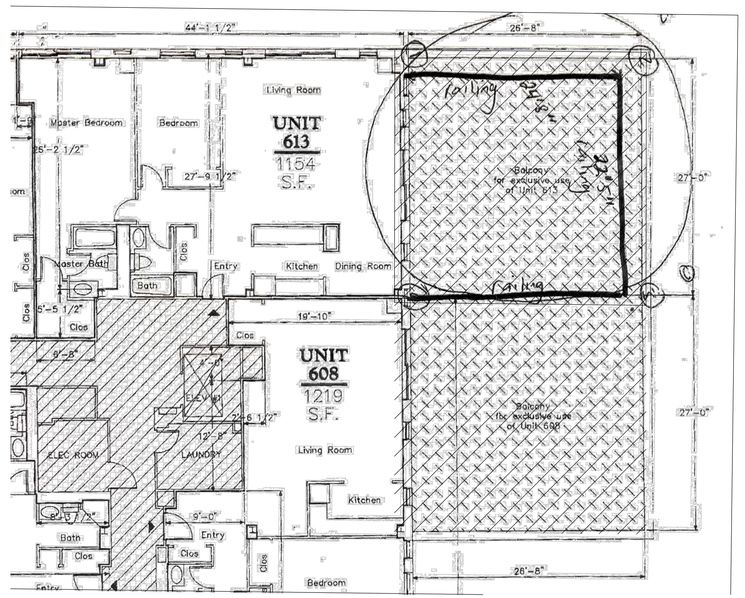
950 MASSACHUSETTS AVE
BAY SQUARE
Unit 607 & 613 Balcony, 6th Floor
CAMBRIDGE, MA 02139

AERIAL VIEW

GENERAL NOTES:
1. ALL WORK SHALL BE COMPLETED WIHIN THE MASSACHUSETTS STATE BUILDING CODE 8TH EDITION AND OTHER LOCAL REGULATIONS GOVERNING WORK OF THIS SORT.

2. WORK TO INCLUDE DEMOLITION AND OVERBURDEN LAYOUT AS SHOWN IN DRAWINGS UNLESS OTHERWISE NOTED.

3. ALL DEMOLITION & EXCESS MATERIAL WILL BE REMOVED FROM SITE AND DISPOSED OF LEGALLY.



PROPOSED AREA OF WORK

RECOVER GREEN ROOFS
9 OLIVE SQUARE
SOMERVILLE, MA
Phone: 857.209.4464
Fax: 617.207.4341
www.recovergreenroofs.com
info@recovergreenroofs.com

PROJECT TITLE

950 MASSACHUSETTS AVE
CAMBRIDGE, MA 02139

DESIGNER'S SEAL

SHEET DESCRIPTION

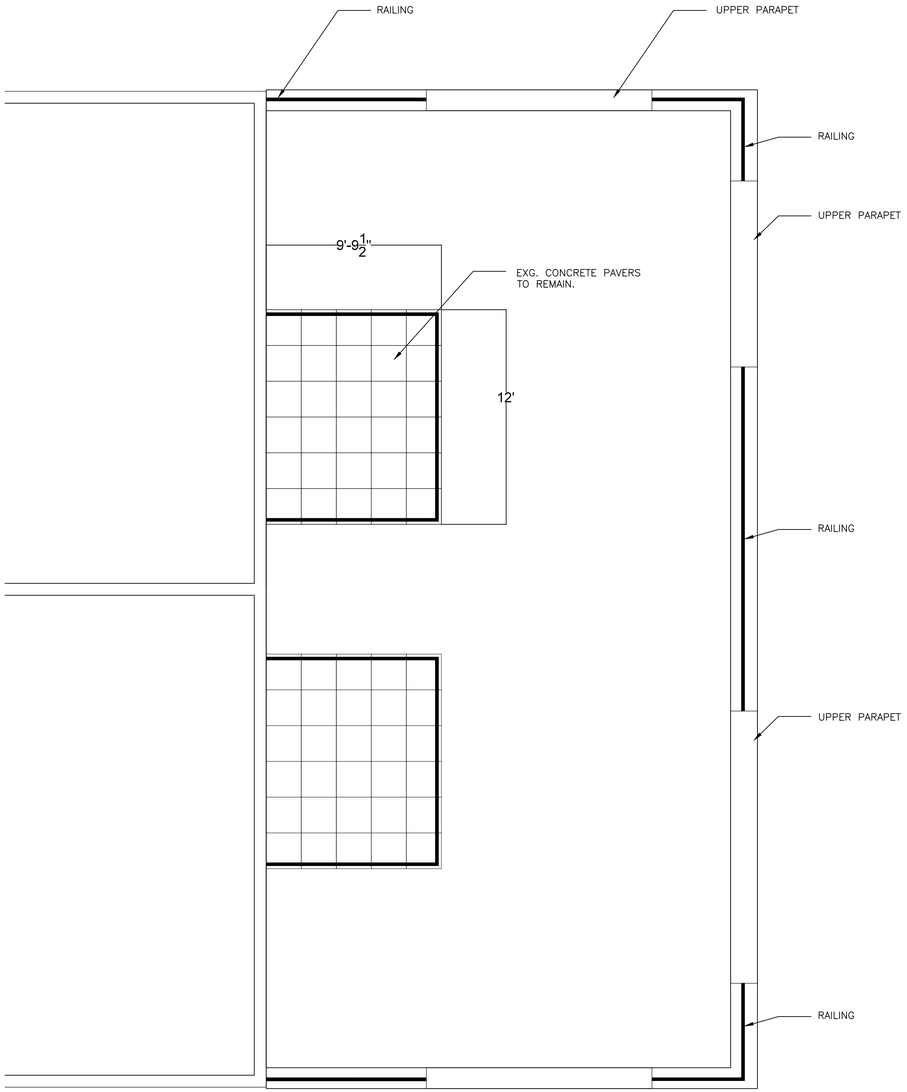
COVER

DESIGN: T. SANCHEZ
REVIEW: B. SHEA
DATE: 03.30.2015
SCALE: N.T.S

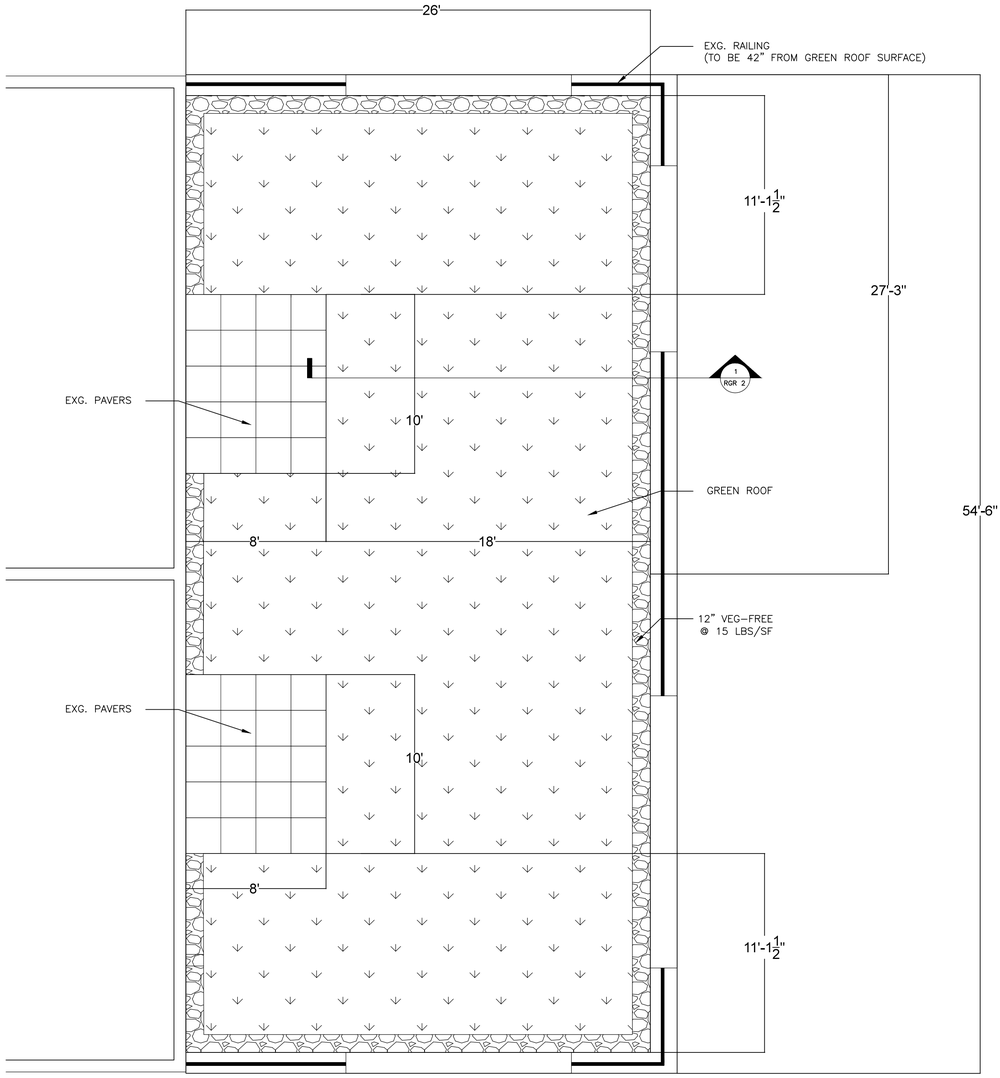
PROJECT NUMBER

RGR-0.0

COVER



1 EXISTING BALCONY PLAN



2 PROPOSED BALCONY PLAN

REcover
GREEN ROOFS, LLC
DESIGN. BUILD. MAINTAIN
RECOVER GREEN ROOFS
9 OLIVE SQUARE
SOMERVILLE, MA
Phone: 857.209.4464
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PROJECT TITLE

950 MASSACHUSETTS AVE
CAMBRIDGE, MA 02139

DESIGNER'S SEAL

SHEET DESCRIPTION

PROPOSED ROOF PLAN

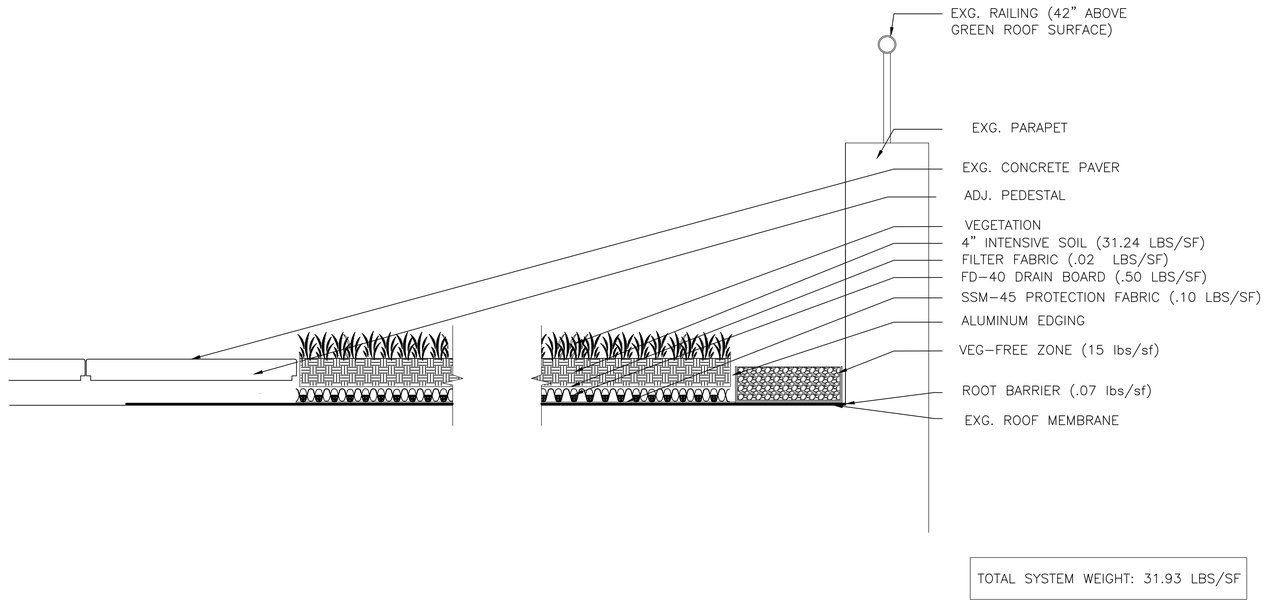
DESIGN: T. SANCHEZ
REVIEW: B. SHEA
DATE: 3.30.15
SCALE: AS NOTED

PROJECT NUMBER

RGR-1.0

2 OF 7

ROOF PLAN



1 PARAPET/ VEG-FREE / GREEN ROOF / PAVER DETAIL

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950 MASSACHUSETTS AVE
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DESIGNER'S SEAL

SHEET DESCRIPTION

DETAILS

DESIGN: T. SANCHEZ
REVIEW: B. SHEA
DATE: 3.30.15
SCALE: N.T.S

PROJECT NUMBER

RGR-2.0

DETAILS

950 Mass Ave

Roof Reserve Capacity for Additional Loads - Rev 3

Cambridge, MA

2/6/2014 **Rev 3 - used 36ksi, added ceiling loads and added plan on wind uplift**
9/19/2013 Revision 2
5/19/2013 Revision 1
3/20/2013 Original

Prepared By:

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Table of Contents

1.0 General Structural Criteria

1.1 Project Description

The scope of this document includes the Roof Reserve Capacity for Additional Loads - Rev 3 of the 950 Mass Ave project. The structural design is based on the building codes and standards listed below.

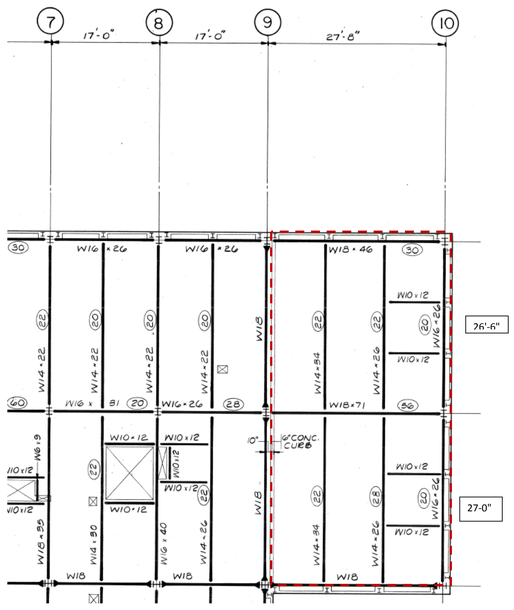
1.2 Codes

- 780 CMR, 8th Edition of the Massachusetts State Building code consisting of the 2010 International Building Code as modified by Massachusetts.
- ASCE 7-05
- American Institute of Steel Construction, "Manual of Steel Construction, 13TH edition"
- American Institute of Steel Construction, "Code of Standard Practice"
- Structural Welding Code, AWS D-1

1.3 Low Roof Location on Building

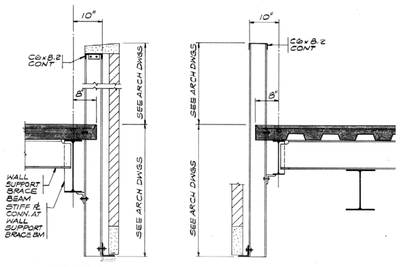
1.4 Images of Current Roof Deck

1.5 Structural Plan



2.0 Code Loads

2.1 Dead Load



NOTES:

1. UNLESS OTHERWISE NOTED, FLOOR CONSTRUCTION SHALL BE 6"-20 GA. COMPOSITE GALVANIZED STEEL DECK W/3½" LIGHTWEIGHT (110 PCF) CONCRETE TOPPING. CONCRETE TOPPING SHALL BE REINFORCED W/4x4-W2.9/W2.9 WELDED WIRE FABRIC IN FLAT SHEETS LOCATED 1" BELOW TOP OF CONCRETE. (SEE DRAWING S-1 FOR DETAIL.)
2. (N) INDICATES THE NUMBER OF ¾"ϕ x 4" LONG SHEAR CONNECTORS WELDED TO EACH BEAM. REFER TO DRAWING S-1 FOR DISTRIBUTION OF SHEAR CONNECTORS.
3. TOP OF STEEL BEAMS SHALL BE AT EL. 95'-2¾" UNLESS OTHERWISE NOTED THUS (±0").

Dead Load of 3 1/4" LW Conc + 2" MD = 42 psf
Tapered Insulation and Roof Membrane = 5 psf
Steel Self Weight = Included in calculations

2.2 Min Live Load

TABLE 1607.1 MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[a]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
1. Apartments (see residential)	-	-
2. Access floor systems		
Office use	50	2,000
Computer use	100	2,000
3. Armories and drill rooms	150	-
4. Assembly areas and theaters		
Fixed seats (fastened to floor)	60	
Follow spot, projections and control rooms	50	
Lobbies	100	-
Movable seats	100	
Stages and platforms	125	
Other assembly areas	100	
5. Balconies (exterior) and decks[b]	Same as occupancy served	-
27. Residential		
One- and two-family dwellings		
Uninhabitable attics without storage[i]	10	
Uninhabitable attics with limited storage[i,k]	20	
Habitable attics and sleeping areas	30	-
All other areas	40	
Hotels and multifamily dwellings		
Private rooms and corridors serving them	40	
Public rooms and corridors serving them	100	
29. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies		
Fabric construction supported by a light weight rigid skeleton structure	5	
All other construction	nonreducible	
Ordinary flat, pitched, and curved roofs	20	
Primary roof members, exposed to a work floor	20	
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs:		
Over manufacturing, storage ware houses, and repair garages		2,000
All other occupancies		300
Roofs used for other special purposes	Note 1	Note 1
Roofs used for promenade purposes	60	
Roofs used for roof gardens or assembly purposes	100	

1607.11.2.2 Special-purpose roofs. Roofs used for promenade purposes, roof gardens, assembly purposes or other special purposes, and marquees, shall be designed for a minimum live load, L_o, as specified in Table 1607.1. Such live loads are permitted to be reduced in accordance with Section 1607.9. Live loads of 100 psf (4.79 kN/m²) or more at areas of roofs classified as Group A occupancies shall not be reduced.

Code Required Min Roof Live Load is 100 psf

Since not group A, LL is permitted to be reduced (to find reserve capacity beyond the 100psf)

2.3 Roof Live Load

MINIMUM ROOF LIVE LOADS IN POUNDS-FORCE PER SQUARE FOOT OF HORIZONTAL PROJECTION

ROOF SLOPE	TRIBUTARY LOADED AREA IN SQUARE FEET FOR ANY STRUCTURAL MEMBER		
	0 to 200	201 to 600	Over 600
Flat or rise less than 4 inches per foot (1:3)	20	16	12
Rise 4 inches per foot (1:3) to less than 12 inches per foot (1:1)	16	14	12
Rise 12 inches per foot (1:1) and greater	12	12	12

For SI: 1 square foot = 0.0929 m², 1 pound per square foot = 0.0479 kPa, 1 inch per foot = 83.3 mm/m.

This 20psf does not control, just added section for completeness - used 100psd Live

Some info from the old 7th Ed CMR code...

1607.11.2.3 Landscaped Roofs. Where roofs are to be landscaped, the uniform design live load in the landscaped area shall be 20 pounds per square foot (0.958 kN/m²). The weight of the landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

So, landscaping weight can be considered a dead load for calc purposes. Still, using 100psf for live.

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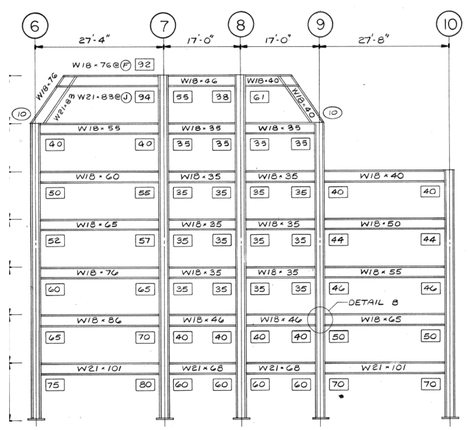
4 OF 7

STRUCTURAL ASSESSMENT

2.4 Snow

As per 780 CMR, the design ground snow load for Cambridge, MA is 45 psf.

City/Town	p_g	V	Ss	S_1
Boston	45	105	0.29	0.068
Bourne	35	120	0.21	0.056
Boxborough	55	100	0.28	0.070
Boxford		110	0.33	0.075
Boylston	55	100	0.25	0.067
Braintree	45	105	0.27	0.066
Brewster	35	120	0.18	0.052
Bridgewater	45	110	0.24	0.062
Brimfield	55	100	0.23	0.065
Brockton	45	110	0.25	0.064
Brookfield	55	100	0.23	0.065
Brookline	45	105	0.28	0.068
Buckland	65	100	0.22	0.068
Burlington	55	105	0.30	0.071
Cambridge	45	105	0.28	0.068
Canton	55	100	0.26	0.066



Roof Deck Height above grade = 95' - 41' (approx) = 54 ft
Height Diff between Upper and Lower Roof = 10.8ft x 2 = 21.6 ft (for drift calc)

SNOW LOADING (ASCE7-05)

TEDDS calculation version 1.0.02

Building details

Roof type	Monopitch
Width of roof	b = **27.70** ft
Slope of roof 1	α = **0.00** deg

Ground snow load

Ground snow load	p_g = **45.00** lbf/ft²
Density of snow	γ = min(0.13 × p_g / 1ft + 14lbf/ft³, 30lbf/ft³) = **19.85** lbf/ft³
Terrain type	B
Exposure condition (Table 7-2)	Fully exposed
Exposure factor (Table 7-2)	C_e = **0.90**
Thermal condition (Table 7-3)	All
Thermal factor (Table 7-3)	C_t = **1.10**
Importance category (Table 1-1)	II
Importance factor (Table 7-4)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	p_{r_min} = I_s × 20 lbf/ft² = **20.00** lbf/ft²
Flat roof snow load (Sect 7.3)	p_r = max(0.7 × C_e × C_t × I_s × p_g, p_{r_min}) = **31.19** lbf/ft²

Cold roof slope factor (C_t > 1.0)

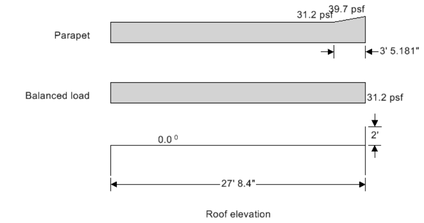
Roof surface type	Slippery
Ventilation	Ventilated
Thermal resistance (R-value)	R = **30.00** °F h ft² / Btu
Roof slope factor Fig 7-2b (dashed line)	C_s = **1.00**

Monoslope

Sloped roof snow load (Cl.7.4)	p_s = C_s × p_r = **31.19** lbf/ft²

Right parapet

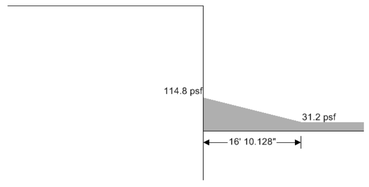
Balanced snow load height	h_b = p_r / γ = **1.57** ft
Height of right parapet	h_{pprk} = **2.00** ft
Height from balance load to top of right parapet	h_{c_pprk} = h_{ppark} - h_b = **0.43** ft
Length of roof - right parapet	l_{u_pprk} = b_1 = **27.70** ft
Drift height windward drift - right parpet	$h_{d\_1\_pprk}$ = 0.75 × (0.43 × (max(25 ft, l_{u_pprk}) × 1ft²)^1/3 × (p_g / 1lbf/ft² + 10)^1/4 - 1.5ft) = **1.53** ft
Drift height - right parapet	h_{d_pprk} = min($h_{d\_1\_pprk}$, h_{ppark} - h_b) = **0.43** ft
Drift width	W_{d_pprk} = min(4 × $h_{d\_1\_pprk}$, 8 × (h_{ppark} - h_b)) = **3.43** ft
Drift surcharge load - right parapet	p_{d_pprk} = h_{d_pprk} × γ = **8.52** lbf/ft²



Roof elevation

Drift calculations

Balanced snow load height	h_b = p_r / γ = **1.57** ft
Length of upper roof	l_u = **116.00** ft
Length of lower roof	l_l = **27.70** ft
Height diff between uppper and lower roofs	h_{diff} = **21.60** ft
Height from balance load to top of upper roof	h_c = h_{diff} - h_b = **20.03** ft
Drift height leeward drift	h_{d_l} = 0.43 × (max(25 ft, l_u) × 1ft²)^1/3 × (p_g / 1lbf/ft² + 10)^1/4 - 1.5ft = **4.21** ft
Drift height windward drift	h_{d_w} = 0.75 × (0.43 × (max(25 ft, l_l) × 1ft²)^1/3 × (p_g / 1lbf/ft² + 10)^1/4 - 1.5ft) = **1.53** ft
Maximum lwww drift height	h_{d_max} = max(h_{d_w}, h_{d_l}) = **4.21** ft
Drift height	h_d = min(h_{d_max}, h_c) = **4.21** ft
Drift width	W_d = min(4 × h_{d_max}, 8 × h_d) = **16.84** ft
Drift surcharge load	p_d = h_d × γ = **83.59** lbf/ft²



Elevation on snow drift

Snow Range = 115psf to 31.2 psf
Average Snow Load on Left Side (16ft from Wall) = (114.8 + 31.2)/2 = 73 psf

2.5 AISC 7-05 LRFD Load Combinations

2.3 COMBINING FACTORED LOADS USING STRENGTH DESIGN

2.3.1 Applicability. The load combinations and load factors given in Section 2.3.2 shall be used only in those cases in which they are specifically authorized by the applicable material design standard.

2.3.2 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

2.6 Wind Uplift

Procedure

Occupancy category (table 1-1)	Category = **2**
Basic wind speed (sect. 6.5.4)	V = **105.0** mph
Region	**Hurricane Prone**
Importance factor (table 6-1)	I = **1.00**
Exposure category (sect. 6.5.6)	**B**
Wind directionality factor	K_d = **0.85**
Topographic factor	K_{zt} = **1.00**

Design procedure - analytical procedure (Method2)

Velocity pressure at mean roof height 'h'(ASCE 7-05, cl. 6.5.10)

Case of loading system (table 6-3)	Case = **1**
Velocity pressure exposure coefficient	K_h = **0.83**
Velocity pressure at mean roof height 'h'	q_h = 0.00256 × K_h × K_{zt} × K_d × V^2 × I × 1 psf / mph² = **19.82** psf

Design wind pressure for MWFRS of low-rise enclosed and partially enclosed buildings (alternative procedure)

Velocity pressure at mean roof height 'h'	q_h = **19.82** psf

External and internal pressure coefficients (fig. 6-5)

Positive internal pressure coefficient	GC_{pi_pos} = **0.18**
Negative internal pressure coefficient	GC_{pi_neg} = **-0.18**

Building surface 1

External pressure coeff. for surface 1 (fig. 6-10)	GC_{pf_1} = **0.40**
With positive GC_{pi}	p_{1_s1} = q_h × ((GC_{pf_1}) - (GC_{pi_pos})) = **4.36** psf
With negative GC_{pi}	p_{2_s1} = q_h × ((GC_{pf_1}) - (GC_{pi_neg})) = **11.49** psf

Building surface 2

External pressure coeff. for surface 2 (fig. 6-10)	GC_{pf_2} = **-0.69**
With positive GC_{pi}	p_{1_s2} = q_h × ((GC_{pf_2}) - (GC_{pi_pos})) = **-17.24** psf
With negative GC_{pi}	p_{2_s2} = q_h × ((GC_{pf_2}) - (GC_{pi_neg})) = **-10.11** psf

Building surface 3

External pressure coeff. for surface 3 (fig. 6-10)	GC_{pf_3} = **-0.37**
With positive GC_{pi}	p_{1_s3} = q_h × ((GC_{pf_3}) - (GC_{pi_pos})) = **-10.90** psf
With negative GC_{pi}	p_{2_s3} = q_h × ((GC_{pf_3}) - (GC_{pi_neg})) = **-3.77** psf

Building surface 4

External pressure coeff. for surface 4 (fig. 6-10)	GC_{pf_4}= **-0.29**
With positive GC_{pi}	p_{1_s4} = q_h × ((GC_{pf_4}) - (GC_{pi_pos})) = **-9.31** psf
With negative GC_{pi}	p_{2_s4} = q_h × ((GC_{pf_4}) - (GC_{pi_neg})) = **-2.18** psf

Building surface 5

External pressure coeff. for surface 5 (fig. 6-10)	GC_{pf_5} = **-0.45**
With positive GC_{pi}	p_{1_s5} = q_h × ((GC_{pf_5}) - (GC_{pi_pos})) = **-12.48** psf
With negative GC_{pi}	p_{2_s5} = q_h × ((GC_{pf_5}) - (GC_{pi_neg})) = **-5.35** psf

Building surface 6

External pressure coeff. for surface 6 (fig. 6-10)	GC_{pf_6} = **-0.45**
With positive GC_{pi}	p_{1_s6} = q_h × ((GC_{pf_6}) - (GC_{pi_pos})) = **-12.48** psf
With negative GC_{pi}	p_{2_s6} = q_h × ((GC_{pf_6}) - (GC_{pi_neg})) = **-5.35** psf

Building surface 1E

External pressure coeff. for surface 1E (fig. 6-10)	GC_{pf_1E} = **0.61**
With positive GC_{pi}	p_{1_s1E} = q_h × ((GC_{pf_1E}) - (GC_{pi_pos})) = **8.52** psf
With negative GC_{pi}	p_{2_s1E} = q_h × ((GC_{pf_1E}) - (GC_{pi_neg})) = **15.65** psf

Building surface 2E

External pressure coeff. for surface 2E (fig. 6-10)	GC_{pf_2E} = **-1.07**
With positive GC_{pi}	p_{1_s2E} = q_h × ((GC_{pf_2E}) - (GC_{pi_pos})) = **-24.77** psf
With negative GC_{pi}	p_{2_s2E} = q_h × ((GC_{pf_2E}) - (GC_{pi_neg})) = **-17.64** psf

Building surface 3E

External pressure coeff. for surface 3E (fig. 6-10)	GC_{pf_3E} = **-0.53**
With positive GC_{pi}	p_{1_s3E} = q_h × ((GC_{pf_3E}) - (GC_{pi_pos})) = **-14.07** psf
With negative GC_{pi}	p_{2_s3E} = q_h × ((GC_{pf_3E}) - (GC_{pi_neg})) = **-6.94** psf

Building surface 4E

External pressure coeff. for surface 4E (fig. 6-10)	GC_{pf_4E} = **-0.43**
With positive GC_{pi}	p_{1_s4E} = q_h × ((GC_{pf_4E}) - (GC_{pi_pos})) = **-12.09** psf
With negative GC_{pi}	p_{2_s4E} = q_h × ((GC_{pf_4E}) - (GC_{pi_neg})) = **-4.95** psf

Therefore, wind uplift = -17.3 psf generally (Zone 2/3) and -24.8psf (around 6ft wide strips at perimeter).

Since load combination is 0.6D - W, the dead load will need to be 17.3/0.6 = 29 psf or more to not need to positively attach to roof. At edges, this becomes 24.8/.6 or 41 psf. This is for future details of anchoring the decking and/or planters.

2.7 Serviceability

780 CMR TABLE 5301.7
ALLOWABLE DEFLECTION OF STRUCTURAL MEMBERS[a,b,c]

STRUCTURAL MEMBER	ALLOWABLE DEFLECTION
Rafters having slopes greater than 3/12 with no finished ceiling attached to rafters	L/180
Interior walls and partitions	H/180
Floors and plastered ceilings	L/360
All other structural members	L/240
Exterior walls with plaster or stucco finish	H/360
Exterior walls—wind loads[a] with brittle finishes	L/240
Exterior walls—wind loads[a] with flexible finishes	L/120

Note: L = span length, H = span height.

a. The wind load shall be permitted to be taken as 0.7 times the Component and Cladding loads for the purpose of the determining deflection limits herein.

b. For cantilever members, *L* shall be taken as twice the length of the cantilever.

c. For aluminum structural members or panels used in roofs or walls of sun-room additions or patio covers, not supporting edge of glass or sandwich panels, the total load deflection shall not exceed L/60. For sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed L/120.

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PROJECT NUMBER

RGR-3.1

5 OF 7

STRUCTURAL ASSESSMENT

3.0 CALCULATIONS OF ROOF

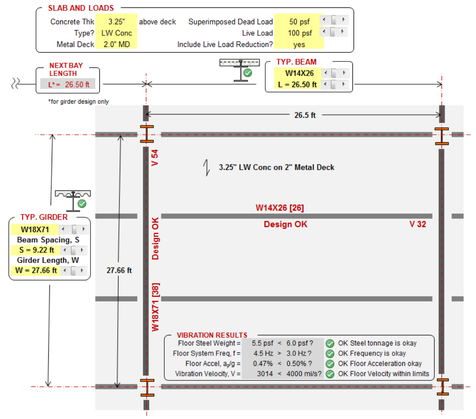
3.1 Material Strength

STRUCTURAL STEEL

1. ALL STRUCTURAL STEEL MATERIALS, WORKMANSHIP AND DETAILS SHALL CONFORM TO THE LATEST EDITION OF THE AISC "SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS". STRUCTURAL STEEL SHALL CONFORM TO SPECIFICATION ASTM-A36, EXCEPT FOR STRUCTURAL STEEL TUBES, WHICH SHALL CONFORM TO ASTM A-500, GRADE B (fy = 46 KSI).

Use 36ksi (building is post 1980s, but 36ksi likely used)

3.2 Design with Max Reserve Load of 35 psf



SDL = 35 psf reserve + 15psf ceiling/lighting/roofing/tapered insulation = 50psf SDL + Steel Weight

Design OK for 35 psf max reserve load.

TYP. BEAM RESULTS

Strength	Precomposite DL Moment Mu =	56 k-ft	< 109 k-ft ?	OK for moment cap (steel only)
	Partially Composite Moment Mu =	212 k-ft	< 212 k-ft ?	OK for moment cap (wl 26 studs)
	Full Composite (if req'd) Moment Mu =	212 k-ft	< 241k-ft ?	OK for moment cap (wl 33 studs)
	Shear Vu =	32 k	< 69 k ?	OK for shear capacity
Deflection	Precomposite DL Deflection =	L / 493	< L / 240 ?	OK for precomposite deflection
	Composite SDL+LL Deflection =	L / 452	< L / 360 ?	OK for composite deflection
	Total Deflection (minus camber) =	L / 261	< L / 240 ?	OK for Total Deflection
	Number of Studs per foot < Max?	1.0 s/ft	< 2 s/ft ?	OK for Stud Count

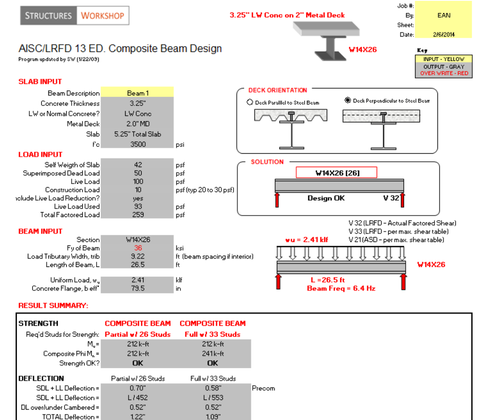
TYP. GIRDER RESULTS

Strength	Precomposite DL Moment Mu =	156 k-ft	< 376 k-ft ?	OK for moment cap (steel only)
	Partially Composite Moment Mu =	490 k-ft	< 500 k-ft ?	OK for moment cap (wl 28 studs)
	Full Composite (if req'd) Moment Mu =	490 k-ft	< 729 k-ft ?	OK for moment cap (wl 79 studs)
	Shear Vu =	53 k	< 178 k ?	OK for shear capacity
Deflection	Precomposite DL Deflection =	L / 739	< L / 240 ?	OK for precomposite deflection
	Composite SDL+LL Deflection =	L / 375	< L / 360 ?	OK for composite deflection
	Total Deflection (minus camber) =	L / 267	< L / 240 ?	OK for Total Deflection
	Beam Depth < Girder Depth Req'd?	14 <	19 ?	OK No Depth Restriction is Req'd
	Number of Studs per foot < Max?	1.4 s/ft	< 2 s/ft ?	OK for Stud Count

NOTES AND OVERWRITES

1. **Precompose Assumptions:** For the moment capacity of the non-composite beam, the program assumes the following:
   a. The beam is considered fully braced (Lb = 0) since the deck is perpendicular to the beam
   b. The girder is considered unbraced for the spacing of the filler beams (Lb = S) since the deck is parallel to the girder.
2. **Camber Assumptions:** Camber is calculated based on the self-weight only (construction DL and SDL not used for camber)
   a. The percentage of dead load (slab+steel) deflection for camber ..... Camber 80% of SW (typ 80-100%)
   b. The minimal camber (cannot camber less than this)..... Min of 3/4" Camber (3/4" typ)
3. **Loading Assumptions:** For the live, live load reduction, and super dead loads, consult the ASCE-7 05 code.
   a. The slab dead load is calculated based on Vulcraft tables (composite) ... Slab DL 42 psf (can't overwrite)
   b. The construction live load for calculating pre-composite strength..... CLL 10 psf (typ 20psf)
4. **Stud Assumptions:** Per ANSI/AWS D1.1-98 Chapter 7 Table 7.1, Type B shear stud connectors from ASTM A108 have Fu = 65k.si.
   a. The stud diameter is typically 3/4" ..... Stud Diameter 0.75 in (0.75 typ.)
   b. The minimum spacing of studs may vary btwn 18 to 36" ..... Min Stud Spacing 16" (24" typ.)
   c. The minimum percent composite action (typically 25% is fine) ... Min %Composite 35% (25% typ.)
   d. For girders w/ 2 point loads, uniform spacing or segmented?.. Uniform/ Segment? Uniform (uniform is typ.)
   e. Maximum Number of Studs per foot (within rib)..... Max Studs/ft 2.0 (1, 1.5, or 2 is typ.)
5. **General Assumptions:**
   a. The concrete strength is generally 3500psi or 4000psi..... f'c 3500 psi (3500-4000 typ.)
   b. Do you want a smaller database of steel sizes to scroll through? .... Max or Min? Max Steel Database
   c. Is the beam composite or non-composite with the concrete? ..... Condition Composite (can't overwrite)
   d. Shall the girder depth be greater than or equal to the beam? Depth Restriction? no (typ yes)
6. **Vibration Assumptions:** Vibration is based on the AISC Design Guide Series 11 (with errata publication if office occupancy)
   a. The vibration LL is typ. 11 psf for office loading (or 1"LL/f50)..... Vibr. Live Load 22 psf (11 typ)
   b. The vibration SDL is typ 4 psf for office loading (or 4*SDL/20)..... Vibr. SDL Load 10 psf (4 typ)
   c. The damping ratio beta is taken as 0.03 for office occupancy..... Damping Ratio b 0.03 (typ 0.03, DGI1 4.1)
   d. The acceleration limit is typically 0.05 for most occupancies..... Max a_p/g 0.50 %(0.50 typ.)
   e. The vibrational velocity limit for sensitive equipment (if not req'd, use 8000)..... V_t 4000 m in/sec (Tab 6.1)

3.3 W14X26 Beam Check



RESULT SUMMARY:

STRENGTH	COMPOSITE BEAM Partial w/ 26 Studs	COMPOSITE BEAM Full w/ 33 Studs	
Req'd Studs for Strength		212 k-ft	
M_u =	212 k-ft	241k-ft	
Composite Phi M_n =	212 k-ft	OK	
Strength OK?	OK		
DEFLECTION	Partial w/ 26 Studs	Full w/ 33 Studs	
SDL + LL Deflection =	0.70"	0.58"	Precom
SDL + LL Deflection =	L / 452	L / 553	
DL overlunder Cambered =	0.52"	0.52"	
TOTAL Deflection =	1.22"	1.09"	
TOTAL Deflection =	L / 261	L / 291	
Deflection OK?	OK	OK	
Deflection Increase Factor	0.00		
Req'd Studs for Deflection:	25.0		
Stud Pg F actor	1.000		
Studs for Drawing:	26		
SDL + LL Deflection =	0.70"		
SDL + LL Deflection =	L / 452		
TOTAL Deflection =	1.22"		
TOTAL Deflection =	L / 261		
	assumes L/360 for SDL + LL and L/240 for Total		

Stud range 20 to 26 - same capacity

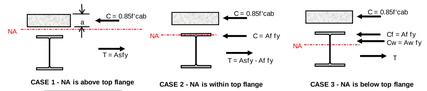
SHEAR			Typ Standard: Shears from using 1.5 or 2 times Max Uniform Load Table		
Shear Vu	32	k	Vu (LRFD) = 2 [4phiNm/y'L]=	33	k
Shear phi Vn	69	k OK	V (ASD)= 2 [4phiNm/y'L]=	21	k (for dwg only)
Shear OK?	OK				

TRANSFORMED PROPERTIES

Total depth of slab, t	5.25	in	Steel only - Ix (in^4)	245	in^4
A_c =	258.375	in^2	Sx (in^3)	35	in^3
E_c =	2158	ksi	n	13.4	
effective thickness of conc, te	3.25	for calcs only	b eff std	5.9	in
"gap" above top flange, g	2.00		y bar	14.50	in
			d/2	7.0	in
Shape	W14X26		d+g+te/2	17.5	in
Area, A (in^2)	7.69		Itr	876.24	in^4
Actual Depth, d (in)	13.9		Lower Bound bef	4.6	in
Thickness of Web, tw (in)	0.255		Lower Bound y bar	13.9	in
Width of Flange, bf (in)	5.030		Lower Bound Itr Method 1	826.5	in^4
Thickness of Flange, tf (in)	0.420		Y_{bna}	11.4	in
Area Factor	1.018		Itr Method 2	604.8	in^4
If case 1, a	1.171		I_{tr}	715.6	in^5

COMPOSITE FLEXURAL STRENGTH

	Area	Area x strength	Remaining	dist PNA	
Concrete	258.38	768.67	245.91	2.21	
Top Flange	2.15	77.41	0.00	0.00	
Web	3.39	122.03	0.00	0.00	
Bottom Flange	2.15	77.41	0.00	0.00	
Sum	1045.51		actual PNA=	2.21	from top
Sum/2	522.75		Plastic Nuetral Axis = Y bar =	16.94	from bott



CASE 1 - NA is above top flange | CASE 2 - NA is within top flange | CASE 3 - NA is below top flange

Case	1			
	Area above/below NA	Area x strength	dist	Mn
Concrete above NA	175.72	276.84	1.11	25.50
Concrete below NA	82.66	0.00	0.52	0.00
Top Flange above NA	0.00	0.00	0.00	0.00
Top Flange below NA	2.15	77.41	0.21	1.38
Web above NA	0.00	0.00	0.00	0.00
Web below NA	3.39	122.03	9.87	100.40
Bottom Flange below NA	2.15	77.41	16.73	107.92
			Mn =	268	k-ft
			phi Mn =	241	k-ft

STUDS

Stud Diameter	0.75	in (typically 3/4")	Ast	0.44	in^2
Full Hor Shear	276.8	k (above NA)	Qn1	19.2	k
			Rg	see below	
Number of Studs for Full	32.1				
PCC	77.9%	for strength	Rp	0.6	
Min Spacing of Studs	18.000		Qn2	17.2	k
Horiz Shear Req'd	215.7	for Strength	Qn	17.2	k
Horiz Shear min 1	152.2	based on stud spacing	As/sds/2	5.53	in^2
Horiz Shear min 2	96.9	on PCC min			
Actual Horiz Shear, H	215.7	k			
Actual PCC	77.9%	Percent Composite			
Min Number of Studs	25.0				
No of Studs for Strength	25.0				
Avg Rg Increase Factor	1.00				
Studs for Strength	25.0	(includes Rg) dwg only			

DEFLECTION

LOADS				
	w_{DL} =	0.41	k-ft	
	w_{LL+SDL} =	1.32	k-ft	
	COMPOSITE BEAM			NONCOMPOSITE
	Delta DL	0.66	in	Delta LL+SDL 2.06 in
	Delta LL+SDL	0.70	in	

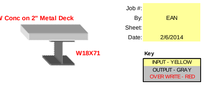
VIBRATION

LOADS			
Live Load	22.00	psf (11psf typ. for offices)	
SDL / Mech't Load	10.00	psf (4psf typ. for offices)	
w	708.28	plf	
Delta J	0.309	in	

$$f = 0.18\sqrt{\frac{g}{\Delta_j}}$$

Freq, f	6.36	Hz
n' = n*1.35	9.95	in^4/ft
Dx	7.7	in^4/ft
Dj	95.0	in^4/ft
Cj	2.00	(2 for int, 1 for edge per errata)
Eff Panel Width fo	28.3	ft
Check Min Entire Floor Width w >	42	ft
Wj	86	kips

3.4 W18X71 Girder Check



RESULT SUMMARY:

STRENGTH	COMPOSITE BEAM Partial w/ 28 Studs	COMPOSITE BEAM Full w/ 79 Studs		PRECOMPOSITE BEAM Unbraced Length=9.2ft	
Req'd Studs for Strength:			Parallel Deck:		
M_u =	490 k-ft	490 k-ft	Precomp Mu =	156 k-ft	
Composite Phi M_n =	500 k-ft	729 k-ft	Precomp Phi M_n =	376 k-ft	k-ft (Steel Only)
Strength OK?	OK	OK	Strength OK?	OK	
DEFLECTION	Partial w/ 28 Studs	Full w/ 79 Studs		Precomposite	
SDL + LL Deflection =	0.88"	0.60"	Precomp Deadload Deflection =	0.45"	
SDL + LL Deflection =	L / 375	L / 554	DL Deflection =	L / 739	
DL overlunder Cambered =	0.36"	0.36"	Deflection OK?	OK	assumes L/240
TOTAL Deflection =	1.24"	0.96"	Defl / Camber Percent =	80%	1/4"
TOTAL Deflection =	L / 267	L / 346	Min Camber =	0.75	
Deflection OK?	OK	OK		NONCOMPOSITE BEAM	
Deflection Increase Factor	0.00		Perpendicular Deck:	Assumes Fully Braced	
Req'd Studs for Deflection:	27.3				
Studs for Drawing:	38	(includes moment and Rg ft			
Studs for Drawing:	14,10,14				
SDL + LL Deflection =	0.88"				
SDL + LL Deflection =	L / 375				
TOTAL Deflection =	1.24"				
TOTAL Deflection =	L / 267				
	assumes L/360 for SDL + LL and L/240 for Total				

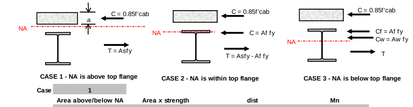
SHEAR			Odeh Standard: Shears from using 1.5 or 2 times Max Uniform Load Table		
Shear Vu	53	k	Vu (LRFD) = 2 [4phiNm/y'L]=	114	k
Shear phi Vn	178	k OK	V (ASD)= 2 [4phiNm/y'L]=	73	k (for dwg only)
Shear OK?	OK				

TRANSFORMED PROPERTIES

Total depth of slab, t	5.25	in	Steel only - Ix (in^4)	1170	in^4
A_c =	352.665	in^2	Sx (in^3)	127	in^3
E_c =	2158	ksi	n	13.4	
effective thickness of conc, te	4.25	for calcs only	b eff std	6.2	
"gap" above top flange, g	1.00		y bar	16.2	
			d/2	9.3	
Shape	W18X71		d+g+te/2	21.6	
Area, A (in^2)	20.80		Itr	2986.6	
Actual Depth, d (in)	18.5		Lower Bound bef	2.2	in
Thickness of Web, tw (in)	0.495		Lower Bound y bar	13.0	in
Width of Flange, bf (in)	7.640		Lower Bound Itr Method 1	2159.7	in^4
Thickness of Flange, tf (in)	0.810		Y_{bna}	12.0	in
Area Factor	1.003		Itr Method 2	1886.0	in^4
If case 1, a	3.033		I_{tr}	2022.8	in^5

COMPOSITE FLEXURAL STRENGTH

	Area	Area x strength	Remaining	dist PNA	
Concrete	352.67	1049.18	150.19	3.64	
Top Flange	6.21	223.51	0.00	0.00	
Web	8.38	301.78	0.00	0.00	
Bottom Flange	6.21	223.51	0.00	0.00	
Sum	1797.98		actual PNA=	3.64	from top
Sum/2	898.99		Plastic Nuetral Axis = Y bar =	20.11	from bott



CASE 1 - NA is above top flange | CASE 2 - NA is within top flange | CASE 3 - NA is below top flange

Case	3			
	Area above/below NA	Area x strength	dist	Mn
Concrete above NA	302.18	748.80	1.82	113.62
Concrete below NA	50.48	0.00	0.30	0.00
Top Flange above NA	0.00	0.00	0.00	0.00
Top Flange below NA	6.21	223.51	0.41	7.57
Web above NA	0.00	0.00	0.00	0.00
Web below NA	8.38	301.78	10.83	272.38
Bottom Flange below NA	6.21	223.51	19.70	366.99
			Mn =	810	k-ft
			phi Mn =	729	k-ft

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PROJECT TITLE

950 MASSACHUSETTS AVE
CAMBRIDGE, MA 02139

DESIGNER'S SEAL

SHEET DESCRIPTION

STRUCTURAL ASSESSMENT

DESIGN: T. SANCHEZ
REVIEW: B. SHEA
DATE: 3.30.15
SCALE: N.T.S

PROJECT NUMBER

RGR-3.2

STRUCTURAL ASSESSMENT

Project: 950 Mass Ave
Re: Roof Reserve Capacity for Additional Loads - Rev 3
Date: 2/6/2014
By: EAN / EPT

STUDS

Stud Diameter	0.75	in (typically 3/4")	Ast	0.44	in^2
Full Horiz Shear	748.8	k	Qn1	19.2	k
			Rg	see below	
Number of Studs for Full PCC	78.0		Rp	0.75	
	0.323				
Min Spacing of Studs	18.000		Qn2	21.5	k
Horiz Shear Req'd	242.1	for Strength	Qn	19.2	k
Horiz Shear Min 1	177.0	based on stud spacing	Astuds/2	6.03	in^2
Horiz Shear min 2	262.1	on PCC min			
Actual Horiz Shear	262.1	k			
Actual PCC	35.0%	Percent Composite			
Studs for Strength	25.2				
Studs for Min spac	27.3				
Studs for Deflection	27.3	see above			
Min Number of Studs	27.3				
Uniform or Segmented?	**Uniform**				

DEFLECTION

Delta DL	0.45		NONCOMPOSITE
Delta LL+SDL	0.88	Delta LL+SDL	1.51

GIRDER VIBRATION

LOADS

Live Load	22.00	psf (11psf typ. for offices)
SDL / Mech'l Load	10.00	psf (4psf typ. for offices)
Slab + Steel DL	47.50	psf
Total	79.50	psf
Delta g	0.32	in

$$f = 0.18\sqrt{\frac{g}{\Delta_j}}$$

Girder Freq, f	**6.25**	**Hz**
Dj	96.04	in^4/ft (from beam spreadsheet)
Dg	112.70	in^4/ft
Cg	1.80	in^4/ft
Eff Panel Width Bg	47.71	ft
Check Min Floor Width w >	72	ft
w_c	2107	plf
W_g	104.91	kips

Project: 950 Mass Ave
Re: Roof Reserve Capacity for Additional Loads - Rev 3
Date: 2/6/2014
By: EAN / EPT

FLOOR VIBRATION

	Girder Span	27.66	ft
From Beam Sheet	Joist Panel Width	28.29	ft
	Reduction factor	0.98	
	Delta g'	0.31	in
	Delta j	0.31	
Equation 3.4	**Floor Freq, fn**	**4.48**	**Hz**
	Wj	86.38	
Equation 4.4	Panel Weight	95.71	kips
Table 4.1	Beta	0.03	
	Beta W	2.87	kips
	Po	65.00	lbs
Equation 4.1	a1/g	0.0047	
	a1/g	**0.47%**	**of gravity**
Midspan Flexibilities	Delta OJ	1.32E-05	in/lb
	Delta GP	4.40E-06	in/lb
	dp	4.25	in
Equation 4.7	Neff	1.90	
	Mid-bay Flexibility = Delta P	9.12E-06	in/lb

For 185lb person walking	Fast (100 steps/min)	Moderate (75 steps/min)	Slow (50 steps/min)	
Fm	315	278	241	lb
Purse Rise Freq, fo	5.0	2.5	1.4	*
fn/fo = fn/fo =	0.90	1.79	3.20	
Am	0.622	0.156	0.049	
Xmax	0.001785	0.000394	0.000107	in
Uv	25000	5600	1500	lb-Hz^2
Vmax if fn/fo >= 0.5	0.050888	0.011195	0.003053	in/sec
Vmax if fn/fo >= 0.5	50888	11195	3053	micro-in/sec
V	0.050279	0.011091	0.003014	in/sec
V	**50279**	**11091**	**3014**	micro-in/sec

Table 6.2

Figure 6.5 pg 48

*(note 1.2 change to 1.4 per DG3 11 Errata)

Project: 950 Mass Ave
Re: Roof Reserve Capacity for Additional Loads - Rev 3
Date: 2/6/2014
By: EAN / EPT

3.5 Concrete Deck

SLAB INFORMATION

Slab Depth	Total Conc. Volume Yds./ 100 Sq. Ft.	Conc. Volume Cu. Ft./ Sq. Ft.	Recommended Welded Wire Fabric
4"	0.94	0.253	6x6-W1.4xW1.4
4 1/2"	1.09	0.294	6x6-W1.4xW1.4
5"	1.24	0.336	6x6-W1.4xW1.4
5 1/4"	1.32	0.357	6x6-W1.4xW1.4
5 1/2"	1.40	0.378	6x6-W2.1xW2.1
6"	1.55	0.419	6x6-W2.1xW2.1
6 1/4"	1.63	0.440	6x6-W2.1xW2.1
6 1/2"	1.71	0.461	6x6-W2.1xW2.1

(N=14) LIGHTWEIGHT CONCRETE (110 PCF)

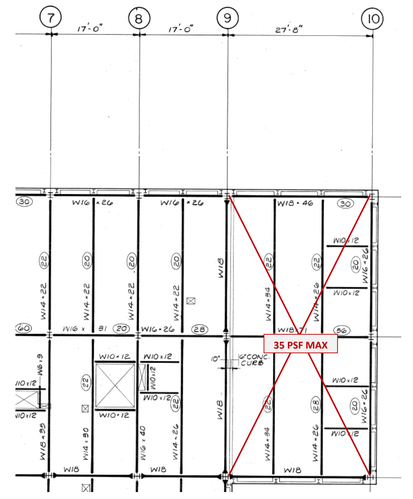
Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span 1 Span	2 Span	3 Span	Superimposed Live Load, PSF Clear Span (ft.-in.) 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"	13'-0"
4"	2VL22	7'-2	9'-6	9'-6	238	209	185	140	133	120	108	98	90	82	75	69	64	59	55
	2VLC1	7'-10	10'-2	10'-6	254	223	198	178	142	128	115	105	96	87	80	74	69	63	58
	2VL20	8'-5	10'-9	11'-1	268	238	209	187	169	138	122	110	101	92	84	78	72	66	61
(N=2")	2VL19	9'-6	11'-11	12'-4	297	260	230	206	185	168	153	141	111	101	93	86	79	73	68
	2VL18	10'-6	12'-10	13'-3	324	285	253	227	205	187	171	156	146	136	127	99	92	86	80
20 PSF	2VL17	11'-5	13'-8	14'-0	352	308	273	245	221	201	184	169	156	145	135	107	99	92	86
	2VL16	12'-1	14'-4	14'-4	377	330	292	261	235	214	195	179	165	153	143	133	118	88	81
	2VL22	6'-8	9'-1	9'-3	276	243	195	173	155	139	126	114	104	96	88	81	75	69	64
4 1/2"	2VLC1	7'-5	9'-8	10'-1	295	259	231	185	165	149	134	122	111	102	93	86	79	73	68
	2VL20	8'-0	10'-4	10'-8	312	273	243	217	196	157	141	128	117	107	98	90	84	77	72
(N= 2 1/2")	2VL19	9'-0	11'-5	11'-9	346	302	268	239	215	195	178	142	129	118	108	100	92	85	79
	2VL18	10'-0	12'-3	12'-8	376	331	294	264	238	217	199	183	170	156	145	135	126	117	109
35 PSF	2VL17	10'-10	13'-1	13'-6	400	358	318	284	256	233	213	196	181	168	134	124	115	107	100
	2VL16	11'-5	13'-8	13'-10	400	384	340	303	273	248	227	209	193	179	166	154	123	114	106
	2VL22	6'-6	8'-8	8'-10	315	277	222	197	176	159	144	130	119	109	100	92	85	79	73
5"	2VLC1	7'-1	9'-4	9'-8	337	296	263	211	189	169	153	139	127	116	107	98	91	84	78
	2VL20	7'-7	9'-11	10'-3	355	312	278	249	199	179	161	146	133	122	112	103	95	88	82
(N=2")	2VL19	8'-7	10'-11	11'-4	394	345	307	272	245	223	178	162	147	135	124	114	105	97	90
	2VL18	9'-6	11'-10	12'-2	400	377	335	300	272	247	227	209	168	156	143	132	122	114	106
40 PSF	2VL17	10'-3	12'-7	13'-0	400	400	362	324	292	266	243	223	207	166	153	142	131	122	114
	2VL16	10'-11	13'-2	13'-6	400	400	387	346	311	283	258	237	219	203	163	151	140	130	121
	2VL22	6'-4	8'-6	8'-8	334	296	267	187	168	152	138	126	116	106	98	90	84	78	
5 1/4"	2VL21	7'-0	9'-2	9'-5	357	314	279	224	200	180	163	148	135	123	113	104	96	89	83
	2VL20	7'-5	9'-8	10'-0	377	331	283	211	189	171	155	142	130	119	110	101	94	87	
(N= 3 1/4")	2VL19	8'-5	10'-9	11'-1	400	366	324	289	260	216	196	179	164	151	140	130	121	113	105
	2VL18	9'-3	11'-7	12'-0	400	400	365	319	288	263	241	195	179	164	151	140	130	121	113
42 PSF	2VL17	10'-1	12'-4	12'-9	400	400	384	344	310	282	258	237	219	177	163	151	140	130	121
	2VL16	10'-8	12'-11	13'-3	400	400	400	367	330	300	274	252	232	215	173	160	148	138	128
	2VL22	6'-3	8'-5	8'-6	353	314	282	222	198	178	161	147	134	122	113	104	96	89	82
5 1/2"	2VLC1	6'-10	9'-0	9'-4	378	333	298	237	212	190	172	156	142	130	120	110	102	94	87
	2VL20	7'-4	9'-6	9'-10	399	350	310	250	223	201	181	165	150	137	126	116	107	99	92
(N= 3 1/2")	2VL19	8'-3	10'-6	10'-11	400	387	342	306	275	222	200	182	165	151	139	128	118	109	101
	2VL18	9'-1	11'-4	11'-8	400	400	378	337	305	278	254	200	189	174	160	148	136	126	119
44 PSF	2VL17	9'-10	12'-1	12'-6	400	400	400	363	328	298	273	251	204	187	172	159	148	137	128
	2VL16	10'-5	12'-8	13'-1	400	400	400	386	350	317	290	266	246	199	184	170	157	146	136
	2VL22	5'-11	7'-10	8'-0	380	331	291	256	231	208	188	171	156	143	131	121	112	103	96
5 1/4"	2VL21	6'-5	8'-7	8'-10	400	385	312	276	247	222	200	182	166	152	140	129	119	110	102
	2VL20	6'-11	9'-1	9'-4	400	400	329	292	260	234	211	192	175	160	147	135	125	115	107
(N= 4 1/4")	2VL19	7'-10	10'-0	10'-4	400	400	388	350	288	259	233	212	193	176	162	149	137	127	118
	2VL18	8'-7	10'-10	11'-2	400	400	400	360	326	293	264	240	220	202	187	173	160	149	139
51 PSF	2VL17	9'-3	11'-6	11'-11	400	400	400	400	381	347	317	259	237	218	201	188	172	160	149
	2VL16	9'-10	12'-1	12'-6	400	400	400	400	400	400	369	337	310	253	232	214	198	183	170

Live + SDL Capacity of Deck = 163 psf - does not control reserve capacity. SDL reserve= 163-100-5 = 58 psf Deck allows 58psf, but beams only 35psf

Project: 950 Mass Ave
Re: Roof Reserve Capacity for Additional Loads - Rev 3
Date: 2/6/2014
By: EAN / EPT

4.0 CONCLUSION

4.1 OPTION 1: Uniform Reserve Capacity for Roof Decking/Planters/Etc



Note this 35psf is allowed above the 115psf psf code loads. The total unfactored load = 35psf reserve + 100psf live/snow + 15psf roofing/insulation/ceiling + 42 psf conc/MD + 6 psf steel self weight = 198 psf

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PROJECT TITLE

950 MASSACHUSETTS AVE
CAMBRIDGE, MA 02139

DESIGNER'S SEAL

SHEET DESCRIPTION

STRUCTURAL ASSESSMENT

DESIGN: T. SANCHEZ
REVIEW: B. SHEA
DATE: 3.30.15
SCALE: N.T.S

PROJECT NUMBER

RGR-3.3

7 OF 7

STRUCTURAL ASSESSMENT