**Suspended high-rise**

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

**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 Schwanzer

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 those 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

UN Center Vienna
built project
Architect: J Staber
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

Concrete core wall thickness \( t = 1' \)
Suspennder cables \( 2 \phi 2 7/8'' \)
Guy cables \( 2 \phi 2 7/8'' + 2 \phi 2 1/2'' \)
Average wind pressure (80mph, Exposure B) \( P = 30 \text{ psf} \)

Live load reductions
Beam: \( R = 50 \% \)
Suspender: \( R = 60 \% \)

Gravity loads
Concrete slab \( = 60 \text{ psf} \)
Partitions \( = 20 \text{ psf} \)
Framing \( = 15 \text{ psf} \)
Floor/ceiling \( = 5 \text{ psf} \)
DL \( = 100 \text{ psf} \)

Beam live load
\( 0.5 \text{ (50)} \) LL = 25 psf

Suspender live load
\( 0.4 \text{ (50)} \) LL = 20 psf

Total loads:
Beam \( = 125 \text{ psf} \)
Suspender \( = 120 \text{ psf} \)
Uniform beam load
\[ w = 125 \text{ psf} \times 12'/1000 \]
\[ w = 1.5 \text{ klf} \]

Beam bending
\[ M = wL^2/8 = 1.5 \times 36^2/8 \]
\[ M = 243 \text{ k' } \]
\[ S = M/F_b = 243 \times 12/22 \]
\[ S = 133 \text{ in}^3 \]

Use W21\times73
\[ S = 151 > 133 \]

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
\[ f = P/A = 1252/15.9 \]
\[ f = 79 \text{ ksi} \]
Outrigger beam

Compression (from vector graph)

Try w36x230

Axial stress

\[ f_a = \frac{P}{A} = \frac{885}{67.6} \]

\[ f_a = 13.1 \text{ ksi} \]

Bending stress

\[ f_b = \frac{M}{S} = \frac{243k'x12''}{837} \]

\[ f_b = 3.48 \text{ ksi} \]

Beam radius of gyration

\[ r = \left( \frac{I}{A} \right)^{1/2} = \left( \frac{15000}{67.6} \right)^{1/2} \]

\[ r = 14.9'' \]

Slenderness ratio (y-direction braced by floor)

\[ kL/r = \frac{36'x12''}{14.9} \]

\[ kL/r = 29 \]

Allowable buckling stress

\[ F_a = 20 \text{ ksi} \]

Check combined stress

\[ \frac{f_b}{F_b} + \frac{f_a}{F_a} \leq 1 \]

\[ \frac{3.48}{22} + \frac{13.1}{20} = 0.81 \]

\[ 0.81 < 1 \]
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<th>$F_y$ (ksi)</th>
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<td>15.69</td>
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$F_y = 36$ ksi

Table C-50

For Compression Members of 50-ksi Specified Yield Stress Steel

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<th>$K_I$</th>
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</tbody>
</table>

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Overtum moment (30 psf wind pressure)
\[ M = 30 \left[ \frac{36(30+144+50)^2}{2} + 2 \times 36 \times 144(30+72) \right] / 1000 = 58,821 \text{ k }' \]

Core moment of Inertia \( I \)
\[ I = (B^4-b^4)/12 - A y^2 = 36^4 - 34^4) / 12 - 2 \times 6 \times 18^2 \]
\[ I = 24.719 \text{ ft}^4 \]

Bending stress \( f_b \)
\[ f_b = \frac{M c / I}{18} = \frac{58.821 \text{ k}' \times 18}{24,719 \text{ ft}^4} = 42.83 \text{ ksf} \]
\[ f_b = 42.83 \text{ ksf} \times 1000 / 144 \]
\[ f_b = 297 \text{ psi} \]

Dead load (13 stories @ 100 psf)
\[ P = 13 \times 100 \text{ psf} \times 108^2 \]
\[ P = 15,163,200 \# \]
\[ f_c = \frac{P}{A} = \frac{15,163,200}{[2(36+30)144]} \]
\[ f_c = 798 \text{ psi} > 297 \]
Overbeek House
Rotterdam ~ 90’x90’ - 11 stories
Architect: Verbruggen & Goldsmidt
Engineer: Aronsohn
Hong Kong Shanghai Bank
Architect: Norman Foster
Engineer: Ove Arup

Fig. 7
Levels 13-28 floor plan

Fig. 11
Basement north-south section
Assume
35 stories
Max. 8 floors per stack
Typical story height $h = 12.8'$
Ground floor story height $h = 24'$
Wind load $P = 3.8$ kPa $P = 80$ psf
HK statutory wind load varies from $P = 1.2$ kPa @ ground to $P = 4.3$ kPa @ 140 m)
Gravity load
$DL = 90$ psf
$LL = 63$ psf (3kN/m$^2$)
$\Sigma = 153$ 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

\[ M = V \frac{h}{2} = 313 \times 12' \times 24' / 2 \]

\[ M = 45072 \text{k}'' \]

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

\[ f_b = \frac{M}{S} = \frac{45072}{7474} \]

\[ f_b = 6.0 \text{ksi} \]

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)

\[ P = \frac{M}{4B} = \frac{73797}{4 \times 126'} \]

\[ P = 146 \text{k} \]

Combined axial load

\[ \sum P = 7450 + 146 \]

\[ \sum P = 7596 \text{k} \]

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

\[ f_a = \frac{P}{A} = 7596/626 \]

\[ f_a = 12.1 \text{ksi} \]

Tributary hanger area

\[ A = 55' \times 27' \]

\[ A = 1485 \text{ft}^2 \]
F_y = 36 ksi
Pipe radius of gyration
\[ r = \left( \frac{D^2 + d^2}{4} \right)^{1/2} = \left( \frac{55^2 + 47.2^2}{4} \right)^{1/2} \]
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 \( \left( \frac{f_a}{F_a} + \frac{f_b}{F_b} \right) \leq 1 \)
\[ \frac{12.1}{20.7} + \frac{6.0}{22} = 0.86 \]
0.86 < 1, ok

Max. hanger load (8 floors)
\[ P = 8 \times 153 \text{ psf} \times 1485 \text{ ft}^2/1000 \]
P = 1818k

Hanger cross section \( A = \pi \left( \frac{D^2 - d^2}{4} \right) \)
\[ A = \pi \left( 16^2 - 11.2^2 \right)/4 \]
A = 103 in²

Hanger stress
\[ f_a = \frac{P}{A} = \frac{1818}{103} \]
f_a = 17.7 ksi

Hanger length per stack (8 stories)
\[ L = 8 \times 12.8' \times 12” \]
L = 1229’

Hanger elongation \( \Delta L = \frac{PL}{EA} = \frac{f L}{E} \)
\[ \Delta L = 17.1 \text{ ksi} \times 1229” / 30,000 \]
\[ \Delta L = 0.7” \]

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
\[ f_a = 12.1 \text{ ksi} \]
suspend