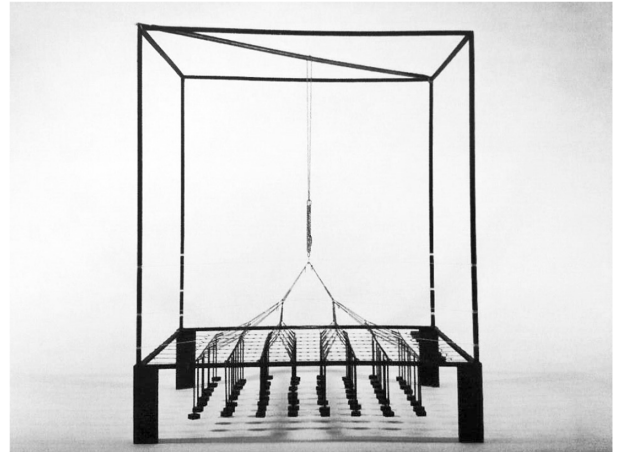
Messe Leipzig Glass Hall - Ian Ritchie Architects

Branching Columns (tree columns) Frei Otto



Kocommas, Majilis al Shura

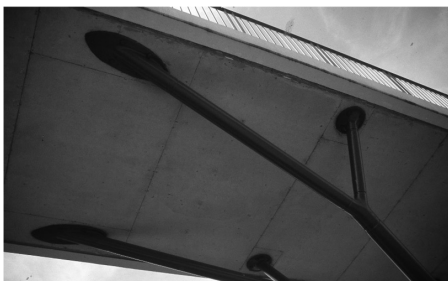
University of Michigan, TCAUP



Structures II

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Branching Columns (tree columns)



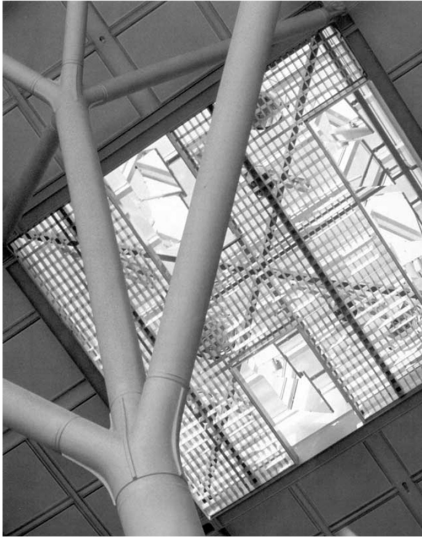
bridge in Pragsattel, Stuttgart, 1992
Schlaich, Bergemann und Partner

University of Michigan, TCAUP

Structures II

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Branching Columns (tree columns)



Stuttgart Airport Terminal,
Gerkan, Marg und Partner
Schlaich, Bergemann und Partner