

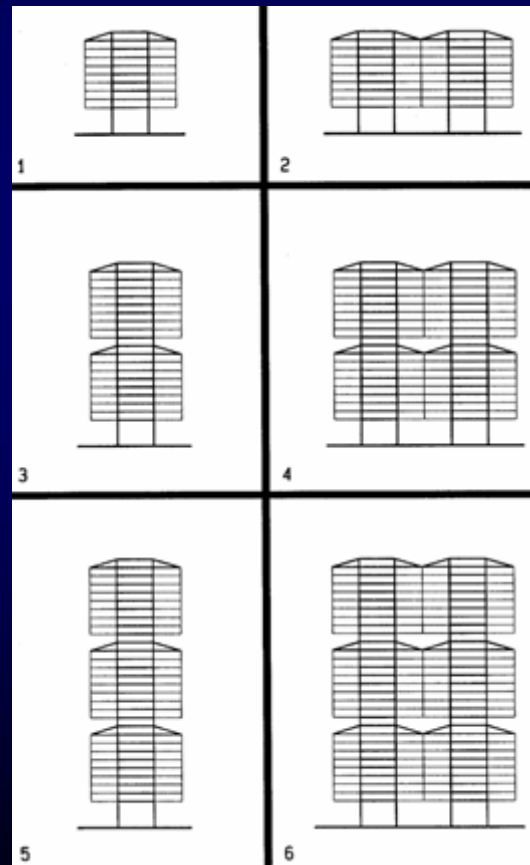
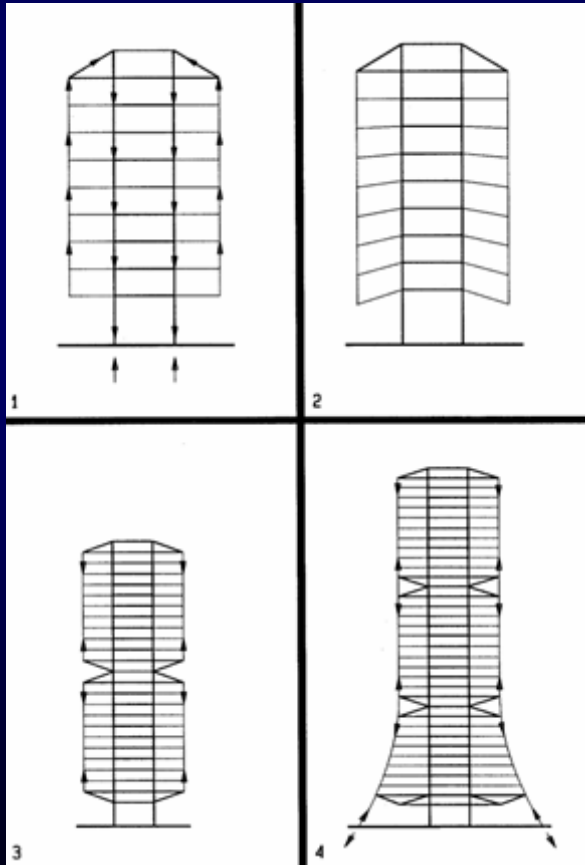


Suspended high-rise

Suspended high-rise

- 1 Gravity load path
- 2 Differential deflection
- 3 Prestress to reduce deflection
- 4 Ground anchors for stability

- 1 Single tower
- 2 Multiple towers
- 3 Multiple stacks
- 4 Multiple stacks / towers
- 5 Triple stacks
- 6 Triple stacks / twin towers



Challenges

- Load path detour: load travels up to top, then down to foundation
- Combined hanger / column deflection yields large differential deflection

Architectural rational

- Column-free ground floor
- Planning flexibility at ground floor
- Facilitates top down future expansion with minimal operation interference
- Small hangers replace large columns

Structural rational

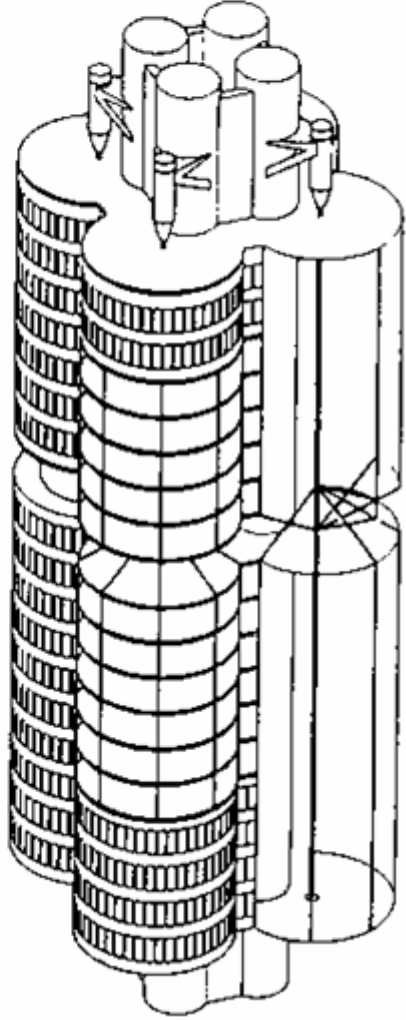
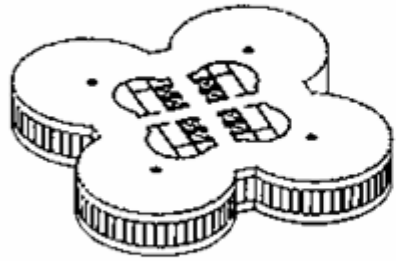
- Eliminates buckling in hangers, replacing compression by tension
- High-strength hangers replace large compression columns
- Concentration of compression to a few large columns minimizes buckling

Options

- Multiple towers to reduce lateral drift
- Multiple stacks control deflection
- Adjust hangers for DL and partial LL to reduce deflection
- Prestress hangers to reduce deflection

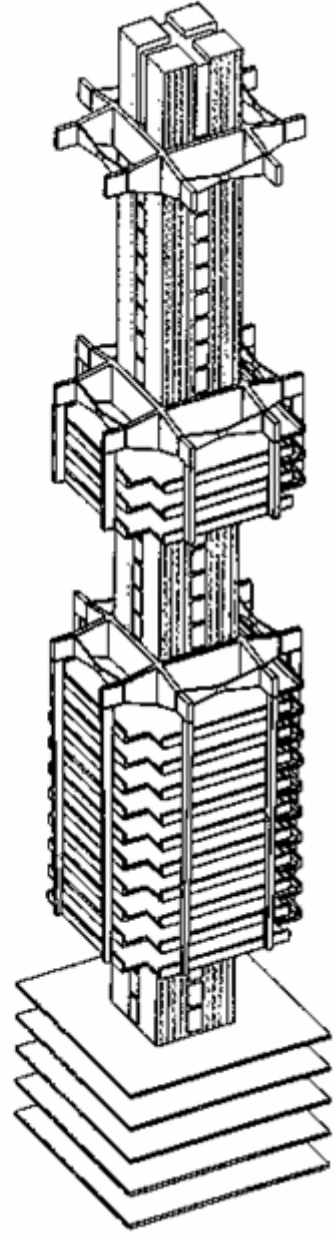
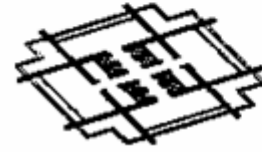
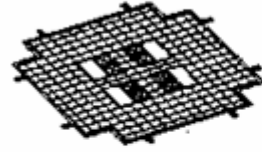
BMW headquarters Munich

Architect: Karl Schwazer



Standard Bank Center, Johannesburg

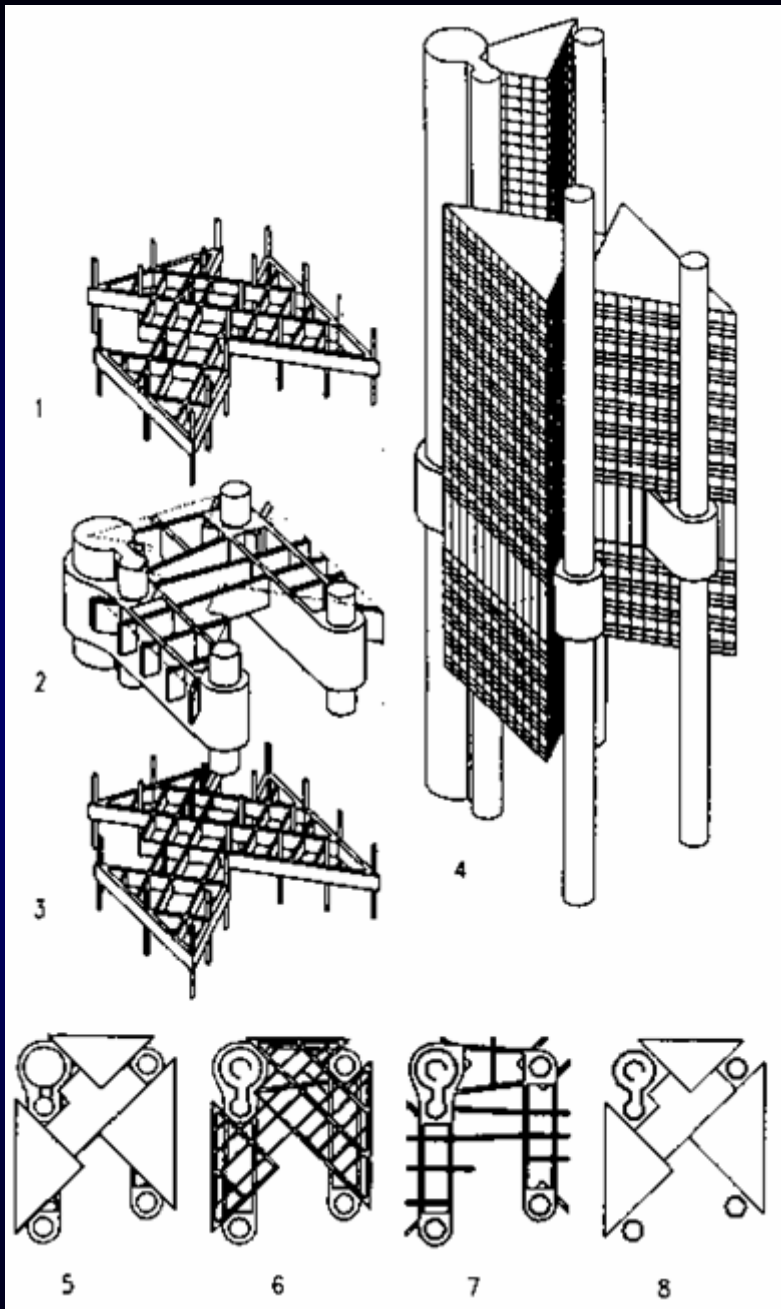
Architect: Hentrich and Petschnigg



Hypo Bank Munich

Architect: Bea and Walter Betz

Four circular towers support a mid-level mechanical floor that supports the floors above while floors below are suspended from it.

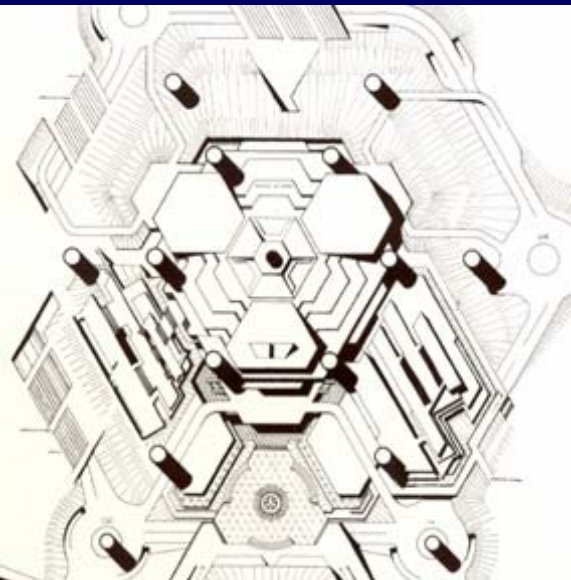




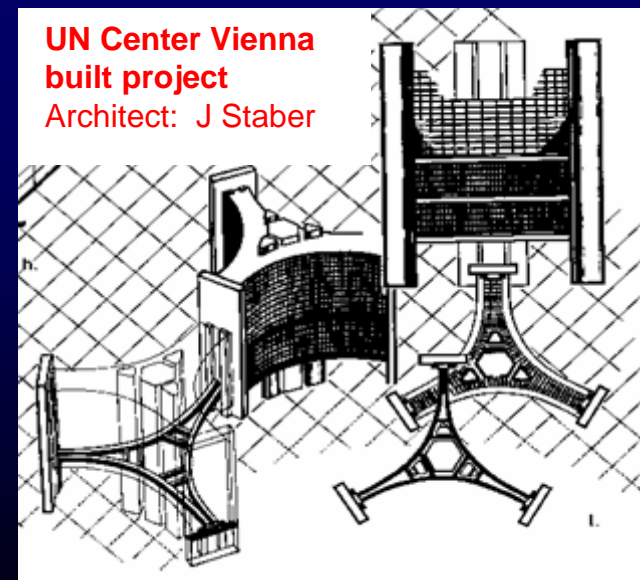
Design objectives:
Independent expansion
of conference center and
offices was required



Triangular grid allows
horizontal expansion of
conference center in three
directions



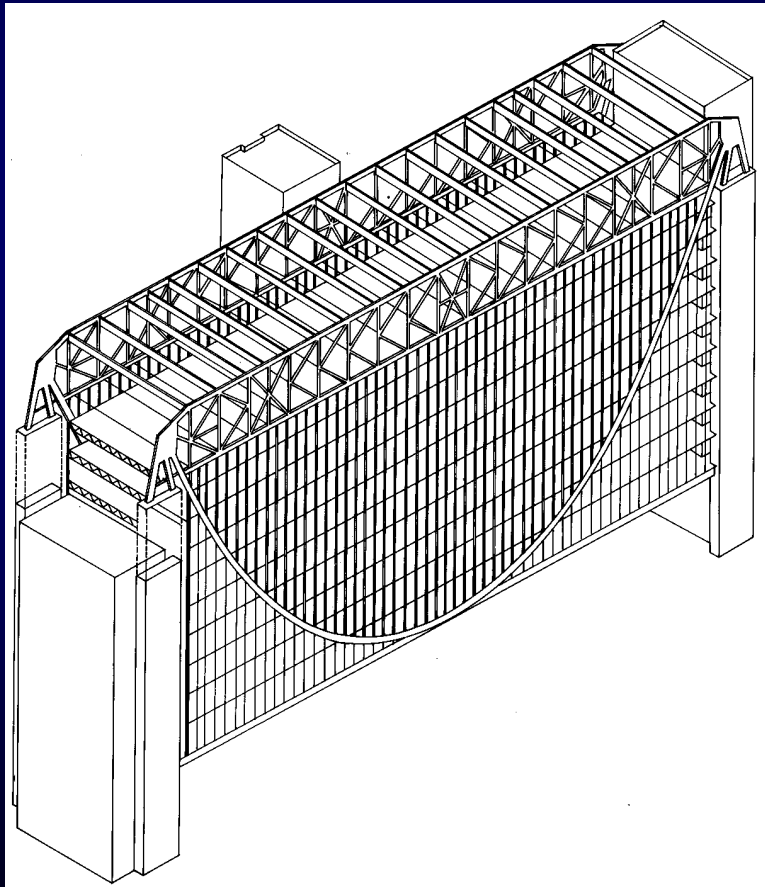
Suspended high-rise
allows independent
top-down expansion



Federal Reserve Bank, Minneapolis

Architect: Gunnar Birkerts

- Parabolic suspenders are supported by 2 towers
- Top trusses resist lateral suspender thrust
- Floors below parabola are suspended
- Floors above parabola are supported by columns
- Support type is expressed on the facade



Westcoast Transmission Tower, Vancouver

Architect: Rhone & Iredale

Engineer: Bogue Babicki

Assume:

Concrete core wall thickness

$$t = 1'$$

Suspender cables

$$2 \phi 2 \frac{7}{8}"$$

Guy cables

$$2\phi 2 \frac{7}{8}" + 2\phi 2 \frac{1}{2}"$$

Average wind pressure (80mph, exposure B, @160')

$$P = q_s C_e C_q = 16.4 \times 1.31 \times 1.4$$

$$P = 30 \text{ psf}$$

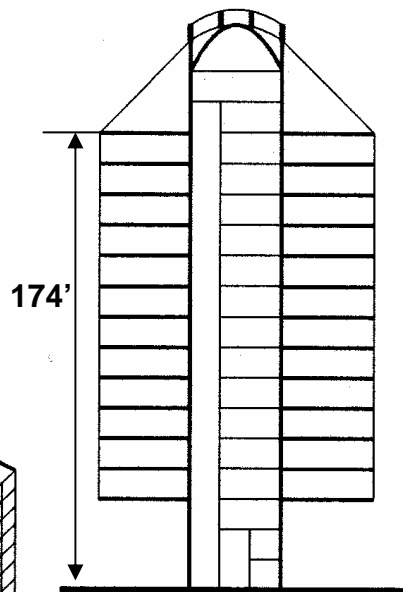
Live load reductions

$$\text{Beam: } R = 0.08(A-150) = 0.08(12' \times 36' - 150)$$

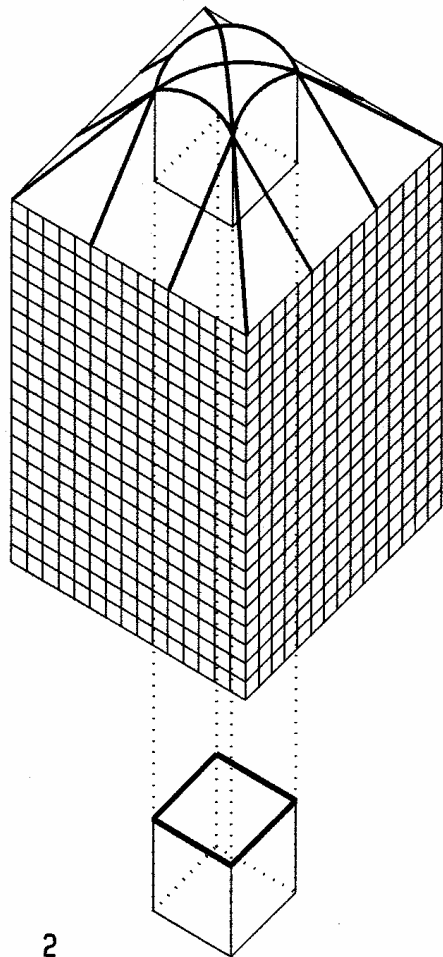
$$R = 23 \%$$

Suspender: max. R

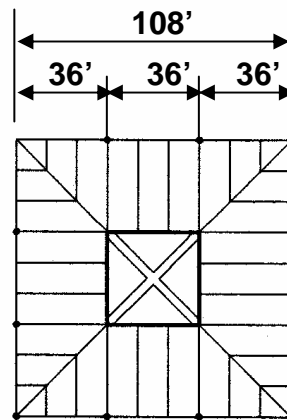
$$R = 60 \%$$



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Gravity loads

Concrete slab = 60 psf

Partitions = 20 psf

Framing = 15 psf

Floor/ceiling = 5 psf

DL = 100 psf

Beam live load

$$(100\% - 23\% = 0.77)$$

$$0.77(50) \quad LL = 39 \text{ psf}$$

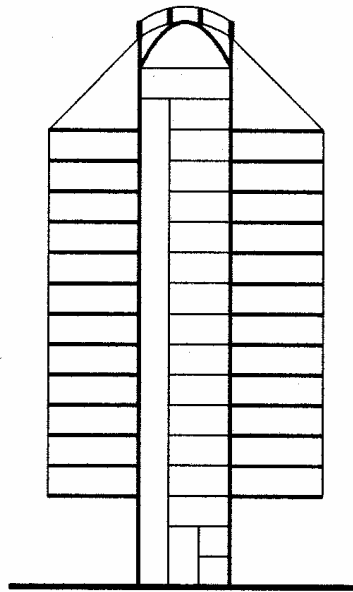
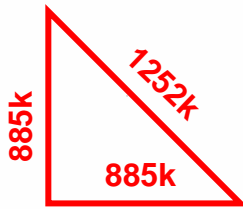
Suspender live load

$$\underline{0.4(50) \quad LL = 20 \text{ psf}}$$

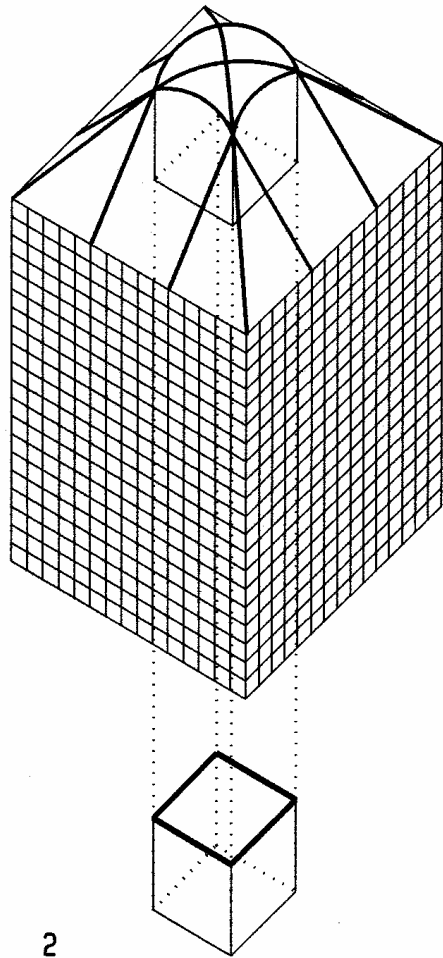
Total loads:

Beam 139 psf

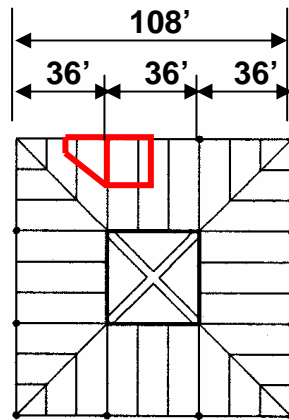
Suspender 120 psf



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Uniform beam load

$$w = 139 \text{ psf} \times 12' / 1000$$

$$w = 1.7 \text{ klf}$$

Beam bending

$$M = wL^2 / 8 = 1.7 \times 36^2 / 8$$

$$M = 275 \text{ k'}$$

$$S = M / F_b = 275 \times 12 / 22$$

$$S = 150 \text{ in}^3$$

Use W21x73

$$S = 151 > 150$$

Suspender load

$$P = 13 \times 120 \text{ psf} \times [18^2 + 18 \times (18 + 9) / 2] / 1000$$

$$P = 885 \text{ k}$$

Suspender cross section (twin 2 7/8", 70% metallic)

$$A = 2 \pi 0.7 (2.875 / 2)^2$$

$$A = 9 \text{ in}^2$$

Suspender stress

$$f = P / A = 885 / 9$$

$$f = 98 \text{ ksi}$$

Guy force (from vector graph)

$$P = 1252 \text{ k}$$

Guy cross section (2 suspenders + 2 - 2.5" strands)

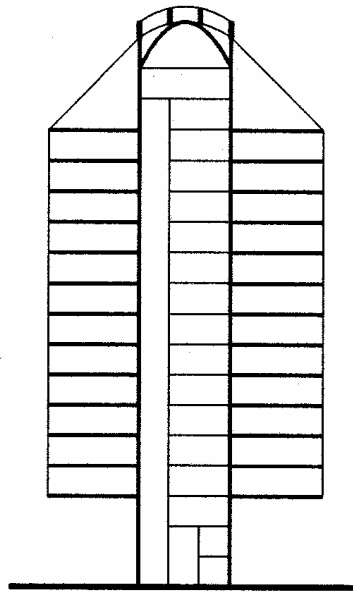
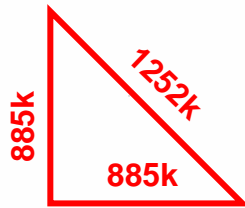
$$A = 9 \text{ in}^2 + 2 \pi 0.7 (2.5 / 2)^2$$

$$A = 15.9 \text{ in}^2$$

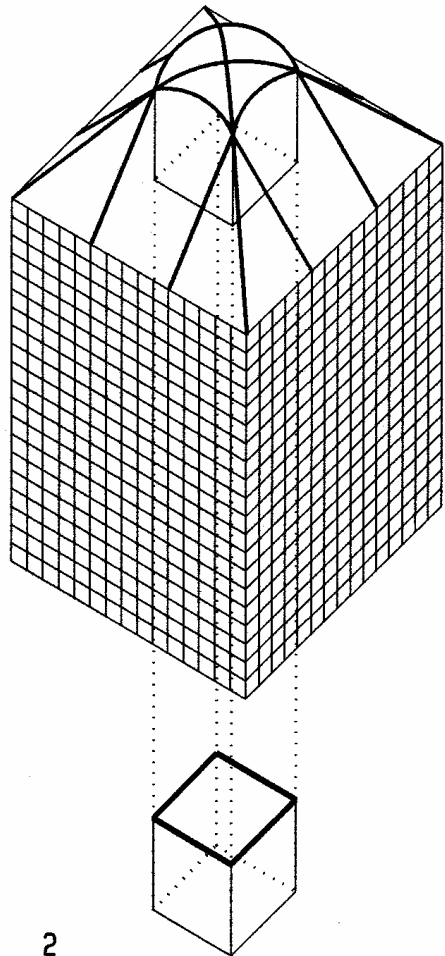
Guy stress

$$f = P / A = 1252 / 15.9$$

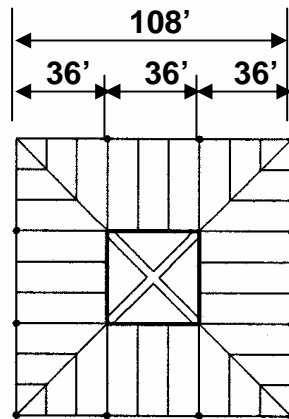
$$f = 79 \text{ ksi}$$



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Outrigger beam

Compression (from vector graph)

Try w36x230

Axial stress

$$f_a = P/A = 885/67.6$$

Bending stress

$$f_b = M/S = 275k \times 12'' / 837$$

beam radius of gyration

$$r = (I/A)^{1/2} = (15000 / 67.6)^{1/2}$$

Slenderness ratio (y-direction braced by floor)

$$KL/r = 36' \times 12'' / 14.9$$

Allowable buckling stress

Check combined stress $f_b/F_b + f_a/F_a \leq 1$

$$f_b/F_b + f_a/F_a = 3.9/22 + 13.1/20 = 0.83$$

$$P = 885 \text{ k}$$

$$A = 67.6$$

$$S = 837 \text{ in}^3$$

$$f_a = 13.1 \text{ ksi}$$

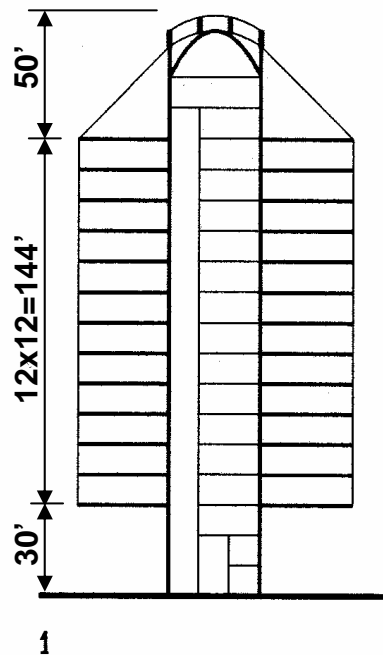
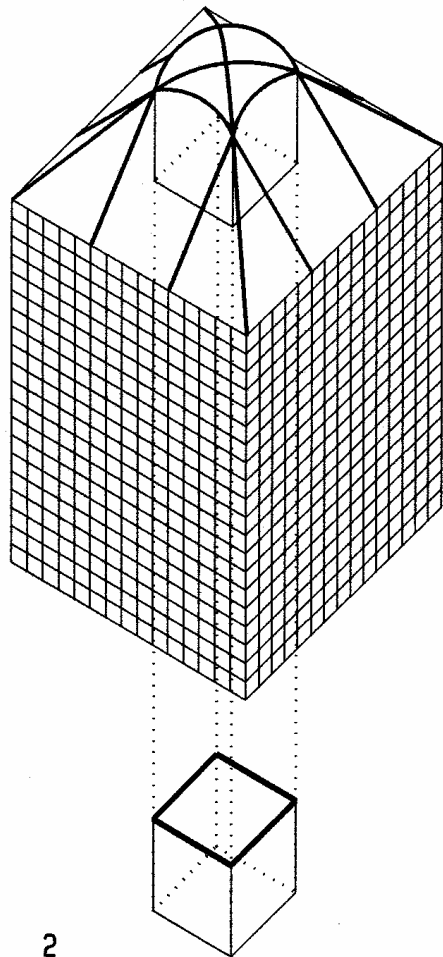
$$f_b = 3.9 \text{ ksi}$$

$$r = 14.9''$$

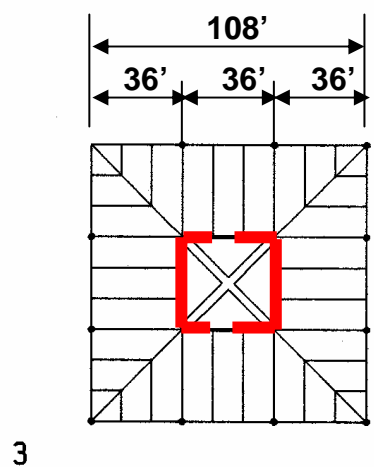
$$KL/r = 29$$

$$F_a = 20 \text{ ksi}$$

$$0.83 < 1$$



1



3

Overturn moment

$$M = 30 \text{ psf} [36(30+144+50)^2/2 + 2 \times 36 \times 144(30+72)] / 1000$$

$$M = 58,821 \text{ k'}$$

Core moment of Inertia I

($I_{\text{outside}} - I_{\text{inside}} - \text{two } 6' \text{ doors}$)

$$I = (B^4 - b^4) / 12 - Ay^2 = 36^4 - 34^4 / 12 - 2 \times 6 \times 18^2$$

$$I = 24,719 \text{ ft}^4$$

Bending stress

$$f_b = Mc/I = 58.821 \text{ k' } \times 18' / 24,719 \text{ ft}^4 = 42.83 \text{ ksf}$$

$$f_b = 42.83 \text{ ksf} \times 1000 / 144$$

$$f_b = 297 \text{ psi}$$

Dead load

$$P = 13 \times 100 \text{ psf} \times 108^2$$

$$P = 15,163,200 \text{ \#}$$

Dead load stress

$$f_c = P/A = 15,163,200 / [2(36+30)144]$$

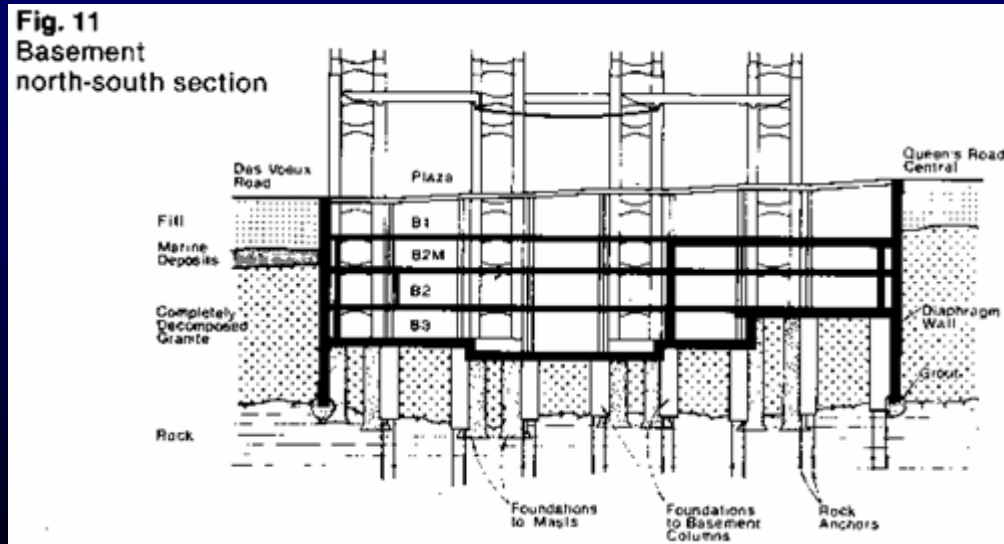
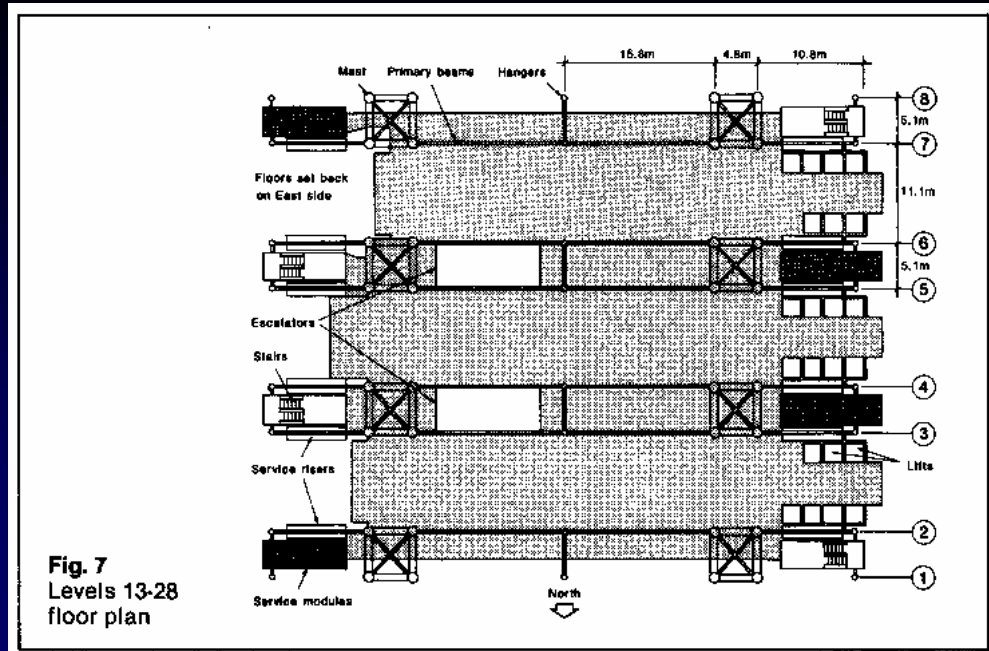
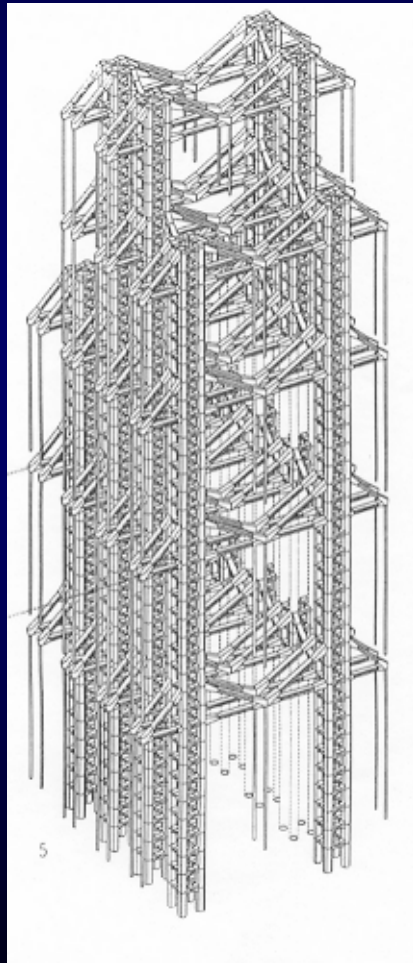
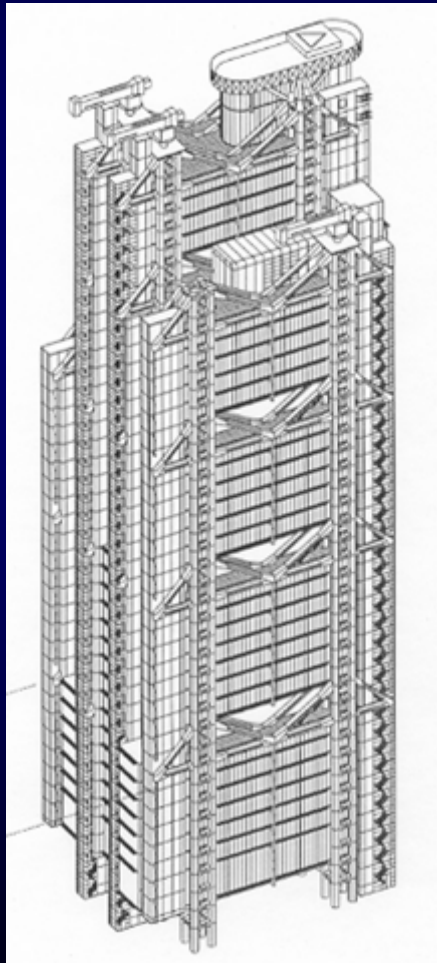
$$f_c = 798 \text{ psi}$$

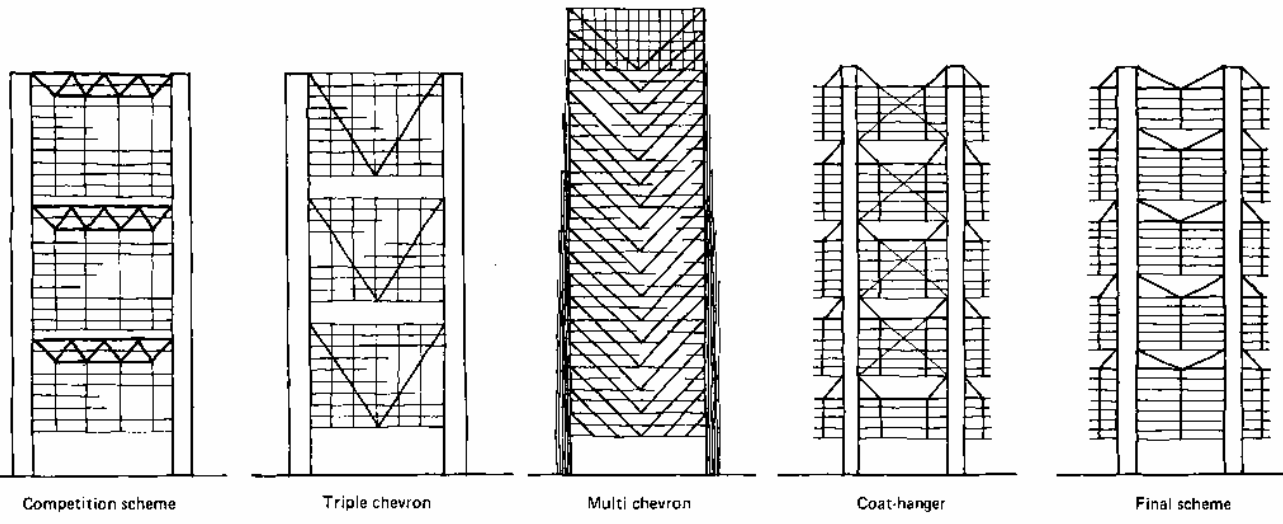
798 > 297, no tensile stress

Hong Kong Shanghai Bank

Architect: Norman Foster

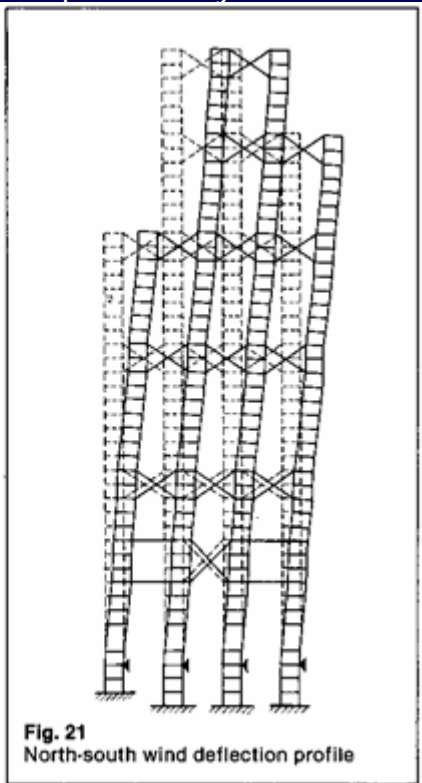
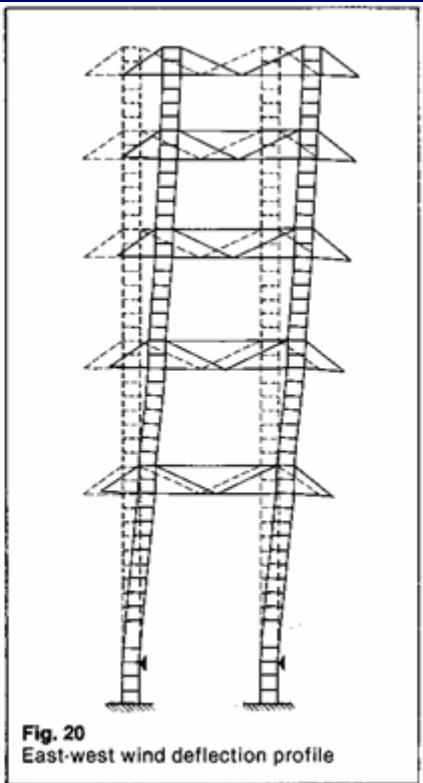
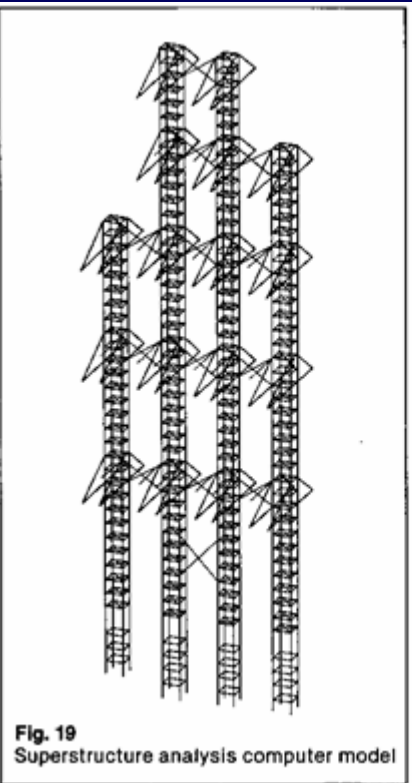
Engineer: Ove Arup



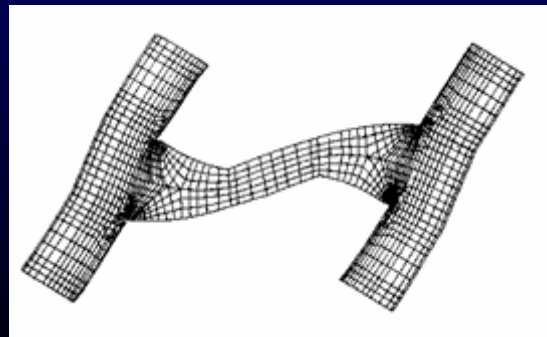


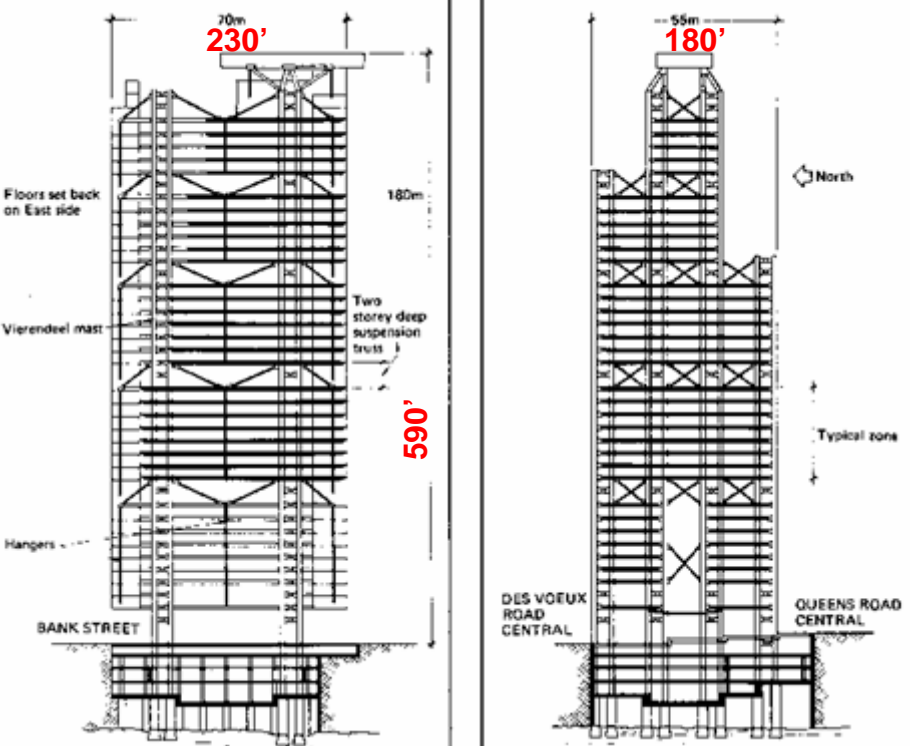
↑ Scheme development ↑

↓ Computer analysis ↓



Assume
 35 stories
 Max. 8 floors per stack
 Typical story height $h = 12.8'$
 Ground floor story height $h = 24'$
 Wind load $P = 3.8 \text{ kPa}$ $P = 80 \text{ psf}$
 HK statutory wind load varies from
 $P = 1.2 \text{ kPa}$ @ ground to
 $P = 4.3 \text{ kPa}$ @ 140 m)
 Gravity load
 DL = 90 psf
 LL = 63 psf (3kN/m²)
 $\Sigma = 153 \text{ psf}$
 Masts: 17'x16' (5.1x4.8m)
 4 pipes, max. $\phi 55$ "x3.9" thick
 (1400x100mm)
 Hangers: max. $\phi 16$ "x2.4" thick
 pipes (400x60mm)
 Finite Element analysis of mast





Base shear (per mast pipe, 8 pipes/bay)

$$V = 80\text{psf} \times 53' \times 590' / (8 \times 1000)$$

$$V = 313 \text{ k}$$

Pipe bending moment

$$M = V h/2 = 313 \times 12' \times 24' / 2$$

$$M = 45072 \text{ k''}$$

Section modulus ($S = \pi(D^4 - d^4) / 32D$)

$$S = \pi(55^4 - 47.2^4) / (32 \times 55)$$

$$S = 7474 \text{ in}^3$$

Bending stress

$$f_b = M/S = 45072 / 7474$$

$$f_b = 6.0 \text{ ksi}$$

Overturn moment (per bay)

$$M = 80\text{psf} \times 53' \times 590^2 / (2 \times 1000)$$

$$M = 73,797 \text{ k'}$$

Lateral load (per pipe, 4 pipes/mast)

$$P = M / (4B) = 73797 / (4 \times 126')$$

$$P = 146 \text{ k}$$

Gravity load (per mast pipe, 4 pipes/mast)

$$P = 35 \times 153 \text{ psf} \times 105' \times 53' / (4 \times 1000)$$

$$P = 7450 \text{ k}$$

Combined axial load

$$\Sigma P = 7450 + 146$$

$$\Sigma P = 7596 \text{ k}$$

Pipe cross section area

$$A = \pi(D^2 - d^2) / 4 = \pi(55^2 - 47.2^2) / 4$$

$$A = 626 \text{ in}^2$$

Pipe axial stress

$$f_a = P/A = 7596 / 626$$

$$f_a = 12.1 \text{ ksi}$$

Tributary hanger area

$$A = 55' \times 27'$$

$$A = 1485 \text{ ft}^2$$

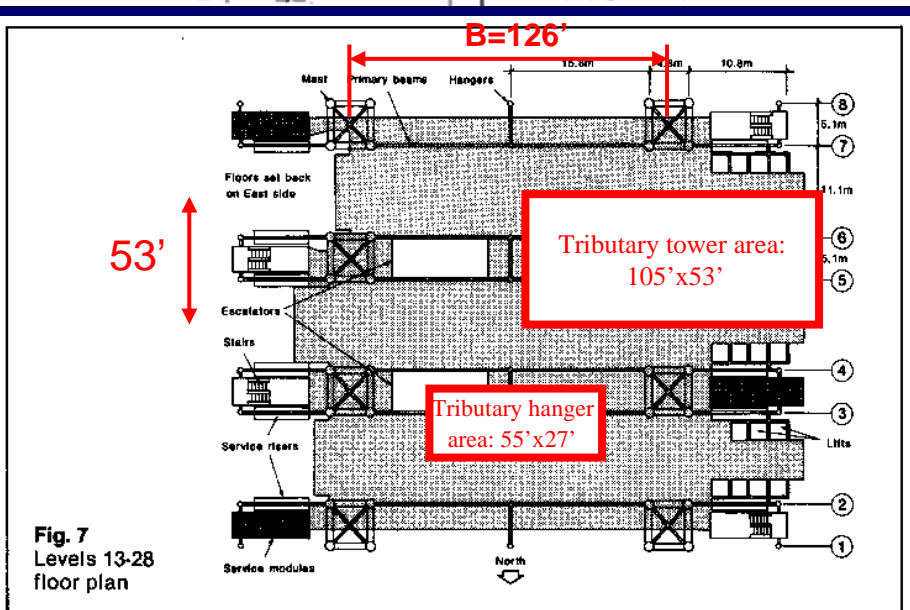
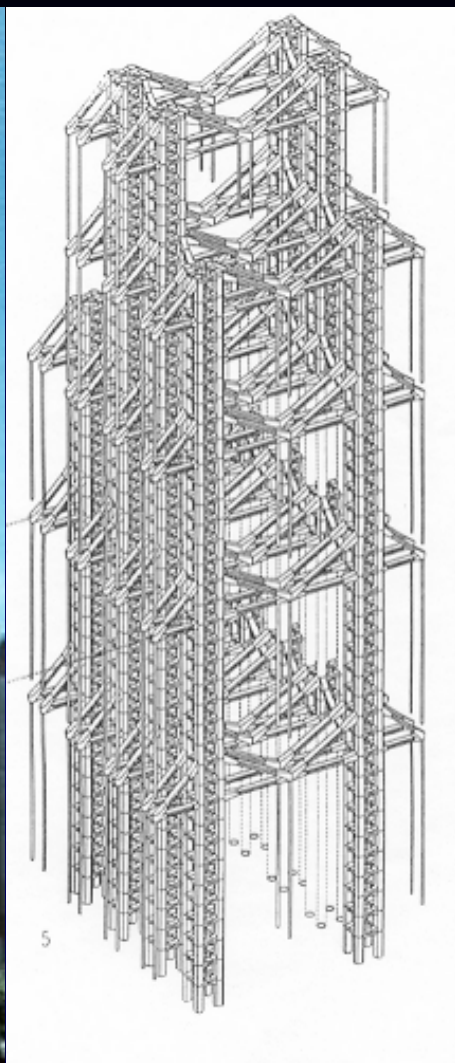


Fig. 7
Levels 13-28
floor plan



Pipe radius of gyration

$$r = (D^2+d^2)^{1/2}/4 = (55^2+47.2^2)^{1/2}/4$$

$$r = 18''$$

Pipe slenderness ratio (KL=1.2x24 = 29')

$$KL/r = 29' \times 12''/18''$$

$$KL/r = 19$$

Allowable buckling stress (from AISC table)

$$F_a = 20.7 \text{ ksi}$$

Check combined stress ($f_a/F_a + f_b/F_b \leq 1$)

$$12.1/20.7 + 6.0/22 = 0.86$$

$$0.86 < 1, \text{ ok}$$

Max. hanger load (8 floors)

$$P = 8 \times 153 \text{ psf} \times 1485 \text{ ft}^2 / 1000$$

$$P = 1818 \text{ k}$$

Hanger cross section ($A = \pi(D^2-d^2)/4$)

$$A = \pi(16^2 - 11.2^2)/4$$

$$A = 103 \text{ in}^2$$

Hanger stress

$$f_a = P/A = 1818/103$$

$$f_a = 17.7 \text{ ksi}$$

Hanger length per stack (8 stories)

$$L = 8 \times 12.8' \times 12''$$

$$L = 1229''$$

Hanger elongation ($\Delta = PL/EA = f L/E$)

$$\Delta L = 17.1 \text{ ksi} \times 1229'' / 30,000$$

$$\Delta L = 0.7''$$

Mast shortening

$$\Delta L = 12.1 \text{ ksi} \times 1229'' / 30,000$$

$$\Delta L = 0.5''$$

Differential deflection

$$\Delta L = 0.7 + 0.5$$

$$\Delta L = 1.2''$$

Note: adjust hanger for DL deflection

From last page:

Tributary hanger area

$$A = 1485 \text{ ft}^2$$

Pipe bending stress

$$f_b = 6.0 \text{ ksi}$$

Pipe axial stress

$$f_a = 12.1 \text{ ksi}$$

