181 Fremont

San Francisco, CA

Tech Report 3

10/17/2014


PSUAE

Structural Option

Advisor: Dr. Thomas Boothby

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Dr. Thomas Boothby
The Pennsylvania State University
209 Engineering Unit A
University Park, PA 16802

Dear Dr. Boothby:

Enclosed is Technical Report 3, a technical report analyzing the existing gravity system of 181 Fremont as well as three alternative gravity systems. This report evaluates the performance of the existing system by assessing the framing under design loads. It also illustrates the alternative framing options and the corresponding strength and deflection calculations.

Included in this report is an abstract describing primary building systems, a list of building codes and specifications used, and calculations determining the performance of the existing and three alternative systems: concrete framing, post-tensioned slabs, and composite beams with lightweight concrete deck.

Thank you for taking the time to review this report.

Sincerely,

Caroline Klatman

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Executive Summary

181 Fremont is a 54 story high-rise in the South of Market neighborhood in San Francisco, California. Its construction is a part of the San Francisco Transit Center District Plan – a redevelopment plan that allows for greater building heights within that area of the city. As such, the building rises to 700 feet, the maximum height allowed per the limitations on the site.

In response to the high seismic loading brought about by the site location, the structure expresses a unique and complicated design solution. A mega-frame system, expressed on the exterior of the building, acts as the primary lateral system of the structure into which all other lateral forces are carried.

Buckling restrained brace frames in the interior of upper stories of the structure and moment frames at the lower story exteriors supplement the mega-frame in providing lateral-force-resistance. Other contributors to the lateral system include collectors at each floor and viscous dampers in the exterior braces of the structure.

Because the mega-frame system is not defined in ASEC 7-05, an in depth seismic analysis was completed that conforms to the San Francisco Department of Building Inspection Administrative Bulletin on the Seismic Design & Review of Tall Buildings Using Non-Prescriptive Procedures (SF AB-083, 2010) and the PEER Guidelines for Performance-based Seismic Design of Tall Buildings (PEER TBI, 2010).

181 Fremont

San Francisco, California

General Information

Dates of Construction | Nov 2013 - 2016
Project Delivery Method | Design-Bid-Build
Occupancy| Mixed-use Office and Residential
Cost | $375 Million
Number of Stories | 54 Stories
Height | 700 ft.
Size | 411,000 sq. ft.

Project Team

General Contractor | Level 10 Construction
Construction Manager | Jay Paul Company
Owner | Jay Paul Company
Architect | Heller Manus
Structural Engineer | Arup
MEP Engineer | Arup

Structural Systems

The structure rests atop a mat foundation, below which roughly 60 piles extend 150 feet down to reach bedrock. Various systems such as viscous dampers and steel moment frames provide lateral force resistance, but the primary lateral force resisting system is an exterior steel mega-frame.

Sustainability

In pursuit for LEED Platinum, multiple steps toward sustainability including a curtain wall system that favors natural lighting, a green roof, grey water system, and use of recycled materials are featured.

Architecture

The architectural design features transparency in the structural system by exposing the exterior steel mega-frame, which extends beyond the roofline. A curtain wall system with angular glass units and walls that taper in as the building rises also add to the building's exterior aesthetic expression.

Various amenities are provided for residents, including a two-story open air terrace that wraps around the 36th floor. Also features is a pedestrian bridge on the 5th floor that allows residents to access the Transit Tower's rooftop City Park, as shown in the photos at left and below.

Mechanical Systems

181 Fremont's mechanical system is comprised of a forced-air ventilation system, with air intake and filtration occurring on the mechanical floor on level 37. Air is then transferred to each individual residential unit, where it is again filtered and either heated or cooled by a fan coil unit.

CAROLINE KLATMAN

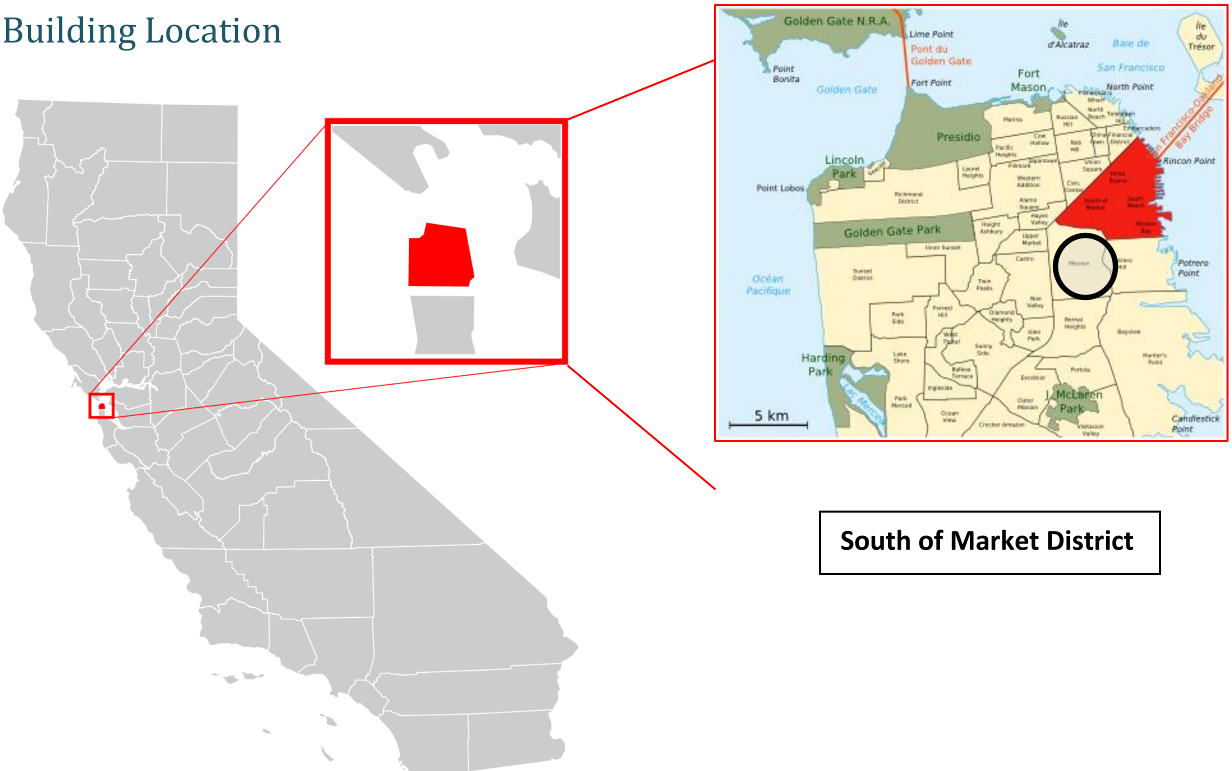
STRUCTURAL OPTION

ADVISOR | DR. THOMAS BOOTHBY

http://www.engr.psu.edu/ae/thesis/portfolios/2015/cjk5258/index.html

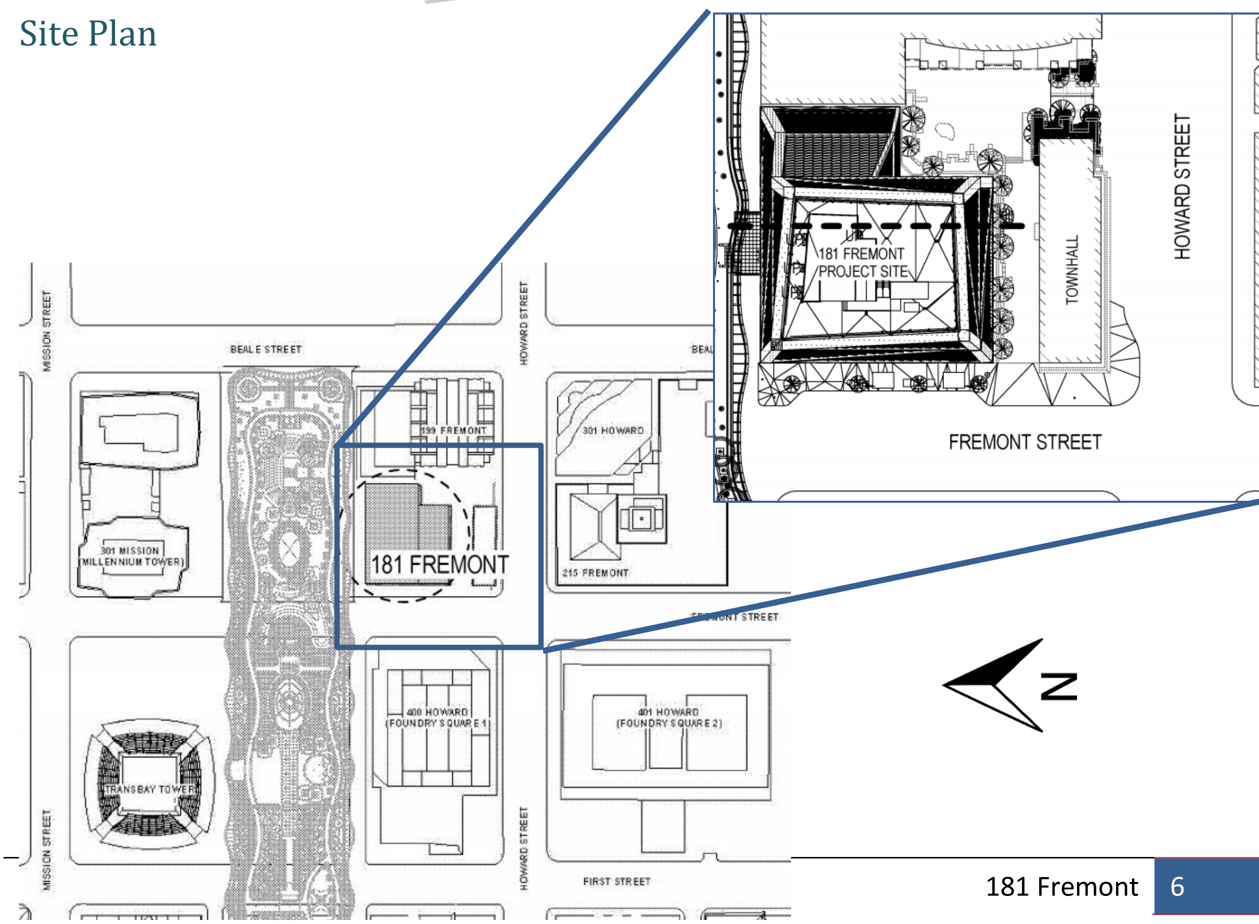
Building Location and Site Plan

Building Location



South of Market District

Site Plan



Documents Used in Preparation of This Report

2010 California Building Code

- ASCE 7-05

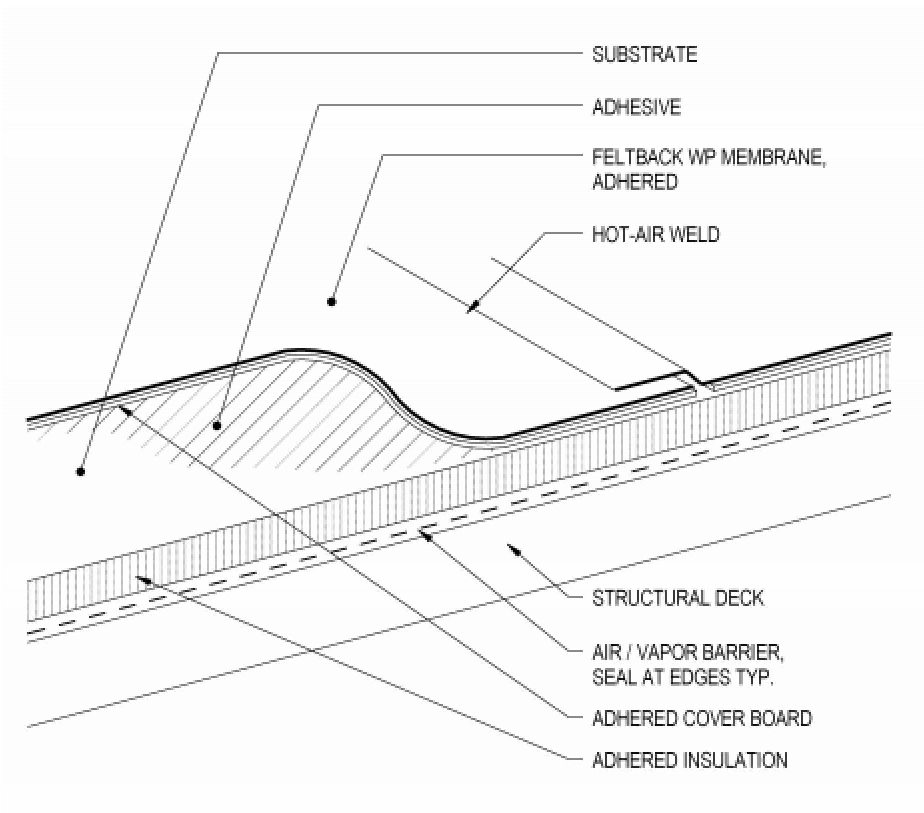
2010 San Francisco Building Code

Other Documents

- AISC Manual of Steel Construction
- ACI 318-11
- RS Means Online

Gravity Load Calculations

Typical Roof Bay Cross Section



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Typical Roof Bay

Dead Load:

6" NW conc. on 2" metal deck	- 91 psf	→ use for entire roof to be conservative
Membrane + Air/Vapor barrier + cover board	- 5 psf	
Rigid Insulation	- 1.5 psf (4") = 6 psf	
Steel Framing	- 10 psf	
MEP	- 3 psf	
Ceilings	- 5 psf	
Sprinklers	- 3 psf	
	123 psf	

This is higher than the 107 psf load on S-019

Live Load:

20 psf per ASCE 7-05 table 4-1

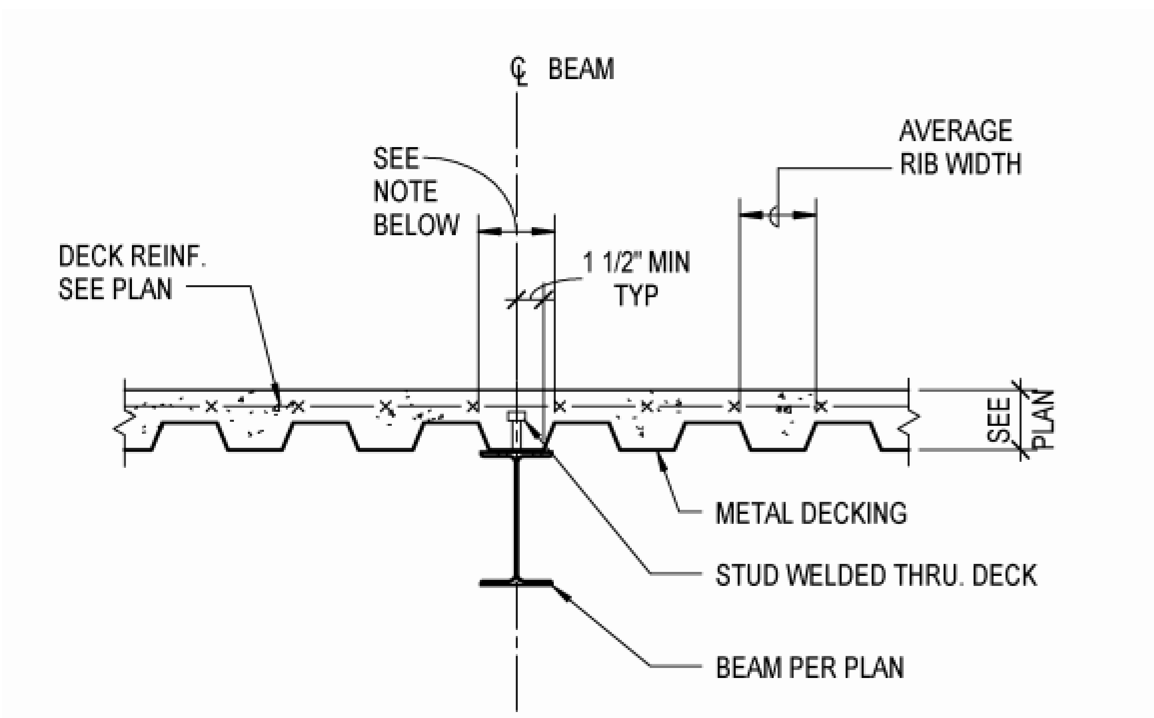
Snow Load: (ASCE 7-05)

$p_f = 0.7 C_e C_t I p_g$ (eqn 7-1)

$p_g = 0$ (Fig 7-1)

$\therefore p_f = 0$

Typical Floor Cross Section, Deck Parallel to Beam



Gravity Loads

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Typical Residential Floor

Dead:

3¼" NW conc. on 2" Epicore Metal Deck	-	65 psf
MEP	-	15 psf
Ceilings	-	5 psf
Sprinklers	-	3 psf
Additional concrete	-	5 psf
Steel Framing	-	10 psf
		103 psf

103 psf = typical residential load on S-019, but different allowances used

Live:

(ASCE 7-05) (Table 4-1)

Residential, Private Rooms and areas serving them	-	40 psf
Partitions	-	15 psf
		55 psf

Typical Office Floor

Dead:

3¼" LWC on 2" metal deck	-	44 psf
MEP	-	15 psf
Ceilings	-	5 psf
sprinklers	-	3 psf
additional concrete	-	5 psf
steel framing	-	10 psf
		82 psf

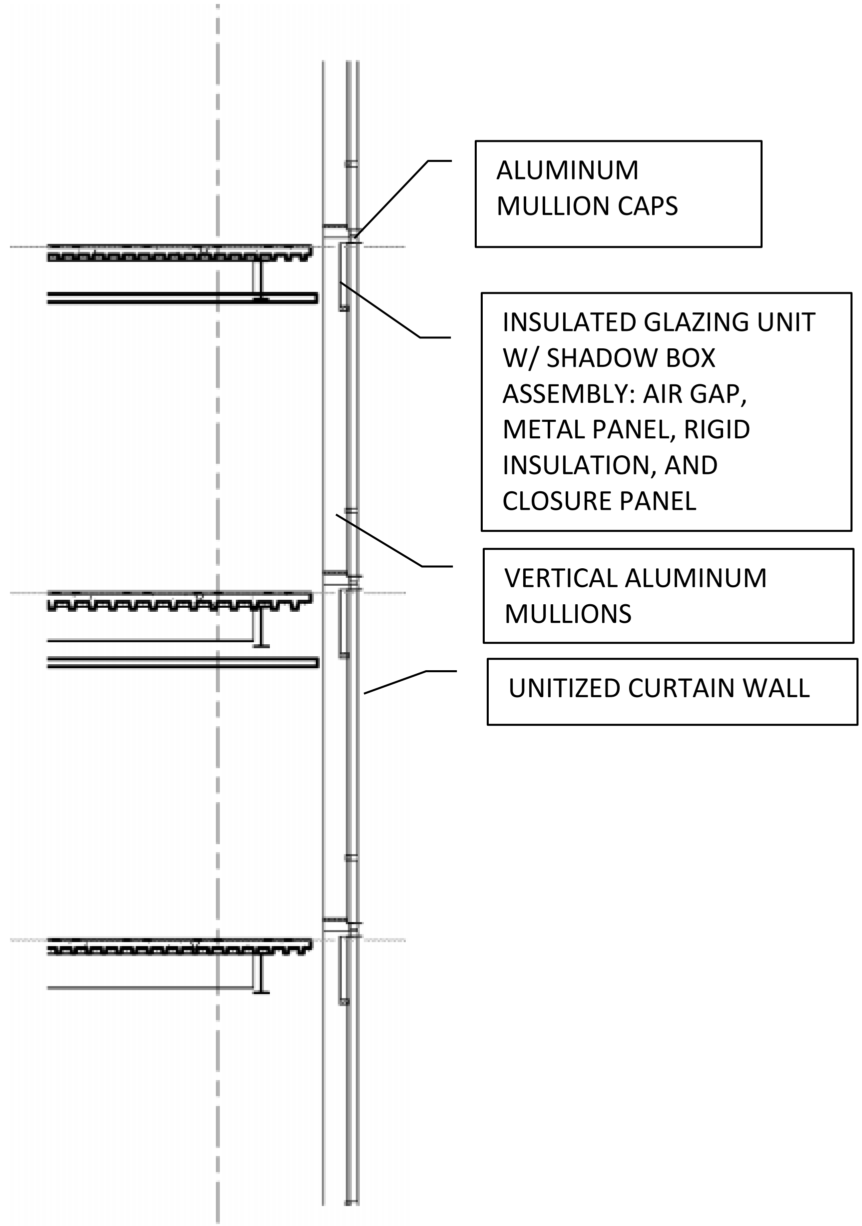
82 psf > typical office load of 60 psf on S-019

Live:

(ASCE 7-05)

Offices	-	50 psf
Partitions	-	15 psf
		65 psf = design load on S-019
corridors	-	80 psf = design load on S-019

Typical Exterior Wall Detail Cross-Section



Gravity Loads

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Typical Exterior Wall

Dead Load:

Curtain Wall System - 13 psf

Load Path:

The curtain wall anchors into the concrete slabs at each level using angles embedded in the slab edges. Through these connections, the lateral loads experienced by the curtain wall and the wall's self-weight are transferred to the structure's diaphragm.

Non-typical Dead Loads:

Roof Mechanical Equipment - actual weight (2 chillers, waiting on size)

Mechanical Floors - 100 psf (deck) + typical allowances
Levels 2, 38 + 25 psf allowance for concrete curbs and housekeeping

Retail Space - extra 16 psf for ceramic floors
Level 5

Lobby - 150 psf (12" slab) + typical allowances = 163 psf

Non-Typical Live Loads:

Roof Mechanical Room - 150 psf (ASCE Armories & Drill Rooms)

Mechanical Floors - 125 psf (ASCE light manufacturing)
Level 2, 38

Storage - 125 psf (ASCE light storage)
Level 2

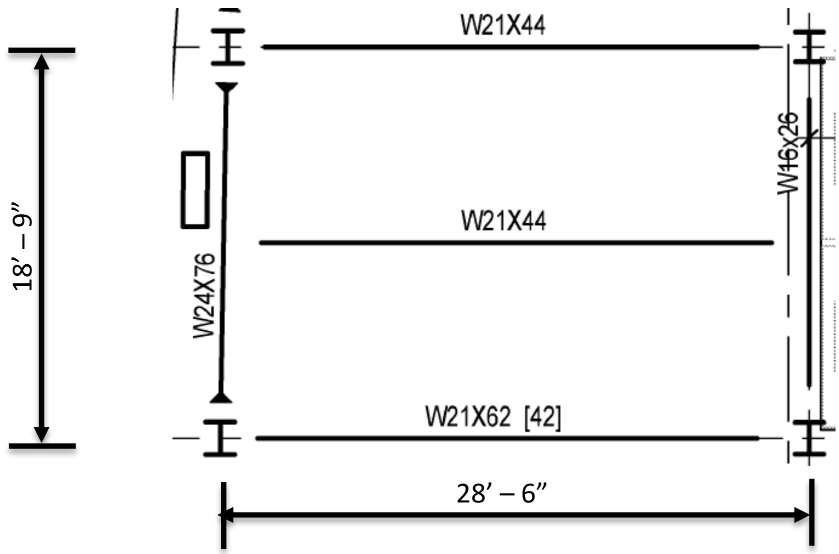
Retail - 100 psf (ASCE Store retail)
Level 5

Lobby - 100 psf

Existing System

Typical Floor Plan

Typical Bay



Bay Information:

- 5 ¼" slab on deck
  - Lightweight concrete
  - 2", 18 gage metal deck
  - #5 bars @ 12" slab reinforcing
  - f'c = 4000 psi
  - Deck spans = 9'-4 ½"
- Superimposed dead load = 60 psf (as designed)
- Live load = 65 psf (as designed)
- Curtain wall = (13 psf)*(12.5' tributary height) = 163 plf along the W24x76
  - Value designed for = 175 plf

Framing Checks

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Verco Formlok W2 Steel Deck (LW conc.):

DL = 60 psf

* Assume 3 span condition

- Span = 9' 6" (≥ 9' 4½" actual span)

$w_u = 1.2(60) + 1.6(65) = 176$ psf

Allowable superimposed loads = 346 psf > 176 psf ✓
(in addition to deck weight)

- no shoring required
- deflection considered in Verco tables

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER

COMET

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W21 × 44 :

$DL = 60\text{ psf} + \frac{44\text{ plf}}{9.375} = 65\text{ psf}$

$LL = 65\text{ psf}$ (reduced)

$w_u = 1.2(65) + 1.6(65) = 182\text{ psf} \rightarrow 182(9.375') = 1.706\text{ klf}$

$M_u = \frac{wl^2}{8} = \frac{1.706(28.5)^2}{8} = 173.21\text{ k-ft}$

$\phi M_n = 358^{k\text{-}ft} > 173.2 \therefore$ ok bending (AISC Steel Manual, Table 3-2, LRFD)

Live Load Deflection

$\Delta_{LLmax} = L/360 = 28.5(12''/1')/360 = 0.95''$

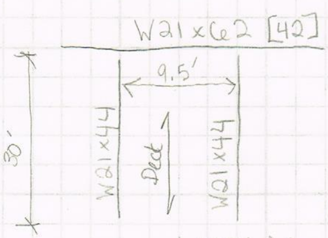
$\Delta_{LL} = \frac{5wL^4}{384EI}$ $w = \frac{1}{2}(80\text{ psf})(9.375) = 375\text{ plf}$ (unreduced LL)

$E = 29{,}000$ $I_x = 843\text{ in}^4$

$\Delta_{LLred} = \frac{5(0.375)(28.5)^4}{384(29{,}000)(843)}(1728) = 0.228'' \ll 0.95''$

$\Delta_{LL} = \frac{5(0.75)(28.5)^4}{384(29{,}000)(843)} = 0.46'' < 0.95''$ $\therefore$ serviceability deflection ok at AISC DG 3 reduced and unreduced LL

W21 × 62 [42] :



$DL = 60\text{ psf} + \frac{1}{2}\left(\frac{44\text{ plf}}{9.375}\right) + \frac{62\text{ plf}}{9.375} = 69\text{ psf}$

$LL = 65\text{ psf}$

$w_u = [1.2(69) + 1.6(65)]\left(\frac{9.375}{2}\right) = 0.876\text{ klf}$

Point loads $w_u = 2(182\text{ psf})(9.5')(15') = 52^k$

$M_{u,max} = \frac{0.876(28.5^2)}{8} + 52(9.5) = 583'^k$

$b_{eff} = 2 \min \begin{cases} 28.5(12)/8 = 42.75'' \\ \frac{1}{2}(9.375)(12) = 56.25'' \end{cases} \Rightarrow 85.5''$

Studs: (3/4" Ø) Strong Position, $R_p = 0.75$

2 per rib

Deck perpendicular

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$$Q_n = \min \begin{cases} 18.3^k \text{ (AISC table 3-21)} \\ 0.5A_{sc}\sqrt{f'_c E_c} = 0.5\pi(\tfrac{3}{8}^2)\sqrt{4{,}000(33)(110^{1.5})\sqrt{4000}} = 21.7^k \end{cases}$$

$\Rightarrow Q_n = 18.3^k$

assume $a=1$, $y_2 = 5.25 - \frac{1.0}{2} = 4.75$

Table 3-19 (AISC): $\Sigma Q_n = 229^k$, $\phi M_n = 752'^k > 583'^k$ ∴ ok flexure

$229/18.3 = 12.5 \rightarrow 26$ studs needed (46 studs provided)

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{229}{0.85(4)(85.5)} = 0.788 < 1.0 \therefore \text{ok}$$

deflection:

$\Delta_{LL,max} = 0.95''$ (see p.17) $I_{LB} = 2180$

$\Delta_{LL,red} = 0.487''$ under 65 psf LL

(from Risa 2-D output, see appendix A)

W24×76:

$DL_{point} = (4.7\text{ psf}_{(W21\times44s)} + 60\text{ psf})(28.5/2)(9.375) = 8.644^k$, $DL_{dist} = 76 + 175_{\text{curt wall}} = 0.251$ klf

$LL = (65\text{ psf})(28.5/2) = 0.926$ klf

$w_{u,dist} = 1.2(0.251) + 1.6(0.926) = 1.78$ klf

$w_{u,point} = 1.2(8.644) = 10.373^k$

$$M_{u,max} = \frac{1.78(18.75^2)}{8} + \frac{10.373(18.75)}{4} = 127'^k$$

$462'^k > 127'^k$ ∴ ok flexure

$$\Delta_{LL,max} = \min \begin{cases} 18.75/240 = 0.94'' \\ 1'' \end{cases} \Rightarrow 0.94''$$

$\Delta_{LL,tot} << \Delta_{LL,max}$ (see Risa output in appendix A)

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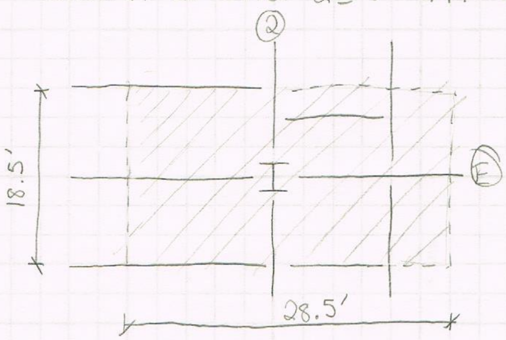
Interior Column Evaluation

Column at Grid 2E : W14 × 398 at level 13



$A_T = 18.5(28.5') = 527 \text{ ft}^2$

unbraced length = 12.5'

Loads:

	DL	LL	Floors
Roof:	$\frac{1}{2}(107)$	$\frac{1}{2}(20)$	1
Residential:	103	40	17
Mechanical:	141	125	3
Office:	60	65	25

Note: $\frac{1}{2}$ used to account for half the trib width

$P_u = 1.2[(\frac{1}{2})(107) + 103(17) + 141(3) + 60(25)]$
$+ 0.5(\frac{1}{2})(20) + 1.6[40(17) + 125(3) + 65(25)] = 8,766 \text{ psf}$

$= 8,766(527)$

$= 4,620^k$

At conservative unbraced length of 13',

$\phi P_n = 4,780^k > 4,620^k \therefore$ ok axial compression

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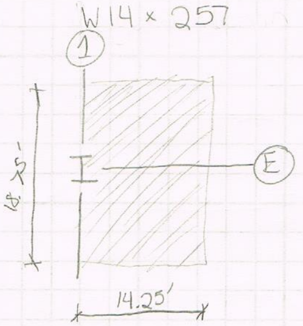
Exterior Column Evaluation

W14 × 257



$A_T = 18.75\,(28.5/2) = 267\ ft^2$

Loads:	DL	LL	Floors	Facade DL
Roof:	107	20	1	15 psf
Residential:	103	40	17	
Mechanical:	141	125	2	
Office:	60	65	25	

$P_u = 1.2(267)[107 + 103(17) + 141(2) + 60(25)]$

$+ 1.6(267)[40(17) + 125(2) + 65(25)] + 0.5(267)(20)$

$= 2{,}262^k + 1.2(18.75')(15\,psf)(735.5' - 171.25')$

$= 2{,}452^k$

At unbraced length of 13',

$\phi P_n = 3060^k > 2{,}452^k$ ∴ ok axial compression

Alternative System 1: Concrete Framing

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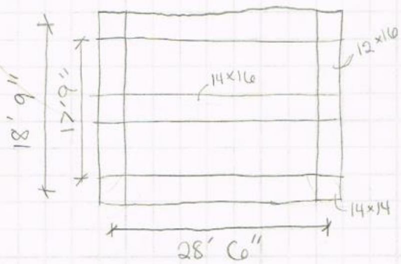
One way slab:

ACI Table 9.5a min thickness, $h = \ell/28 = \frac{18.75(12''/1')}{28} = 8.04'' \rightarrow 8.5''$



$\omega = 8.5''(150\,pcf)(1'/12'') = 106\,psf$

DL = 11 psf (existing superimposed design value)

LL = 65 psf

Assume #5 bars

$d = 8.5 - 0.75 - 0.625/2 = 7.44''$

$\omega_u = 1.2(11+106) + 1.6(65) = 244\,psf\,(1') = 244\,plf$

$M_u = \frac{0.244(17.75')^2}{8} = 9.61\,k\text{-}ft$

$\frac{9.61}{4(7.44)} = 0.323\,in^2/ft \rightarrow$ use #6 @ 12" o.c. $(0.44\,in^2)$ longitudinal reinforcing

$A_{s,min} \geq \max \begin{cases} 3\sqrt{f'_c}\,bd/f_y \\ 200bd/f_y = 200(12'')(7.375)/60{,}000 = 0.295\,in^2/ft \end{cases}$

∴ use #5 @ 12" o.c. transverse reinforcing

$d_{act} = 8.5 - 0.75 - \frac{0.75}{2} = 7.375''$

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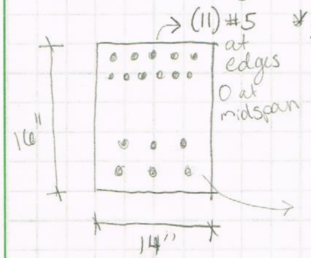
Beam Design:



*See Appendix B for SP Beam Output

DL = 11 psf + 106 psf slab wght

LL = 65 psf

$h_{min} = 8.5''$ unless deflection is calculated (table 9.5a)

$w_u = 9.375\,[1.2(117) + 1.6(65)] = 2.29$ klf

ACI Moment Coefficients:

interior edge $M_u = \frac{1}{9}(w_u l_n^2)$ *assume 14 × 14 columns

$= \frac{1}{9}(2.29)(27.167^2) = 188$ k-ft

midspan $M_u = \frac{1}{14}(2.29)(27.167^2) = 121$ k-ft

M_u's less than ϕM_n ✓

Midspan hand check: $d = 16 - 0.75 - 0.75 - 0.5 = 14''$

$A_{s,min}$

$$A_{s,min} = \max \begin{cases} \dfrac{3\sqrt{f'_c}\,bd}{f_y} \\ \dfrac{200\,bd}{f_y} \end{cases} = \frac{200(14)(14)}{60{,}000} = 0.653\ in^2 < A_{s,prov} = 2.2\ in^2$$

$A_{s,max}$

$$= 0.85\beta_1 \frac{f'_c}{f_y}\left(\frac{\varepsilon_{cu}}{\varepsilon_{cu}\ \varepsilon_y}\right)bd = 0.85(0.85)\left(\frac{4}{60}\right)\left(\frac{0.003}{0.007}\right)(14)(14)$$

$= 4.05\ in^2 > 2.2\ in^2$ ✓

ϕM_n

$= \phi A_s f_y \left(d - \frac{a}{2}\right)$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(2.2)(60)}{0.85(4)(14)} = 2.773$

$\phi M_n = 0.9(2.2)(60)\left(14 - \frac{2.773}{2}\right) = 1499''^k = 125'^k > 121'^k$ ✓

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3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Shear Check:

$V_u = 36.29^k$

$V_c = 2\lambda\sqrt{f'_c}\,bd = 2(1.0)\sqrt{4000}\,(14)(14) = 24.78^k < 36.29 \rightarrow$ stirrups required

$V_s = \frac{A_v f_{yt} d}{s} = \frac{2(0.11)(60)(14)}{6.4} = 28.9^k$

$\phi V_n = 0.75(24.78 + 28.9) = 40.28^k > 36.29^k$ ✓ ∴ ok shear

Shear Spacing Check:

$4\sqrt{f'_c}\,bd = 4\sqrt{4000}\,(14)(14) = 49.6^k > V_u$

Max spacing

$= \min \begin{cases} d/2 = 7'' \\ 24'' \end{cases}$

$6.4'' < 7''$ ∴ ok spacing

Reinforcing:

Min Reinforcing, $A_{v,min}$

$= \max \begin{cases} \frac{0.75\sqrt{f'_c}\,b_w s}{f_{yt}} \\ \frac{50 b_w s}{f_{yt}} = \frac{50(14)(6.4)}{60{,}000} = 0.0747\,in^2 < 2(0.33) = 0.66\,in^2 \end{cases}$ ✓

Deflection:

$\Delta_{allow} = l/240 = 28.5(12)/240 = 1.43''$

$\Delta_{max,inst} = 0.512'' < 1.43''$ ✓

$\Delta_{max,long\,term} = 1.335'' < 1.43''$ ✓

* deflections found using spBeam. Long term deflection includes 60% sustained Live Load over 60 months.

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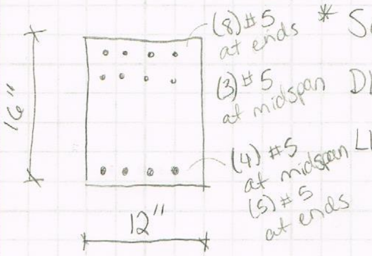
Girder Design:



* See Appendix B for spBeam Output

DL = 117 psf

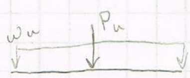
LL = 65 psf

Check deflection:

$\Delta_{TL,allow} = L/240 = 18.75(12)/240 = 0.94''$

$\Delta_{max} = 0.18'' < 0.94''$ ✓ ∴ ok

Midspan Check:



$P_u = 2.29\text{ klf}\left(\frac{28.5}{2}\right) = 32.63^k$

$w_u = 1.2(12'')(16''-12'')(150\text{ pcf})(1/144) = 60\text{ plf}$

$M_{max} = 78'^k$ (from Risa 2D → 3 spans, fixed at colums)

$A_s = 1.24\text{ in}^2$ $\quad d = 16 - 0.75'' - 0.375'' - \frac{0.625''}{2} = 14.563''$

$A'_s = 0.93\text{ in}^2$ $\quad d' = 0.75'' + 0.375'' + \frac{0.625''}{2} = 1.438''$

Assume $\varepsilon_s > \varepsilon_y$, $\varepsilon'_s > \varepsilon_y$:

$$a = \frac{A_s f_y - A'_s f_y}{0.85 f'_c b} = \frac{(1.24-0.93)(60)}{0.85(4)(12)} = 0.456''$$

$c = a/\beta_1 = 0.536''$

$$\varepsilon_s = \frac{\varepsilon_{cu}}{c}(d-c) = \frac{0.003}{0.536}(14.563-0.536) = \underset{\varepsilon_s}{0.076} > \underset{\varepsilon_y}{0.00207}$$ ✓

$$\varepsilon'_s = \varepsilon_{cu}\left(\frac{c-d'}{c}\right) = 0.003\left(\frac{0.536-1.438}{1.438}\right) = 0.002 < 0.00207$$ ∴ case 2: $\varepsilon'_s < \varepsilon_y$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

c :

$A_s f_y = A'_s f'_s + 0.85\beta_1 c b f'_c$

$1.24(60) = 0.93\left(\frac{0.003}{c}\right)(c - 2.375)(29{,}000) + 0.85^2 c\,(12)(4)$

$0 = -74.397 - \frac{1}{c}(192.161) + 34.68c$

$0 = 34.68c^2 - 74.397c - 192.161$

$c = 3.659''$

$a = 3.659(0.85) = 3.11''$

verify $\varepsilon'_s < \varepsilon_y$ and $\varepsilon_s > \varepsilon_y$

$\varepsilon'_s = \frac{0.003}{3.659}(3.659 - 1.438) = 0.00182 < 0.00207$ ✓

$\varepsilon_s = \frac{0.003}{3.659}(14.563 - 3.659) = 0.00894 > 0.00207$ ✓

ϕM_n

$= \phi\left[0.85f'_c\, b\, a\left(d - \frac{a}{2}\right) + A'_s \frac{\varepsilon_{cu}(c-d')}{c} E_s (d-d')\right]$

$= 0.9\left[0.85(4)(12)(3.11)\left(14.563 - \frac{3.11}{2}\right) + 0.93(0.00182)(29{,}000)(14.563 - 1.438)\right]$

$= 2065''^k = 172'^k > 78'^k$ ∴ ok ✓

Tension controlled?

$\varepsilon_t = \varepsilon_s = 0.00894 > 0.005$ ✓ ∴ tension controlled

Alternative System 2: Post-Tensioned Slab

Post-Tensioned Slab

- unbonded tendons: $\frac{1}{2}''\ \phi$, 7-wire strands, $A = 0.153\ in^2$
- mild reinforcing for crack control: 60 psi, #5 bars

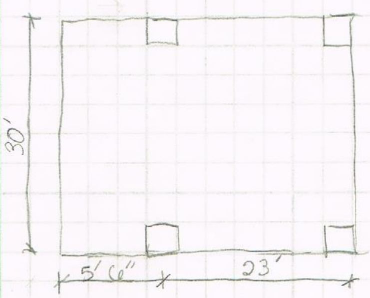
* See appendix C for ADAPT-PT analysis output

ADAPT assumptions:

- Reduce moments to face of support
- use Equivalent Frame Method

Span Change:

Bay size at top floor = 23'



Span-to-depth ratio 1:45
→ try $h = 7''$

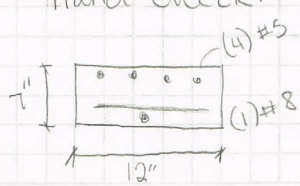
- $f'_c = 5{,}000$ psi
- $f'_{ci} = 0.75(5{,}000) = 3{,}750$ psi
- headed stud shear reinforcing
- $f_{pu} = 270$ ksi

- Average Precompression = 150 psi – 300 psi
- Percentage of load to balance = 40% – 150%
- Tendon profile = reversed parabola
- min CGS = 1"

Output:

- 19 strands
- 12.8^k force per unit width

Hand Check:



$w_u = 1.2(15 + 150\,pcf\,(7/12)) + 1.6(65) = 227$ psf $= 0.227$ klf/ft

$\frac{wl^2}{8} = \frac{0.227(23^2)}{8} = 15$ k-ft

27

$A'_s = (\ )(0.31) = 0 \ in^2$

$A_s = (1)(0.79) = 0.79 \ in^2$

$A_{ps} = (15)(0.153) = 2.907 \ in^2$

$d' = 1.5 + \frac{0.625}{2} = 1.813 \ in$

$d = 7 - 1.813 = 5.187$

$d_p = 7 - 3.5'' = 3.5''$

Unbonded tendons with span-to-depth ratio > 35:

$f_{ps} = f_{se} + 10{,}000 + \frac{f'_c}{300\rho_p}$ $f_{se} = 135$

$= 135 + 10 + 5 / (300 \times 2.907 / 3.5) = 145 \ ksi$

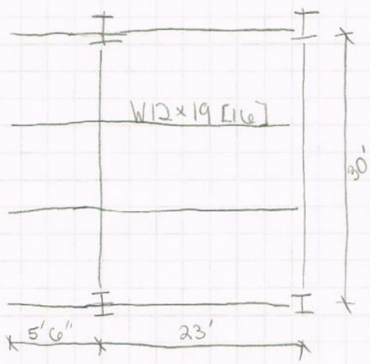
$f_{ps} \leq \min \begin{cases} f_{py} = 0.85 f_{pu} = 229{,}500 \\ f_{se} + 30{,}000 = 165{,}000 \end{cases}$

$145 < 165$ ✓

Alternate System 3: Lightweight Composite Framing

28

Slab On Deck



Verco deck

- lightweight concrete deck thickness to achieve 2hr fire rating
  → t = 3 1/4"
- need 5 1/4" total thickness

$w_u = 1.2(11) + 1.6(65) = 117$ psf
(superimposed)

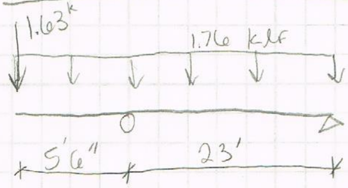
10' spacing → W2 Formlock (39 psf)

min gage to avoid shoring → 20

3 span condition, 10' span → 233 psf

29

Cantilevered Beam



$w_u = 10'[1.2(60) + 1.6(65)] = 1.76$ klf

$P_u = 163$ plf $(10') = 1.63^k$
(curtain wall)

$M_u = \frac{wx}{2l}(l^2 - a^2 - xl) - \frac{Pax}{l} = \frac{1.76(11.5)}{2(23)}(23^2 - 5.5^2 - 11.5(23)) - \frac{1.63(5.5)(\ldots)}{23}$

$= 103.07 - 4.483 = 98.6'^k$

uncomposite: W16×26 ($\phi M_n = 101'^k$), $26(28.5) = 741$ lbs

$Q_n = \min \begin{cases} 0.5 A_{sc}\sqrt{f'_c E_c} = 21.7^k & \text{(see p.18)} \\ R_g R_p A_{sc} F_u = 17.2^k & \text{(AISC table 3-21)} \end{cases}$

assume $a = 1$, $y2 = 5.25 - \frac{1.0}{2} = 4.75$

Table 3-19: want $\phi M_n \geq 98.6$
(AISC)

W10×12, $\Sigma Q_n = 115^k$, 14 studs → $12(28.5) + 14(10) = 482$ lbs

W10×15, $\Sigma Q_n = 83.8^k$, 10 studs → $15(28.5) + 10(10) = 527.5$ lbs

→ use W10×12

$b_{eff} = \min \begin{cases} 28.5(12)/8 = 42.75" \\ \frac{1}{2}(10')(12) = 60" \end{cases} \Rightarrow 85.5"$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{115}{0.85(4)(85.5)} = 0.396" < 1.0$ ∴ ok

$\Delta_{LL,max} = L/360 = 28.5(12)/360 = 0.95"$

actual deflection too large → need $I \geq 320$ in⁴

W12×19, $\Sigma Q_n = 138$, 16 studs → $19(28.5) + 16(10) = 762$ lbs → still more economical

$I_{LB} = 347 \rightarrow \Delta_{max} = 0.9" < 0.95"$ ∴ ok ✓

*see appendix D for Risa output

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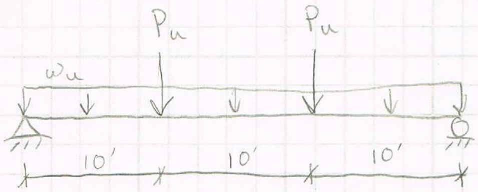
Girder Design



$P_u = 1.63 + 1.76(17') + 0.019(17)(1.2) = 32^k$

$w_u = 1.2(16) = 19.2 \text{ plf}$

$5 \text{ psf}(1.2)(17) = 102 \text{ plf}$

$w_u = 0.121 \text{ klf}$

$$M_{max} = \frac{wl^2}{8} + Pa = \frac{0.121(30^2)}{8} + 32(10) = 334'^k$$

$W24\times62 \rightarrow \phi M_n = 344'^k$

$$\Delta_{LL,max} = \min \begin{cases} L/360 = 1'' \\ 1'' \end{cases}$$

$\Delta_{TL,max} = -0.639''$ ∴ ok

(see Appendix D for Risa output)

Systems Comparison

Cost

At $21.07 per square foot, the cheapest system to build in San Francisco is a concrete slab on beams. Adding in $3 per square foot for formwork brings the approximate cost of alternative system 1 to $24.07.

Post-tensioned slabs requires dimensional lumber rather than traditional plywood form to prevent distortion during stressing of the tendons. This adds extra cost to the post-tensioning system, as does the extra labor in the post-tensioning process. An equivalent flat slab system would cost about $20 per square foot. Adding in an extra $4 for forms and $1 for labor puts the equivalent cost at $25 at least.

On average, composite construction is $26.21/sf for the bay size being considered in the third alternative system. This closely compares to the predicted average cost of the existing system as described in the next comparison. Because fire rating controls, the same deck type is used as in the existing system, so no savings are found there. The amount of steel material needed, however, is more than 40lbs less per beam member than a non-composite version of the same system would be (see bottom of page 29). This system is therefore still more practical to use compositely.

Wide flange framing has an average cost of $19.12 per square foot in San Francisco for the bay size required in the existing framing. Adding in the $8.14 per square foot deck on top of that brings the cost to $27.26. This in turn makes the existing framing the more expensive option from solely a materials standpoint.

Impact on Lateral System

Although concrete framing is the cheapest on average, its effect on 181 Fremont specifically needs to be considered. Considering the significantly greater weight of concrete framing rather than steel framing, the seismic forces of the building are going to be greater. In turn, more money would need to be spent on a lateral system.

Post-tensioned slabs reduce the slab depth and eliminate the need for concrete beams to span between members. Consequently, a lot of benefit is seen in the reduction of the building's weight. Post-tensioned slabs also experience greater amounts of story drift than traditional concrete however, and extra care and cost has to be spent in the detailing of slip connections to prevent cracking.

Alternative 3, the use of composite slabs, causes the slab on deck to be affected by the lateral system and loading. Whereas a non-composite deck would have little to no interaction with the lateral system, a composite deck may be affected by lateral loading and thus the design would require that to be considered.

Fireproofing and Compatibility With Other Disciplines

Because of the imposed 700' height limit on the project, limiting the depth of the gravity system at each floor is an important factor to consider. Being able to fit more floors within the same height due to a shallower gravity system generates more square footage and rental income for the building owner.

A major benefit of the concrete systems is the inherent fireproofing. Steel framing require a 3 hour fire proofing for this project, which adds additional material cost to the already more expensive material framing.

The small depth of the post-tensioned slabs are a great bonus for interdisciplinary compatibility, as they open up a lot of the overhead space of each level. Composite Steel Framing is the next best with the 5 ¼ inch depth slab on beams, as the composite beams allow for a much shallower depth than the existing framing does.

The large mass and depth of traditional concrete slab on beams is the least ideal as far as integration is concerned. Deeper beams are required, and as a result there would be much more difficulty in fitting MEP systems into the overhead space.

Constructibility, Labor, and Time

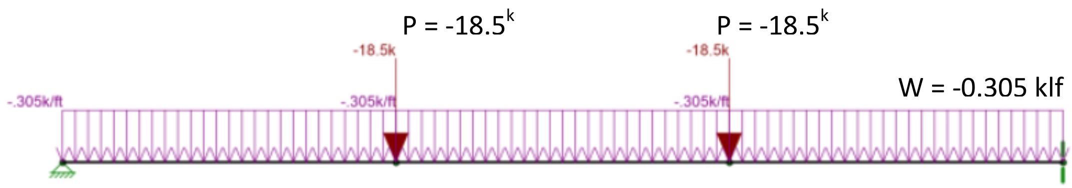
Concrete slab on beams for this project have the added benefit of less detail work in connections. Steel seismic connections are a constructability issue with the high-rise building and putting laborers at risk. Concrete, however, requires 3 days strength before workers can stand on a floor and continue work. In turn, this extends the schedule and adds time to the overall construction. The need to shore each level also much be considered.

The composite system was designed so no shoring would be required, giving it the advantage compared to concrete in that respect. Both the existing and the alternative steel systems provide quicker erection, but more expense in connection detailing.

Post-tensioned slab over 54 stories of a building require extra detail and attention to the connections with supporting columns and members. The span length and pours as well would be affected by the design, which may pose challenges in the building layout and scheduling of construction. In addition, the learning curve for the laborers is greater, extending the time of the project. The number of floors of the building, however, allow for economy in repetition.

Appendix A: Existing System Risa 2D Output

W21x62 Reduced Live Load Risa 2D Diagram



W21x62 Reduced Live Load Risa 2D Deflection Output

Joint Loads/Enforced Displacements

Joint Label	[L]oad or [D]isplacement	Direction	Magnitude (k, k-ft, in, rad)
N3	L	Y	-18.5
N4	L	Y	-18.5

Member Distributed Loads

Member Label	Direction	Start Magnitude (k/ft, F)	End Magnitude (k/ft, F)	Start Location (ft or %)	End Location (ft or %)
M1	Y	-.305	-.305	0	0
M2	Y	-.305	-.305	0	0
M3	Y	-.305	-.305	0	0

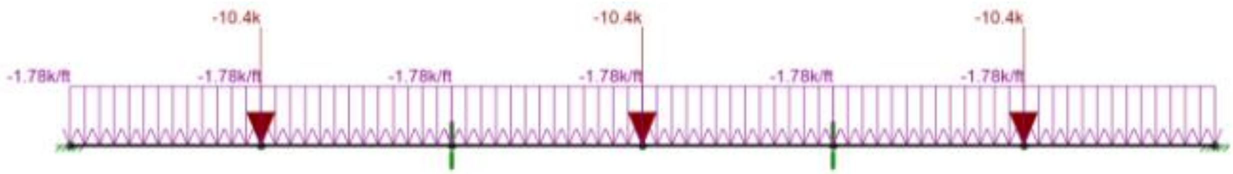
Joint Displacements

Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	-4.473e-3
N2	0	0	4.473e-3
N3	0	-.424	-2.224e-3
N4	0	-.424	2.224e-3

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	0	22.846	0
	2	0	22.122	53.4
	3	0	21.398	105.079
	4	0	20.673	155.038
	5	0	19.949	203.276
M2	1	0	1.449	203.276
	2	0	.724	205.857
	3	0	0	206.717
	4	0	-.724	205.857
	5	0	-1.449	203.276
M3	1	0	-19.949	203.276
	2	0	-20.673	155.038
	3	0	-21.398	105.079
	4	0	-22.122	53.4
	5	0	-22.846	0

W24x76 Total Load Risa 2D Diagram



W24x76 Total Load Risa 2D Deflection Output

Joint Loads/Enforced Displacements

Joint Label	[L]oad or [D]isplacement	Direction	Magnitude (k, k-ft, in, rad)
N3	L	Y	0
N4	L	Y	0
N2	L	Y	-10.4
N4	L	Y	-10.4
N6	L	Y	-10.4

Member Distributed Loads

Member Label	Direction	Start Magnitude (k/ft, F)	End Magnitude (k/ft, F)	Start Location (ft or %)	End Location (ft or %)
M1	Y	-1.78	-1.78	0	0
M2	Y	-1.78	-1.78	0	0
M3	Y	-1.78	-1.78	0	0
M4	Y	-1.78	-1.78	0	0
M5	Y	-1.78	-1.78	0	0
M6	Y	-1.78	-1.78	0	0

Joint Displacements

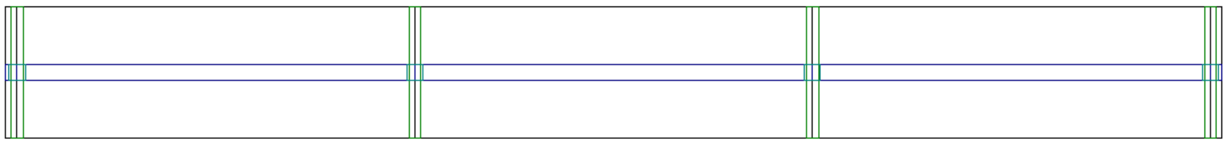
Joint Label	X Translation (in)	Y Translation (in)	Rotation (radians)
N1	0	0	0
N2	0	-.026	0
N3	0	0	0
N4	0	-.026	0
N5	0	0	0
N6	0	-.026	0
N7	0	0	0

Member Section Forces

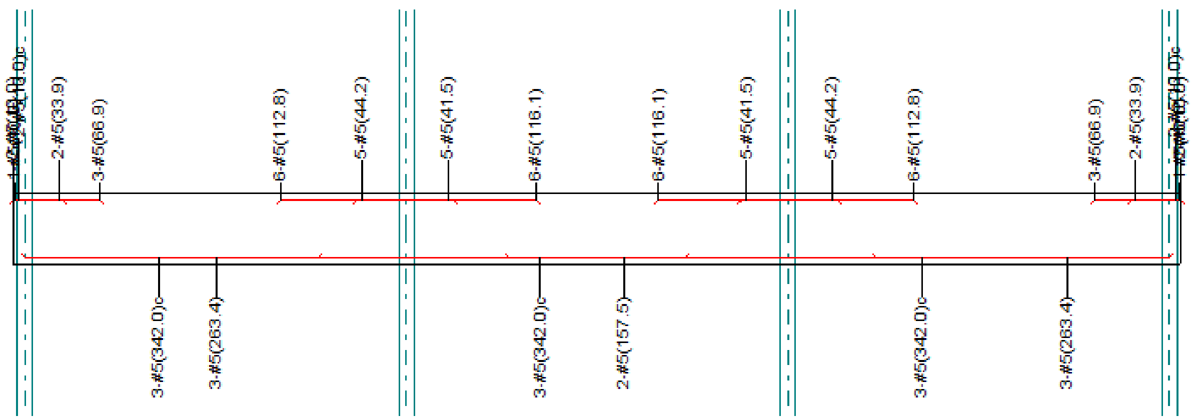
Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	0	21.887	-76.523
	2	0	17.716	-30.114
	3	0	13.544	6.519
	4	0	9.372	33.373
	5	0	5.2	50.449
M2	1	0	-5.2	50.449
	2	0	-9.372	33.373
	3	0	-13.544	6.519

Appendix B: Alternative 1 spBeam Output

Transverse Beams Input Diagram



Transverse Beam Reinforcing



Transverse Beam Strength

Units: x (ft), As (in^2), PhiMn (k-ft)

Span	x	AsTop	AsBot	PhiMn-	PhiMn+
2	0.000	1.24	2.20	-74.81	137.59
	0.500	1.24	2.20	-74.81	137.59
	0.890	1.24	2.20	-74.81	137.59
	1.890	0.93	2.20	-56.92	137.59
	3.694	0.93	2.20	-56.92	137.59
	4.694	0.00	2.20	0.00	137.59
	10.125	0.00	2.20	0.00	137.59
	14.250	0.00	2.20	0.00	137.59
	18.184	0.00	2.20	0.00	137.59
	18.375	0.00	2.12	0.00	132.88
	20.381	0.00	1.32	0.00	83.09
	28.000	0.00	1.32	0.00	83.09
	28.500	0.00	1.32	0.00	83.09

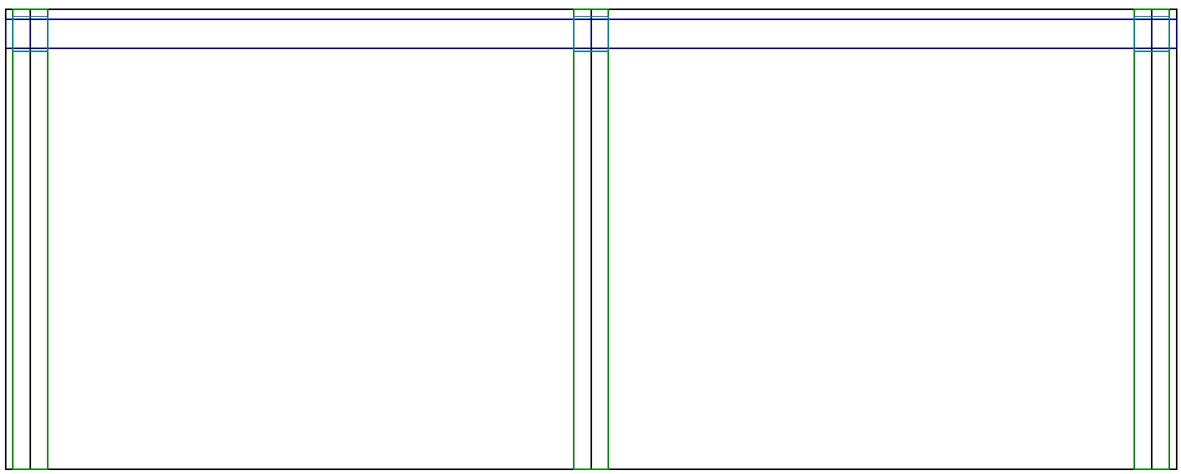
Transverse Beam Instantaneous Deflection

Span	Dsust	Lambda	Dcs	Dcs+lu	Dcs+l	Dtotal
3	0.411	2.000	0.822	0.924	1.075	1.335

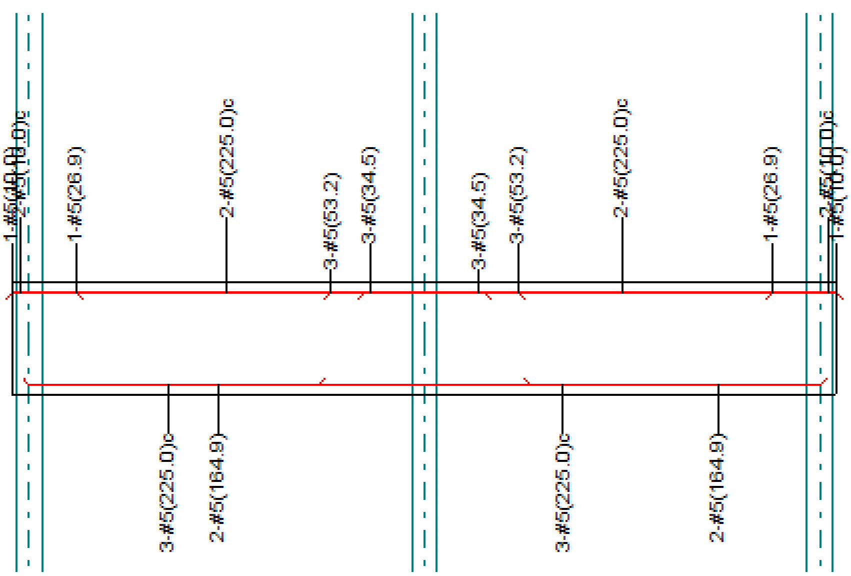
Transverse Beam Long-term Deflections

Span	Ddead	Dlive	Dtotal
3	0.260	0.253	0.512

Longitudinal Beam Input Diagram



Longitudinal Beam Reinforcing



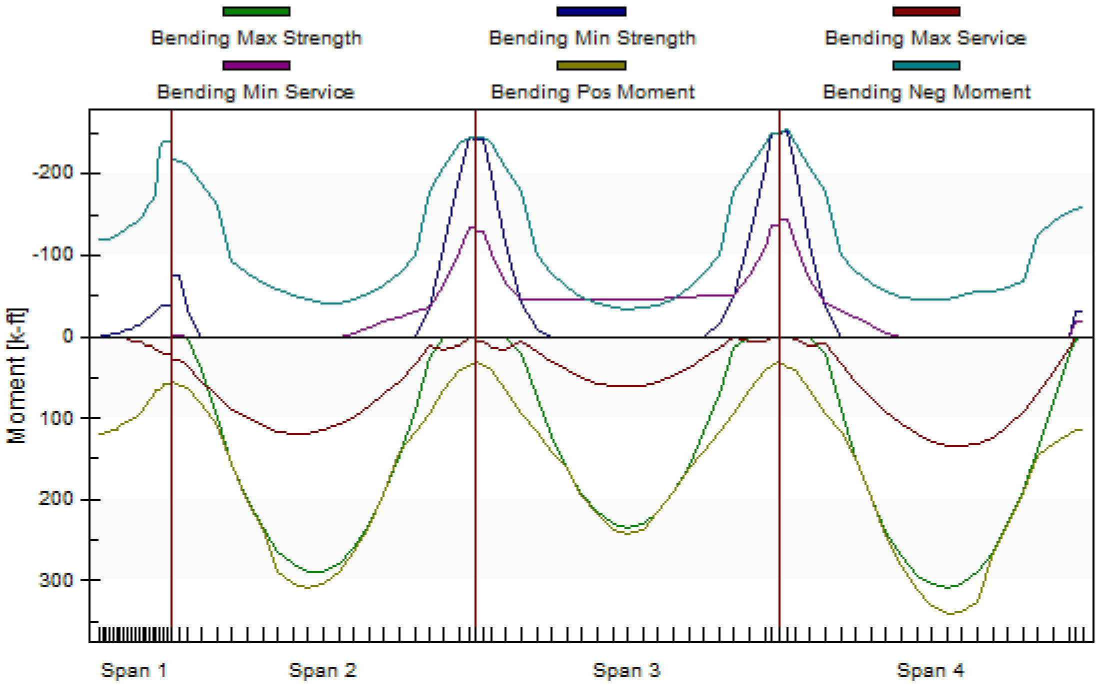
Longitudinal Beam Strength

Units: x (ft), As (in^2), PhiMn (k-ft)

Span	x	AsTop	AsBot	PhiMn-	PhiMn+
2	0.000	0.93	1.55	-56.51	97.26
	0.583	0.93	1.55	-56.51	97.26
	1.113	0.93	1.55	-56.51	97.26
	2.242	0.62	1.55	-38.31	97.26
	6.737	0.62	1.55	-38.31	97.26
	9.375	0.62	1.55	-38.31	97.26
	11.859	0.62	1.55	-38.31	97.26
	12.013	0.62	1.50	-38.31	94.15
	13.745	0.62	0.93	-38.31	58.76
	14.317	0.62	0.93	-38.31	58.76
	15.798	1.55	0.93	-91.01	58.76
	15.878	1.55	0.93	-91.01	58.76
	17.360	2.48	0.93	-137.98	58.76
	18.167	2.48	0.93	-137.98	58.76
	18.750	2.48	0.93	-137.98	58.76
3	0.000	2.48	0.93	-137.98	58.76
	0.583	2.48	0.93	-137.98	58.76
	1.390	2.48	0.93	-137.98	58.76
	2.872	1.55	0.93	-91.01	58.76
	2.952	1.55	0.93	-91.01	58.76
	4.433	0.62	0.93	-38.31	58.76
	5.005	0.62	0.93	-38.31	58.76
	6.737	0.62	1.50	-38.31	94.15
	6.891	0.62	1.55	-38.31	97.26
	9.375	0.62	1.55	-38.31	97.26
	12.013	0.62	1.55	-38.31	97.26
	16.508	0.62	1.55	-38.31	97.26
	17.637	0.93	1.55	-56.51	97.26
	18.167	0.93	1.55	-56.51	97.26
	18.750	0.93	1.55	-56.51	97.26

Appendix C: Alternative 2 ADAPT Output

Design Moment



Provided Additional Rebar

Total Strip Provided Rebar

Span	ID	Location	From	Quantity	Size	Length	Area
			ft			ft	in2
CL	1	TOP	4.40	7	5	6.00	2.17
1	2	TOP	18.40	7	5	9.50	2.17
2	3	TOP	18.40	8	5	9.50	2.48
3	4	TOP	18.40	7	5	5.00	2.17
1	5	TOP	19.71	7	5	7.00	2.17
2	6	TOP	19.71	8	5	7.00	2.48
1	7	BOT	2.47	3	8	16.00	2.37
2	8	BOT	9.37	1	8	4.50	0.79
3	9	BOT	4.77	4	8	16.00	3.16
1	10	BOT	5.92	3	8	10.00	2.37
3	11	BOT	7.07	4	8	10.00	3.16

Punching Shear

(Assuming 14"x14" columns)

Critical Section Stresses

Label	Layer	Cond.	Factored shear	Factored moment	Stress due to shear	Stress due to moment	Total stress	Allowable stress	Stress ratio
			k	k-ft	ksi	ksi	ksi	ksi	
1	1	1	-89.37	+67.12	0.20	0.104	0.300	0.220	1.362
2	1	1	-171.63	-10.89	0.37	0.017	0.392	0.220	1.781
3	1	1	-175.03	+12.77	0.38	0.020	0.402	0.220	1.828
4	1	2	-69.57	-70.55	0.22	0.104	0.328	0.212	1.548

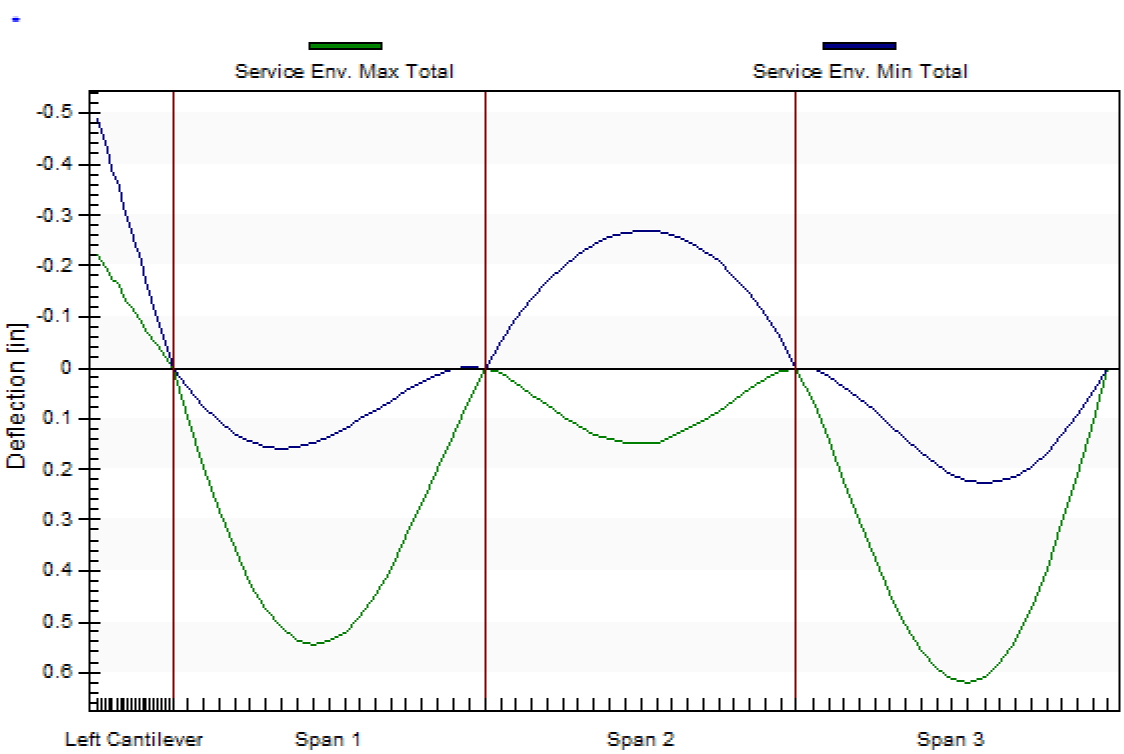
Punching Shear Reinforcement

Reinforcement option: Shear Studs
Stud diameter: 0.5
Number of rails per side: 2

Col.	Dist	Dist	Dist	Dist	Dist	Dist	Dist	Dist	Dist	Dist
	in	in	in	in	in	in	in	in	in	in
1	2.9	5.8	8.7	11.6	14.5					
2	2.9	5.8	8.7	11.6	14.5	17.4	20.2	23.1	26.0	
3	2.9	5.8	8.7	11.6	14.5	17.4	20.2	23.1	26.0	
4	2.9	5.8	8.7	11.6	14.5	17.4	20.2			

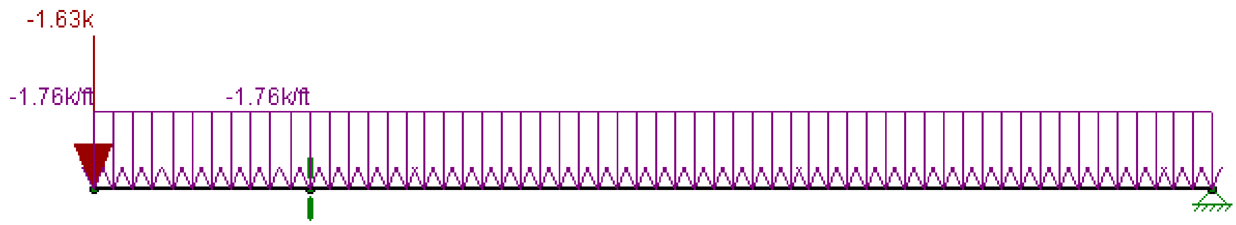
Dist. = Distance measured from the face of support
Note: Columns with --- have not been checked for punching shear.
Note: Columns with *** have exceeded the maximum allowable shear stress.

Deflection



Appendix D: Alternative 3 Risa 2D Output

Cantilever Beam Input



Cantilever Beam Deflection

Member Label	S...	x [in]	y [in]
M1	1	0	.535
	2	0	.406
	3	0	.275
	4	0	.141
	5	0	0
M2	1	0	0
	2	0	-.608
	3	0	-.899
	4	0	-.658
	5	0	0

Girder Input

Girder Deflection

Member Label	S...	x [in]	y [in]
M1	1	0	0
	2	0	-.182
	3	0	-.349
	4	0	-.49
	5	0	-.589
M2	1	0	-.589
	2	0	-.639
	3	0	-.521
	4	0	-.29
	5	0	0