The City College Of New York
Grove School of Engineering

Civil Engineering Department

Mechanics of Deformable Bodies

New Jersey Transit Authority
Beam Design Project

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CONTRACT NO. 13–006

NJ TRANSIT
BAY HEAD YARD SUBSTATION
30% SUBMITTAL
BOROUGH OF BAY HEAD, OCEAN COUNTY, N.J.

Table of Content:

Abstract

Our company has been hired by the New Jersey Transit Authority to provide engineering services for their new building project which is located in Borough of Bay Head, Ocean County, NJ. The following report contains calculations regarding the design of specific portions of the New Jersey Transit Authority's new Substation Service Building design project.

As specified by the client, the flexural design, maximum deflection limit, revised flexural design, lightest W-shaped column, max footing size and revised calculations for all portions of the design are included. As per the request of the client, we have also considered the inclusion of a generator on the roof in our calculations. Explanations are also included along with the calculations in the report so that the client can have a better understanding of the design project.

This report is the next step to the 30% design package submitted and fulfills all of the client's requirements. Our engineers went through all of the questions and have provided answers with detailed analysis and design calculations.

Known: For lightest W-Shapes on 1st floor.

- The factor of Safety = 2.0 → $\frac{\sigma_{ult}}{\sigma_{all.}}$
- Ignore deflection
- USE "1st Floor Drawings" (S-102) for Beams 1 and 2
- USE "Roof Drawings" (S-104) for Beams 3 and 4

— Location of Beam 1: Electrical Room. (See figure 1)

- Based on Beam Type table: Rolled Shapes: ASTM A922
  → Stress: $\sigma_y = 50000$ psi

— Knowing total load; by using General Notes (S-001):

- Design loads for electrical room: Dead load = $75 \frac{lb}{ft^2}$, Live load = $100 \frac{lb}{ft^2}$ } $175 \frac{lb}{ft^2}$ total

$L_{Tot} = L_D + L_L$

Load total = $\left(175 \frac{lb}{ft^2}\right)\left(\frac{1 ft}{12 in}\right)^2 = \underline{1.2152 \text{ psi}}$

- The weight of the concrete itself:

If ρ of concrete is $= \left(150 \frac{lb}{ft^3}\right)\left(\frac{1 ft}{12 in}\right)^3 = 0.08680 \frac{lb}{in^3}$ ✓

then, the weight for slab with 6" thickness:

$\left(0.08680 \frac{lb}{in^3}\right)(6\ in) = 0.52083 \frac{lb}{in^2}$ ✓

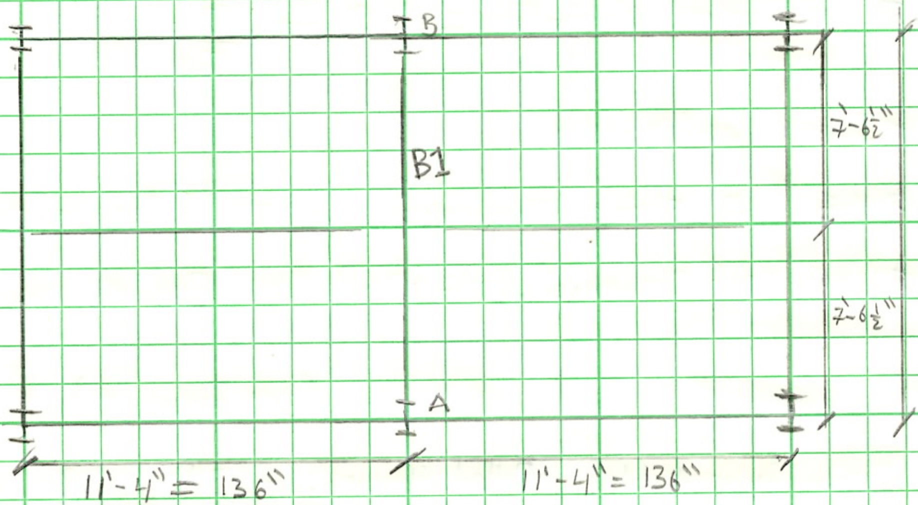
$L_{Tot} = 1.2152\ lb/in^2$ and $L_{self} = 0.52083\ lb/in^2$

— Distributed Load: $W = L_{Tot} + L_{self} = (L_{Tot} + L_{self})$ (Length of section) at B1

- See figure 1 for more details: length of section for B1: 11'-4" = 136"
- $W = (1.2152 + 0.52083) \frac{lb}{in^2} (136\ in) = \boxed{236.100 \frac{lb}{in.} = W}$

— Allowable stress: since $SoF = 2.0 = \frac{\sigma_{ult}}{\sigma_{all}} \rightarrow \sigma_{all} = \frac{50000 \text{ psi}}{2.0} = 25$ ksi

Figure 1: Beam 1 (NTS)



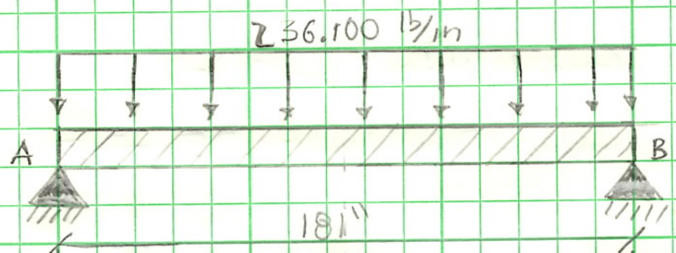


Figure 2: Load Distribution on Beam 1

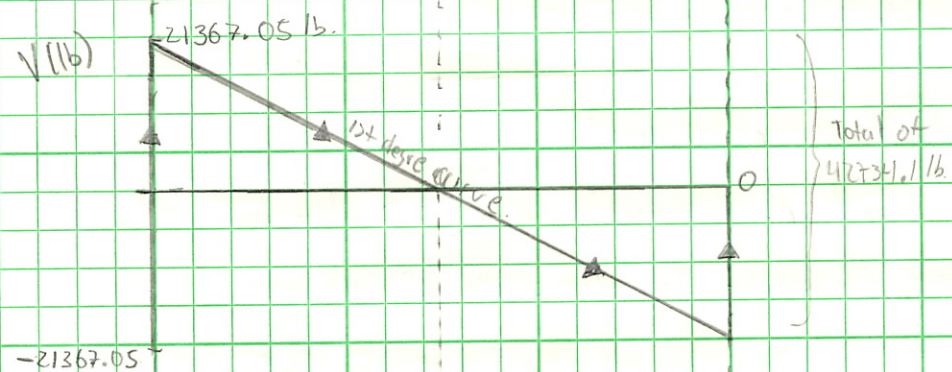


Figure 3:

Shear Diagram.

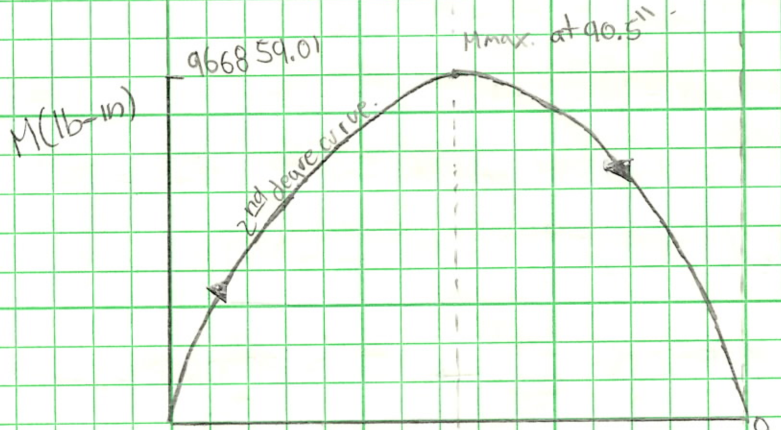


Figure 4:

Moment Diagram.

- Since our W is Distributed along the Beam (B1). (see figure 2).

FBD:

Distributed load becomes: (236.100 lb/in)(181 in) = 42734.1 lb. at the center

- To Find Reactions: Use Basic Statics Knowledge.

  • Because of symetry: $\frac{42734.1\text{ lb}}{2} = 21367.05\text{ lb} = R_A = R_B$.

OR

$\Sigma M_A = 0 \rightarrow R_B(181\text{ in}) - (42734.1\text{ lb})(90.5\text{ in}) = 21367.05 = R_B$

$\Sigma F_y = 0 \rightarrow R_A + R_B = 42734.1\text{ lb} \rightarrow R_A = 21367.05\text{ lb}.$

- From Reactions: we know $V_{max} = 21367.05$ lb since R_A is pointing upward

  • For Shear Diagram, since the Beam only experiences an evenly distributed load, then our graph will look linear with negative slope until it reaches R_B. which then brings it back to zero.

  (see figure 3)

- In order to graph out Moment Diagram:

  • Max Moment: $\frac{(V_{max})\left(\frac{181}{2}\right)}{2}$ (since its at the center) =

  $M_{max} = \frac{(21367.05\text{ lb})(90.5\text{ in})}{2} = 966859.01$ lb·in

  • Because of symetry: Total moment = 0 when we reach to 181"

  (see figure 4)

- We can obtain our section Modulus. by using equation: $\frac{I}{c} \geq \frac{M_{max}}{\sigma_{all}}$

But, $\frac{I}{c} = S$ so $\rightarrow S_x = \frac{M_{max}}{\sigma_{all}}$. $S_x = \frac{966859.01\text{ lb·in}}{\left(50000\text{ lb/in}^2 / 2\right)} = 38.674\text{in}.$

- The Beam options based on our S_x, and Based on the W-table: (Choose slightly bigger S_x).

  (see table 1.1).

Table 1.1 - Beam Options.

Designation #	Ix (in⁴)	Sx (in³)	d(in)
W8X48	184	43.3	8.5
W10X39	209	42.1	9.92
W12x35	285	45.6	12.5
W14x30	291	42.0	13.8

- Since Beam 2 is also located in the Electrical Room,

  The total load $= 175\ lb/ft^2 = 1.2152\ lb/in^2$

- We know from B1 that: • $\sigma_{yield} = 50{,}000\ psi$ • $\sigma_{all} = 25\ ksi$

  Load 1 $= T_L$ (length of section) $= (1.215\ psi)(136\ in) = 165.280\ lb/in$

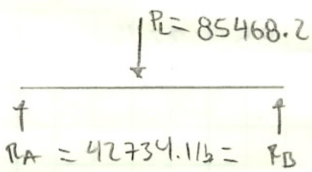
  Load 2 $= (0.5208\ psi)(136\ in) = 70.833\ lb/in.$

  $W = L_1 + L_2 = 236.100\ lb/in.$

- $P_L = (W)(\text{Span length}) = (236.100\ lb/in)(181\ in + 181\ in) = \underline{85468.2\ lb}$

- If we want to find the reactions: (see figure 5).

FBD



Because of symetry:

$R_A = R_B$ or we can use our statics knowledge:

$R_A = R_B\ \frac{1}{2}(85468.2\ lb) = \underline{42734.1\ lb.}$

- For our shear diagram: (see figure 6).

  • $R_A = V_{max} = 42734.1\ lb\ (\uparrow)$

- For our Moment diagram (see figure 7).

  • $M_{max} = \frac{V_{max}(181\ in)}{2} = \frac{(42734.1\ lb)(181\ in)}{2} = 3867436.05\ lb.in.$

- For our Section Modulus: $S_x = \frac{M_{max}}{\sigma_x} = \frac{3867436.05\ lb.in}{\left(\frac{50000\ lb/in^2}{2}\right)} =$

  $\sigma_x \rightarrow \sigma_{all}$

  $S_x = 154.691\ in^3.$

- We want to find Beam Options: (see table 2.1).

Figure 5: load Distribution on Beam 2.

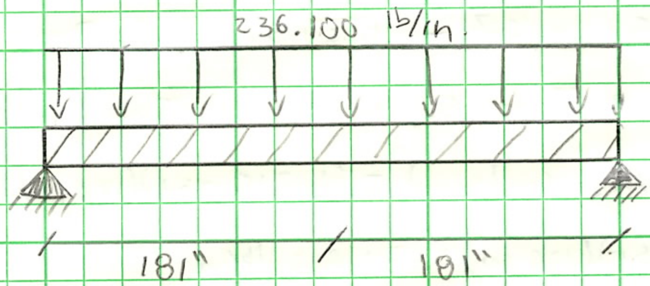


Figure 6: Shear Diagram for Beam 2.

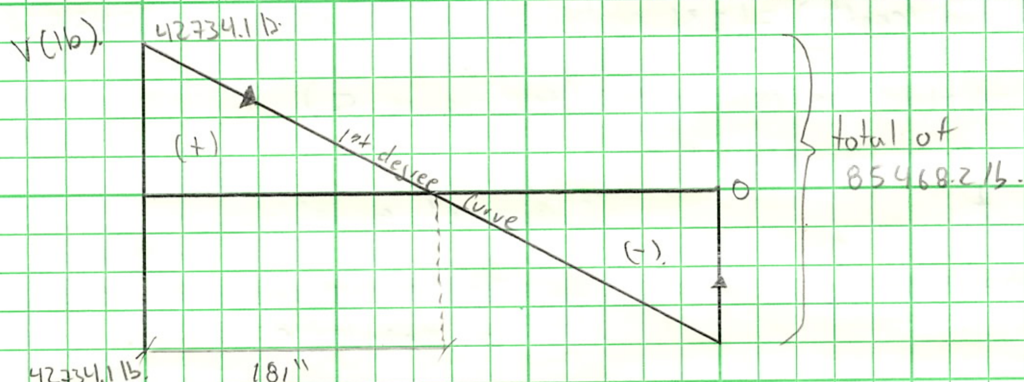


Figure 7: Moment Diagram of Beam 2.

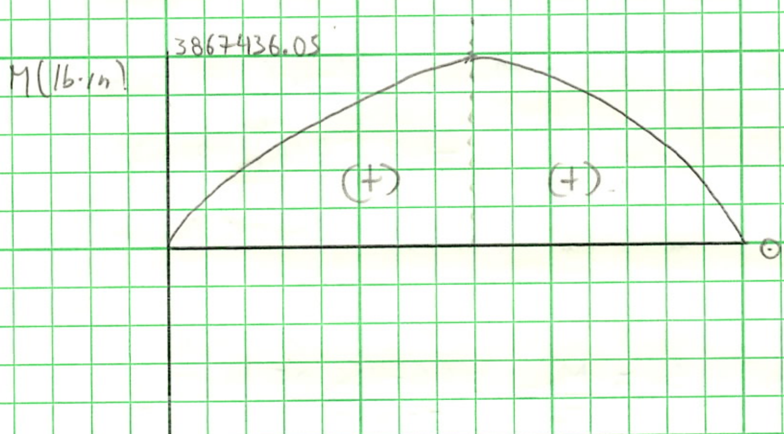


Table 2.1 - Beam options.

Designation #	I_x (in^4)	Sx (in^3)	d (in)
W24X68	1830	154	23.7
W16X89	1300	155	16.75
W14X99	1110	157	14.16
W18X86	1530	166.	18.4
W21X83	1830	171.	21.4

$\sigma_y = 50{,}000$ psi

$\sigma_{all} = 25{,}000$ psi

Loads : Dead $= 25\ \text{lb/ft}^2$

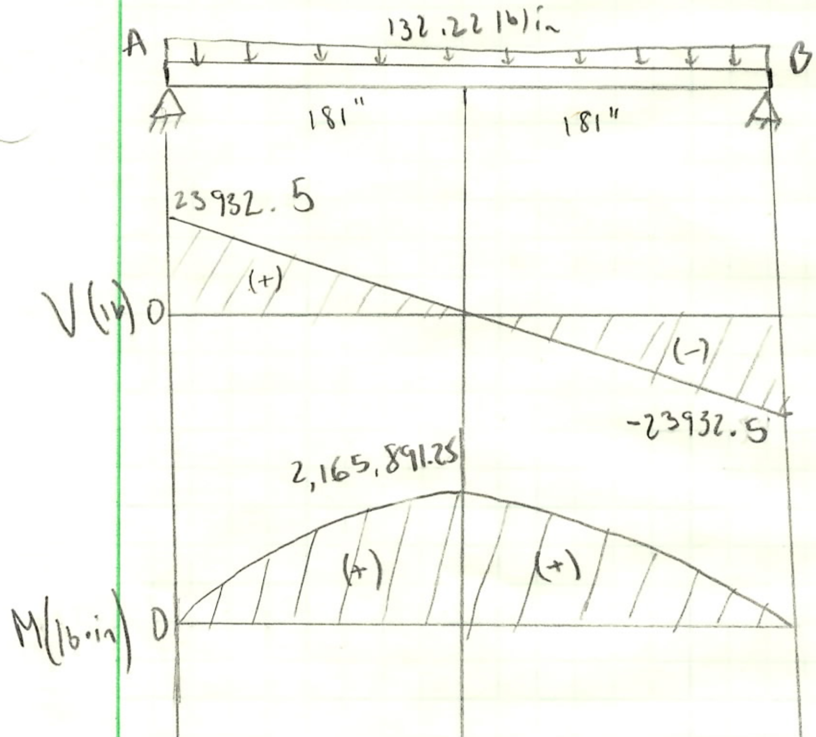
Live $= 40\ \text{lb/ft}^2$

Total $= 65\ \text{lb/ft}^2 \left(\frac{1\ \text{ft}^2}{144\ \text{in}^2}\right) = 0.4514$ psi

$w_1 = (0.4514)(136) = 61.39\ \text{lb/in}$

$w_2 = \left(150\ \text{lb/ft}^3\right)\left(\frac{1\ \text{ft}^3}{1728\ \text{in}^3}\right)(6\ \text{in})(136\ \text{in}) = 70.83\ \text{lb/in}$

$w = w_1 + w_2 = 61.39 + 70.83 = 132.223\ \text{lb/in}$



$P_L = (L)(w) = (362)(132.224)$

$P_L = 47865\ \text{lb}$

$R_A = R_B = \frac{P_L}{2}$

$R_{A,B} = \frac{47865}{2} = 23932.5\ \text{lb}$

$M_{max} = \frac{(V_{max})(181)}{2}$

$M_{max} = \frac{(23932.5)(181)}{2}$

$= 2165891.25$

$S_x = \frac{M_{max}}{\sigma_x} = \frac{2165891.25}{25000}$

$S_x = 86.636\ \text{in}^3$

Table 3.1 - Beam Options

Designation	I_x (in^4)	S_x (in^3)	d (in)
W12x72	597	97.4	12.3
W16x57	758	92.2	16.4
W18x50	800	88.9	18.0
W14x68	722	103	14.0

Total load = 0.4514 lb/in^2

Concrete Density = 150 lb/ft^3

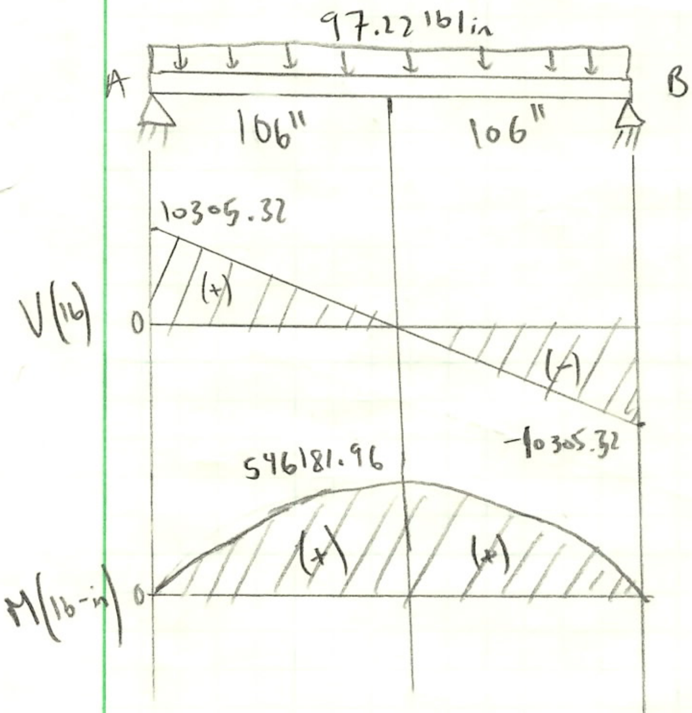
σ_{all} = 25 ksi

Distributed loads:

$w_1 = (0.4514)(100) = 45.139\ lb/in$

$w_2 = (150\ lb/ft^3)\left(\frac{1\ ft^3}{1728\ in^3}\right)(6\ in)(100\ in) = 52.0833\ lb/in$

$w = w_1 + w_2 = 45.139 + 52.0833 = 97.22\ lb/in$



$P_L = (L)(w) = (212)(97.22)$

$P_L = 20,610.6\ lb$

$R_A = R_B = \frac{P_L}{2}$

$R_{A,B} = \frac{20610.6}{2} = 10305.32\ lb$

$M_{max} = \frac{(V_{max})(106)}{2}$

$M_{max} = \frac{(10305.32)(106)}{2}$

$M_{max} = 546181.96\ lb\text{-}in$

$S_x = \frac{M_{max}}{\sigma_x} = \frac{546181.96}{25000} = 21.84\ in^3$

Table 4.1 – Beam options

Designations	I_x (in^4)	S_x (in^3)	d (in)
W14x22	199	29	13.7
W12x22	156	25.4	12.3
W10x22	118	23.2	10.2
W8x28	98	24.3	8.06

$$\text{Live load} = \left(40 \frac{lb}{ft^2}\right)\left(\frac{1 ft^2}{144 in^2}\right)(136 in) = 37.78 \, lb/in$$

$$\frac{L}{360} = \frac{362}{360} = 1.0056$$

$$\delta_{max} = \frac{5 w L^4}{384 E I}$$

(1) W12X72

$$\delta_{max} = \frac{5(37.78)(362)^4}{(384)(29 \times 10^6)(597)} = 0.488 < 1.0056 \quad \underline{\text{Pass}}$$

(2) W16 x 57

$$\delta_{max} = \frac{5(37.78)(362)^4}{(384)(29 \times 10^6)(758)} = 0.3843 < 1.0056 \quad \underline{\text{Pass}}$$

(3) W16 x 50

$$\delta_{max} = \frac{5(37.78)(362)^4}{(384)(29 \times 10^6)(800)} = 0.364 < 1.0056 \quad \underline{\text{Pass}}$$

(4) W14 x 68

$$\delta_{max} = \frac{5(37.78)(362)^4}{(384)(29 \times 10^6)(722)} = 0.4034 < 1.0056 \quad \underline{\text{Pass}}$$

☆ We know from our Flexural Design Calculations that:

- The total load = $W = W_1 + W_2 = 97.22$ lb/in. ☆ But we want Live!
- Live load = $\left(40 \frac{lb}{ft^2}\right)\left(\frac{1 ft}{12 in}\right)^2 (100 in) = 27.7778$ lb/in
- Not use Dead Load
- $E = 29 \times 10^6$ psi

☆ If a max deflexion limit of $\frac{L}{360}$ under live loads.

- Live load $= \frac{L}{360} = \frac{181''}{360} = 0.5027$

☆ Use $\frac{5wL^4}{384EI}$ (chart from book).

☆ Using our Beam options: Our $S_x = 21.84 in^3$.

- 1st Beam: W14 X 22 : $I_x (in^4) = 199$, $S_x (in^3) = 29$.

Live load: $\frac{5(Load)(181 in)^4}{384(E)(I_x)} = \frac{5(27.7778\ lb/in)(181 in)^4}{384(29 \times 10^6\ lb/in^2)(199\ in^4)} = 0.06723$

$0.06723 < 0.5027$ (Good)

- W12X22 : $I_x(in^4) = 156$, $S_x(in^3) = 25.4$

Live load: $\frac{5(27.7778\ lb/in)(181\ in)^4}{384(29 \times 10^6\ lb/in^2)(156\ in^4)} = 0.0858 < 0.5027$ (Good)

- W10X22 : $I_x(in^4) = 118$, $S_x(in^3) = 23.2$

Live load: $\frac{5(27.7778\ lb/in)(181\ in)^4}{384(29 \times 10^6\ lb/in^2)(118\ in^4)} = 0.11344 < 0.5027$ (Good).

- W8X28 : $I_x(in^4) = 98$, $S_x(in^3) = 24.3$.

Live load: $\frac{5(27.7778\ lb/in)(181\ in)^4}{384(29 \times 10^6\ lb/in^2)(98\ in^4)} = 0.13659 < 0.5027$ (Good).

We use our previous tables. (table 3.1).

B3

$$\frac{L}{d} = 16 \Rightarrow d = \frac{L}{16} = \frac{362}{16} = 22.625 \text{ in}$$

① W12×72

$d = 12.3 < 22.625$ fail

② W16×72

$d = 16.4 < 22.625$ fail

③ W18 × 50

$d = 18 < 22.625$ fail

④ W14×68

$d = 14 < 22.625$ fail

New choice : W24×68

$$\delta_{max} = \frac{5(37.78)(362)^4}{(384)(29\times10^6)(1830)} = 0.1592 < 1.005 \text{ Pass}$$

$d = 23.7 > 22.625$ Pass

☆ Use table 4.1 with "d" values.

B4

$$\frac{L}{d} = 16 \Rightarrow d = \frac{L}{16} = \frac{212}{16} = 13.25 \text{ in}$$

(1) W14 x 22

$d = 13.7 > 13.25$ Pass

(2) W 12 x 22

$d = 12.3 < 13.25$ fail

(3) W10 x 22

$d = 10.2 < 13.25$ fail

(4) W 8 x 28

$d = 8.06 < 13.25$ Pass

☆ Under axial loads, ignore deflection limits in previous questions and no Buckling. ☆ eccentricity of 12''. ☆ SoF = 2.0.

☆ $\sigma_{all} = \frac{50000 \text{ psi}}{2.0} = \frac{\sigma}{SF} = 25 \text{ ksi}$

☆ From our flexural design calculations: we said that: the load distribution:

$P_R = R_A = R_B = 21367.05 \text{ lb}$, load applied then is $2 \times R_A$.

$= 42734.1 \text{ lb}$.

☆ For the column conditions:

- Total load = Live load + Dead load = W_1

$$= (75 \text{ lb/ft}^2) + (100 \text{ lb/ft}^2) = 175 \text{ lb/ft}^2 \left(\frac{1 \text{ ft}^2}{144 \text{ in}^2}\right) = 1.2152 \frac{\text{lb}}{\text{in}^2}$$

$$W_1 = (1.2152 \text{ lb/in}^2)(90.5 \text{ in}) = 109.985 \text{ lb/in}.$$

- Concrete: W_2 = (Density concrete)(thickness of slab) x (length of col.) =

$$(150 \text{ lb/ft}^3)(6 \text{ in})(90.5 \text{ in})\left(\frac{1 \text{ ft}}{12 \text{ in}}\right)^3 = 47.1354 \frac{\text{lb}}{\text{in}}$$

(see figure 8). for reference.

$$W_T = 109.985 + 47.1357 = 157.120 \text{ lb}.$$

- From Distributed load:

- $P_L = W(\text{Span length}) = (157.12 \frac{\text{lb}}{\text{in}})(136 \text{ in}) = 21,368.32 \text{ lb}$.

- $P_{Tot} = P_L + P_R$

$P_{tot} = 21368.32 \text{ lb} + 21367.05 \text{ lb} = 42,735.37 \text{ lb}$.

- Reactions from a total Force P: $R_A = R_B = \frac{21,368.32 \text{ lb}}{2} = 10684.16 \text{ lb}$.

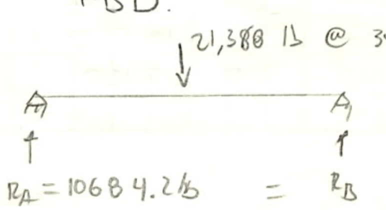
- $R_A = V_{max}$. - $M_{max} = 363261.1 \text{ lb·in}$.

(see figure 9) for distributed load.

- Shear Diagram (figure 10).

- Moment Diagram (figure 11).

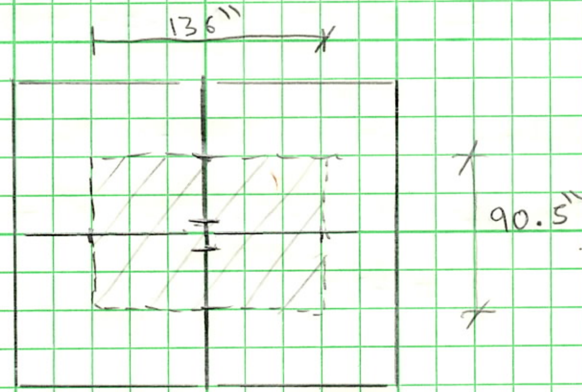
FBD.



☆ $S_y = \frac{M}{\sigma_{all}} = \frac{(P_{tot})(12 \text{ in})}{25000 \frac{\text{lb}}{\text{in}^2}} = \frac{(42735.35 \text{ lb})(12 \text{ in})}{25000 \text{ lb/in}^2} = 25.64\text{d in}^3$.

Figure 8: Column

Problem states eccentricity of 12"



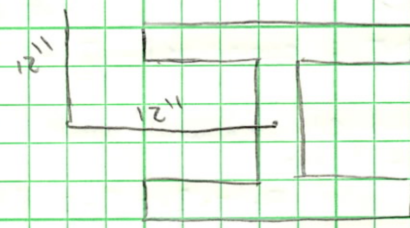


Figure 9: Distributed load.

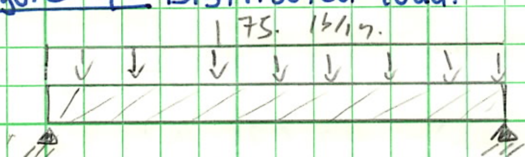


Figure 10: Shear Diagram

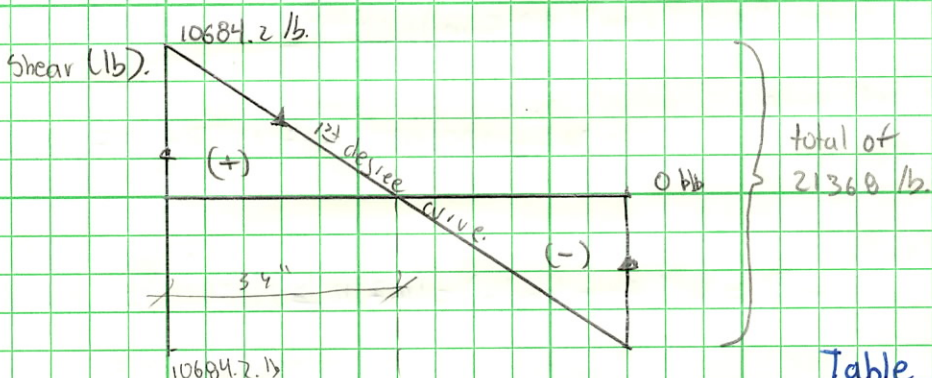


Figure 11: Moment Diagram

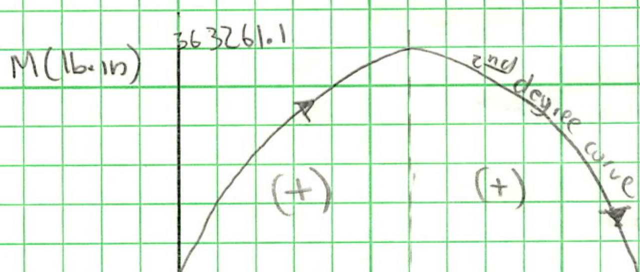


Table 4.1:

Designation #	I_x (in^4)	I_y (in^4)	S_y (in^3)
W16X77	1106	138	26.9
W14X74	796	134	26.6
W10X68	394	134	26.4

d(in)	Area	Width (in)
16.77	22.6	10.295
14.17	21.8	10.070
10.40	20.0	10.130

From our table 4.1, test our W shape beam

$$\sigma = \frac{P_{tot}}{A} + \frac{M_x(Y_{max})}{I_x} + \frac{M_y(X_{max})}{I_y}$$

$M_x = M_y = (42736.35\ lb)(12\ in) = 512836.2\ lb\cdot in$

- **W16X77** $I_x(in^4) = 1160$, $I_y(in^4) = 138$, $S_y(in^3) = 26.9$, $d = 16.77$, $A = 22.6\ in^2$, $width = 10.295\ in$

$Y_{max} = \frac{d}{2} = \frac{16.77}{2} = 8.385\ in$

$X_{max} = \frac{Width}{2} = \frac{10.295}{2} = 5.147$

$$\sigma = \frac{42735.35\ lb}{22.6\ in^2} + \frac{(512836.2\ lb\cdot in)(8.385\ in)}{1100\ in^4} + \frac{(512836.2)(5.145)}{138}$$

$\sigma = 24{,}920\ \frac{lb}{in^2} < \sigma_{all}\ (25ksi)$ Good

- **W14X74** $I_x(in^4) = 796$, $I_y(in^4) = 134$, $S_y(in^3) = 26.6$, $d = 14.17$, $width = 10.070$

Area $= 21.8\ in^2$

$Y_{max} = 14.17/2 = 7.085$

$X_{max} = 10.070/2 = 5.035$

$$\sigma = \frac{42735.35\ lb}{21.8\ in^2} + \frac{(512836.2\ lb\cdot in)(7.085\ in)}{796\ in^4} + \frac{(512836.2\ lb\cdot in)(5.035\ in)}{134\ in^4}$$

$\sigma = 25{,}794.5\ \frac{lb}{in^2} > 25\ ksi\ (\sigma_{all})$ Failure

- **W10X68** $I_x = 394\ in^4$ $I_y = 134\ in^4$ $S_y = 26.4\ in^3$, $d = 10.40\ in$, $A = 20\ in^2$, $width = 10.130\ in$.

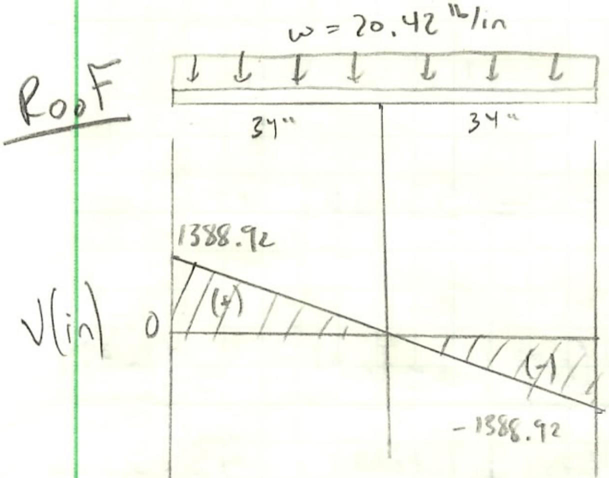
$Y_{max} = 10.40/2 = 5.2\ in$

$X_{max} = 10.130/2 = 5.065\ in$

$$\sigma = \frac{42735.35\ lb}{20\ in^2} + \frac{(512836.2\ lb\cdot in)(5.2\ in)}{394\ in^4} + \frac{(512836.2\ lb\cdot in)(5.065\ in)}{134\ in^4}$$

$\sigma = 28{,}284.6 > \sigma_{all}$ Failure.

Roof

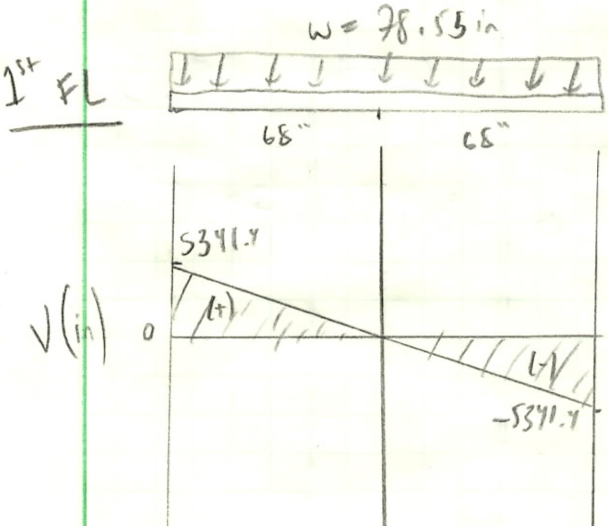


$P_1 = 24021.05$ lb

$P_2 = 2777.8$ lb

$P = P_1 + P_2 = 26799$ lb

1st FL

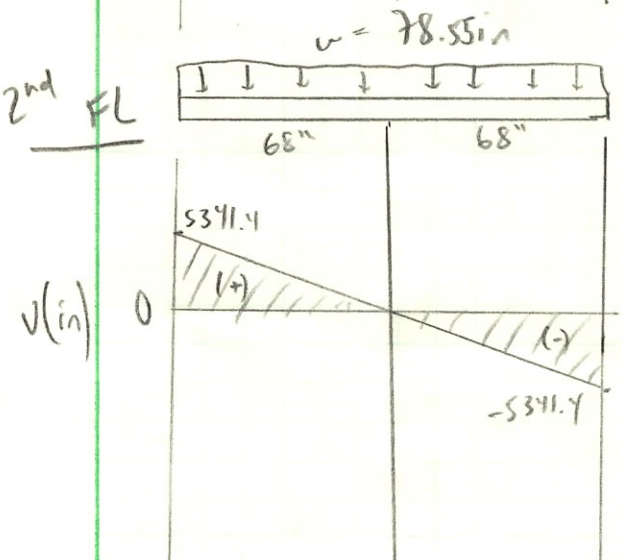


$P_1 = 42735.97$ lb

$P_2 = 10682.8$ lb

$P = P_1 + P_2 = 53418.8$ lb

2nd FL



$P_1 = 21368$ lb

$P_2 = 10682.8$ lb

$P = P_1 + P_2 = 32050.8$ lb

$P_{total} = 112268.5$ lb

$$M_x = M_y = (112268.5)(12) = 1347221.5 \text{ lb-in}$$

$$S_y = \frac{M}{\sigma_{all}} = \frac{1347221.5}{25000} = 53.889 \text{ in}^3$$

Designation	A (in^2)	d (in)	b_f (in)	I_x (in^4)	I_y (in^4)	S_y (in^3)
W27X146	43.1	27.4	14	5660	443	63.5
W14X145	42.7	14.8	15.5	1710	677	87.3
W30X173	51	30.4	15	8230	598	79.8

$$\sigma = \frac{P}{A} + \frac{M_x(Y_{max})}{I_x} + \frac{M_y(X_{max})}{I_y}$$

① W 27x146

$$\sigma = \frac{32050}{(43.1)} + \frac{(1347221.04)(27.4/2)}{(5660)} + \frac{(1347221.04)(14/2)}{443}$$

$$\sigma = 743.62 + 3260.94 + 21287.9 = 25292.48 > \sigma_{all}$$

fail

② W 14x145

$$\sigma = \frac{32050}{42.7} + \frac{(1347221.04)(14.8/2)}{1710} + \frac{(1347221.04)(15.5/2)}{677}$$

$$\sigma = 750.6 + 5830.08 + 15422.4 = 22003 < \sigma_{all}$$

Pass

③ W 30x173

$$\sigma = \frac{32050}{51} + \frac{(1347221.04)(30.4/2)}{8230} + \frac{(1347221.04)(15/2)}{598}$$

$$= 628.43 + 2488.18 + 16896.58 = 20013.2 < \sigma_{all}$$

Pass

Choose $\boxed{\text{W14X145}}$ because it is lightest

(14) 2.5) Determine min size of 18" thick sqr footings. B/4 and D/4 columns.

$$\sigma_{all} = \frac{7000}{2} = 3500 \frac{lb}{ft^2}\left(\frac{1}{144}\right) = 24.31 \text{ psi}$$

$$\sigma_c = (18")(.15 \text{ lb/ft}^3)\left(\frac{1 ft^3}{1728 in^3}\right) = 0.156 \text{ psi}$$

$$P = 42\,736.6 \text{ lb}$$

$$\sigma_{all} = \sigma_c + \frac{P}{A} = 24.31 = 0.156 + \frac{42736.6}{A}$$

B4

$$\frac{42736.6}{A} = 24.15 \Rightarrow A = 1769.36 \text{ in}^2$$

$$S = \sqrt{1769.36} = 42.06 \text{ in} \approx \underline{42 \text{ in}}$$

D4

$$A = \frac{P}{A} = \frac{11268.46}{24.31} = 4618.2 \text{ in}^2$$

$$S = \sqrt{4618.2} = 67.95 \text{ in} \approx \underline{68 \text{ in}}$$

2.6) Investigation of impact | Calculations.

- New Request: 250 psf on roof by adding a new generator

$$\Sigma M_A = \overset{L_{1A}}{(66{,}669\ lb)(96.5\ in)} + \overset{L_{1B}}{(23935\ lb)}(0.75) + R_B\ 0$$

$$R_B\ 24{,}519\ lb.$$

$$\Sigma F_y = R_B + R_A - (66{,}669\ lb) - (23935\ lb) = 0$$

$$R_A = 55.989\ lb$$

W_1 = total weight = 236.096 lb/in

W_2 = 132.7 lb/in

$$S_{min} = \frac{4.254.775\ lb\text{-}in.}{23{,}690\ \frac{lb}{in^2}} = 170.191\ in^3$$

$$\delta_{tot} = \frac{-5 w_2 L^4}{384EI} + \frac{w_1 y}{24EIL}\left(a^2 4 l^2 a^2 - 2ax^2\ 2La + Lx^3\right)$$

$$L = 181\ in$$

when, $\frac{dy}{dx} = 0$, $x = 166.4\ in$

live load definition

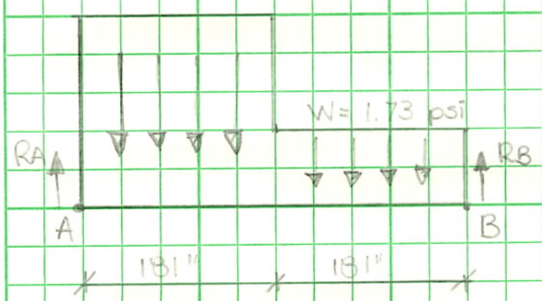
$$total = \delta = \frac{5 w_2 L^4}{384EI} + \frac{w_1 (9.79 \times L'')}{24EIL},$$

$$live = \delta = \frac{5 w_2 L^4}{384EI}$$

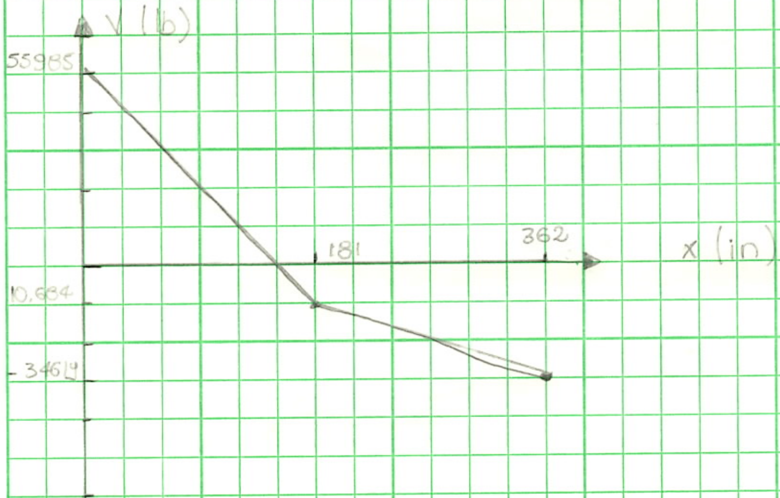
total live load = $274\ \frac{lb}{in}$

- So the lighter beams vertre gring to test are?

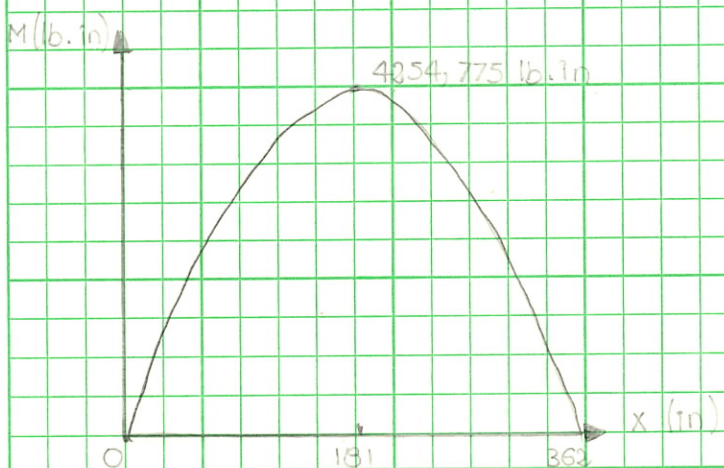
W 18x76
W 24x68
W 27x84



Shear



Moment



2.6) Investigation of Impact Calculations.

Criterio live: $\frac{362 \text{ in}}{360 \text{ in}} = 7{,}005$

total $= \frac{362 \text{ in}}{240 \text{ in}} = 1{,}51$

test.

- W 18 x 75

$I_x = 1330 \text{ in}^4$

$$\text{live} = \frac{(5)(229 \text{ lb/in})(362 \text{ in})^4}{384\,(29 \times 10^6 \text{ lb/in}^2)(1330 \text{ in}^4)} = 1.6 > 1.005 \quad \text{fail}$$

- W 24 x 68

$I_x = 1830 \text{ in}^4$

$$\text{live} = \frac{(5)(274 \text{ lb/in})(363 \text{ in})^4}{384\,(29 \times 10^6 \text{ lb/in}^2)(1830 \text{ in}^4)} = 1.157 > 1.005 \quad \text{fail}$$

- W 27 x 84

$I_x = 2850 \text{ in}^4$

$$\text{total} = \frac{5\,(132.7 \text{ lb/in})(362 \text{ in}^4)}{384\,(29 \times 10^6 \text{ lb/in}^2)(2850 \text{ in}^4)} + \frac{(236 \text{ lb/in})(9.29 \times 10^4)}{24\,(29 \times 10^6 \text{ lb/in})(2850 \text{ in}^4)} = 68$$

$$\text{live} = \frac{5\,(274 \text{ lb/in})(362 \text{ in}^4)}{384\,(29 \times 10^6 \text{ lb/in}^2)(2850 \text{ in}^4)} = 0.74 < 1.005 \quad \text{pass}$$

Depth of W 27 x 84

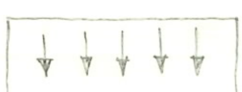
$\frac{362 \text{ in}}{16} \leq \text{depth}$

$26.7 \text{ in} \leq 22.625 \text{ in}$ pass

- W27x84 ✓

- Columns

B4 unaltered by generator

B4 

136 in

total weight $= (.4514 \text{ lb/in}^2)(45.25 \text{ in})$
$+ (190 \text{ lb/in}^2)(1 \text{ lb} / 12 \text{ in})^3 (6 \text{ in})(45.25 \text{ in})$
$+ (1.73 \text{ lb/in}^2)(45.25 \text{ in}) = 122.28 \text{ lb/in}$

$P = (122.28 \text{ lb/in})(136 \text{ in}) = 16{,}630.08 \text{ lb}$

Total depth on all flovel = 158,121 lb

$$M_y = (158{,}121\ lb)(12\ in) = 1{,}897{,}468.4\ lb.in$$

$$S_y = \frac{1{,}897{,}468.4\ lb.in}{23{,}060\ lb/in^2} = 75.9\ in^3$$

Steel shapes to be examind

- W 30 x 173
- W 14 x 145
- W 33 x 201

tely : $\gamma_{all} = 25{,}000\ lb/in^2$

- W30 x 173

$A = 41\ in^2$, $C_y = 15.2\ in$ $C_x = 7.5\ in$, $I_y = 8230\ in^4$ $I_x = 592\ in^4$

$$\gamma = \left(\frac{198121\ lb}{51.0\ in^2}\right) + \left(\frac{1297468.42\ lb.in}{8230\ in^4}\right) + \left(\frac{(1897468.41\ lb.in)(36\ in)}{598\ in^4}\right)$$

$\gamma = 30{,}402{,}5\ psi > 25{,}060\ psi$ foil

- W14 x 143

$A = 42.7\ in^2$, $C_y = 7.4\ in$, $C_x = 7.76\ in$, $I_y = 1710\ in^4$, $I_x = 677\ in^4$

$$\gamma = \left(\frac{158{,}121\ lb}{42.7\ in^2}\right) + \left(\frac{1897468.416\ lb\ (7.9\ in)}{1710\ in^4}\right) + \left(\frac{18974624\ lb.in\ (7.72\ in)}{677\ in^4}\right)$$

$\gamma = 33{,}635{,}7\ psi > 25{,}060\ psi$ foil

- W 33 x 201

$A = 59.2\ in^2$ $C_y = 16.85\ in$, $C_x = 7.25\ in$, $I_y = 7495\ in^4$, $I_x = 1466\ in^4$

$$\gamma = \left(\frac{158{,}12\ lb}{59{,}2\ in^2}\right) + \left(\frac{(187468.4\ lb.in)(16.185\ in)}{744\ in^4}\right) + \left(\frac{(147469.41\ lb.in)(7.85\ in)}{11.800\ in^4}\right)$$

$\gamma = 2015.24\ psi < 25\ 000\ psi$ pass

- W 33 x 201 ✓

footing :

- B/4 is not offerid by generotor
- D/4 foll = 24,31 psi

New area : $A = 7150.75\ in^2$

$$S = \sqrt{7150.75\ in^2} = 84.56\ in \approx 85\ in$$

2.7 – Composite Action

As stated within the instructions of the project, we are to assume all beams are simply supported, steel has elasto-plastic stress-strain characteristics, ignore any lateral load, and ignore any composite action between the concrete slabs and steel beams. However, in a more realistic approach to the design project where the concrete slabs and beams would be connected (by bolts, etc) then you would have to take composite action into consideration.

If the two materials were in composite action, it would result in an overall stronger member because materials that are in composite action are considered as one material. Composite action increases the strength of the beams which increases the yielding stress and allowable stress.

If the member is stronger, it means that it can support a greater allowable load. This greater allowable load would result in a smaller sectional modulus (compared to the one calculated in the report) because allowable stress and section modulus are inversely proportional. As allowable stress increases, section modulus decreases. With a smaller section modulus, we have more choices for beams to choose from for our design. This allows us to choose a beam that is lighter, more economical and supports a greater load.