Lebanese American University
LAU School of Engineering
LAU School of Arts and Sciences
Department of Computer Science & Mathematics

Design of Intermodal Station

Structural Design and Weight Optimization using Genetic Algorithm

Thierry Sayah

ID Number: 201407104

Advisor: Dr. Caesar Abi Shdid & **Co-Advisor**: Dr. Danielle Azar

May 28th

Final Year project

Table of contents

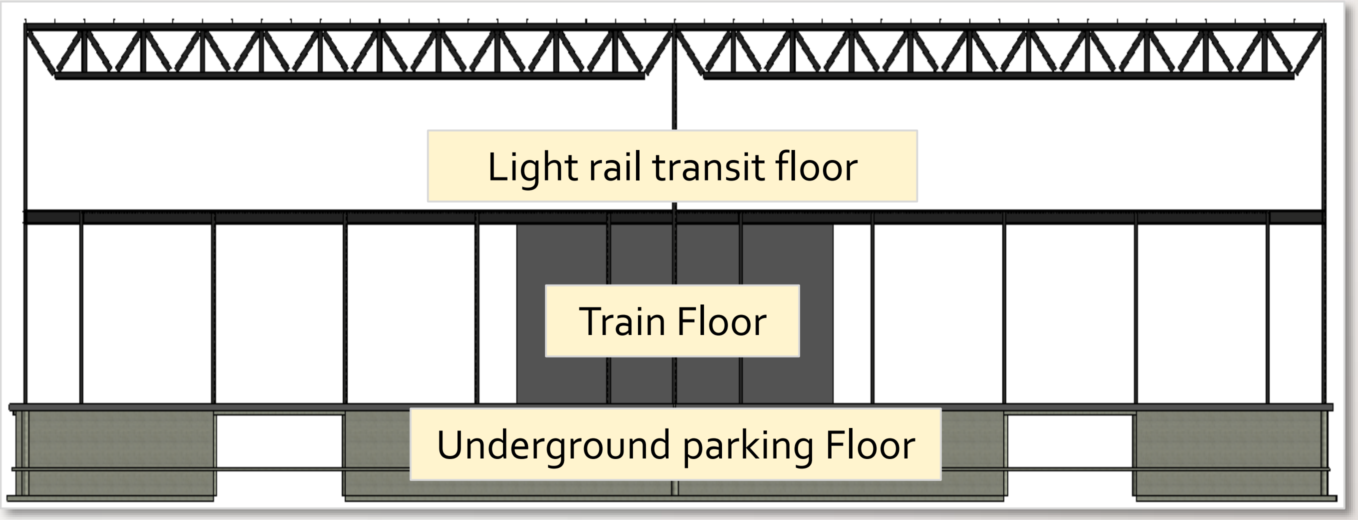
- **Part 1: Structural Design**
  - General overview
  - Software used
  - Materials used
  - Loads used
  - Calculations and software results
- **Part 2: Weight Optimization**
  - Algorithm vs. Genetic Algorithm
  - Genetic Algorithm: How it works ?
  - Terminologies in GA
  - How does it relate to our project ?
  - Running the Program
  - Results
  - Conclusion
- **References**
- **Appendices**

Part 1: Structural Design

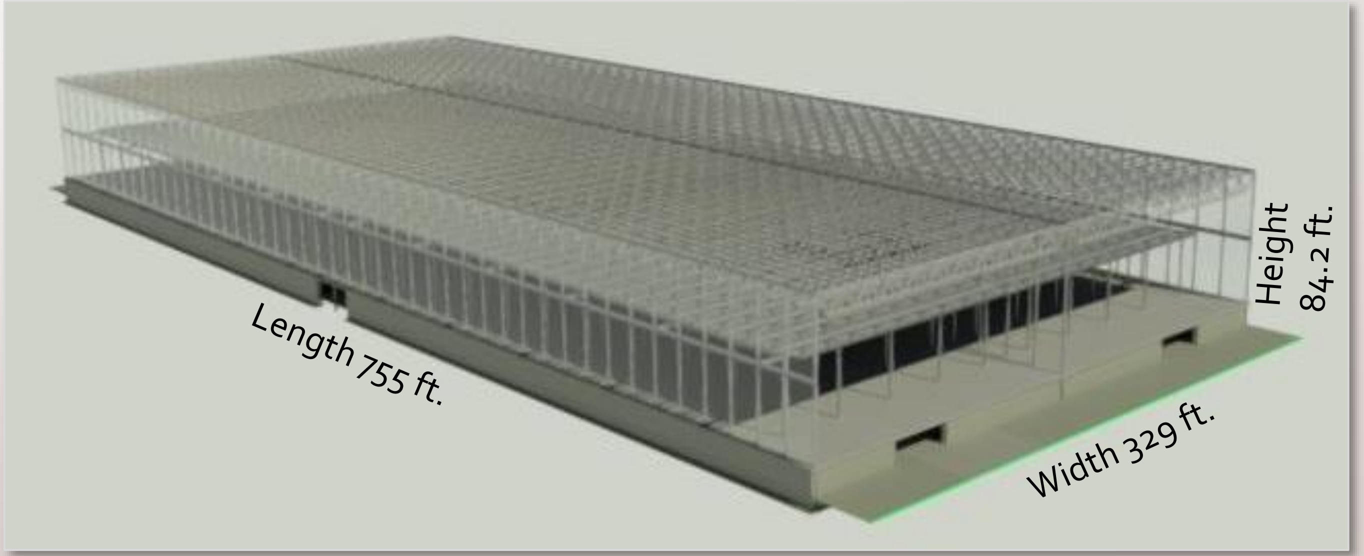
- General overview
- Software used
- Materials used
- Loads used
- Calculations and software results

General Overview (1/3)

- 3 Storey intermodal station, composed of:
  - 1 Underground parking floor
  - 1 Train floor (ground floor)
  - 1 Light rail transit floor (1st floor)

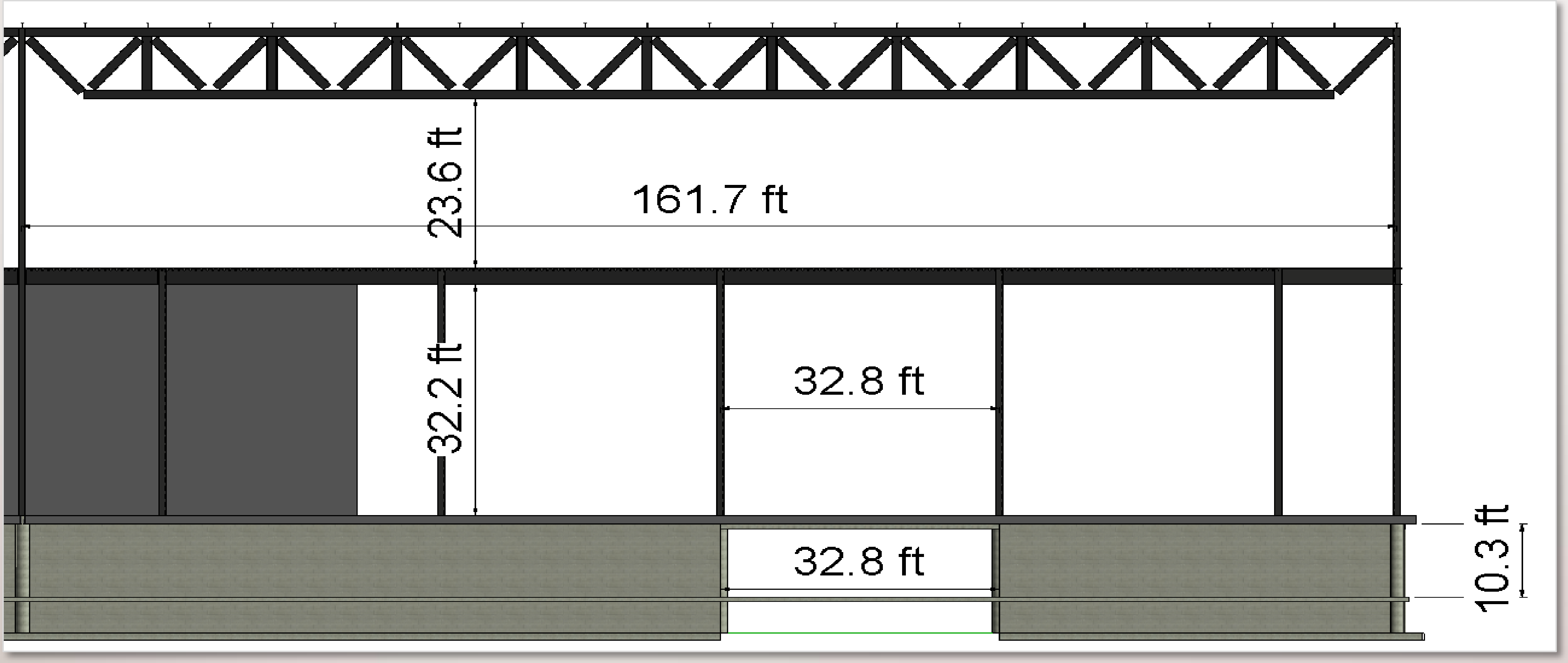


- **Building dimensions:**
  - Length of 755 ft.
  - Width of 329 ft.
  - Height of 84.2 ft.

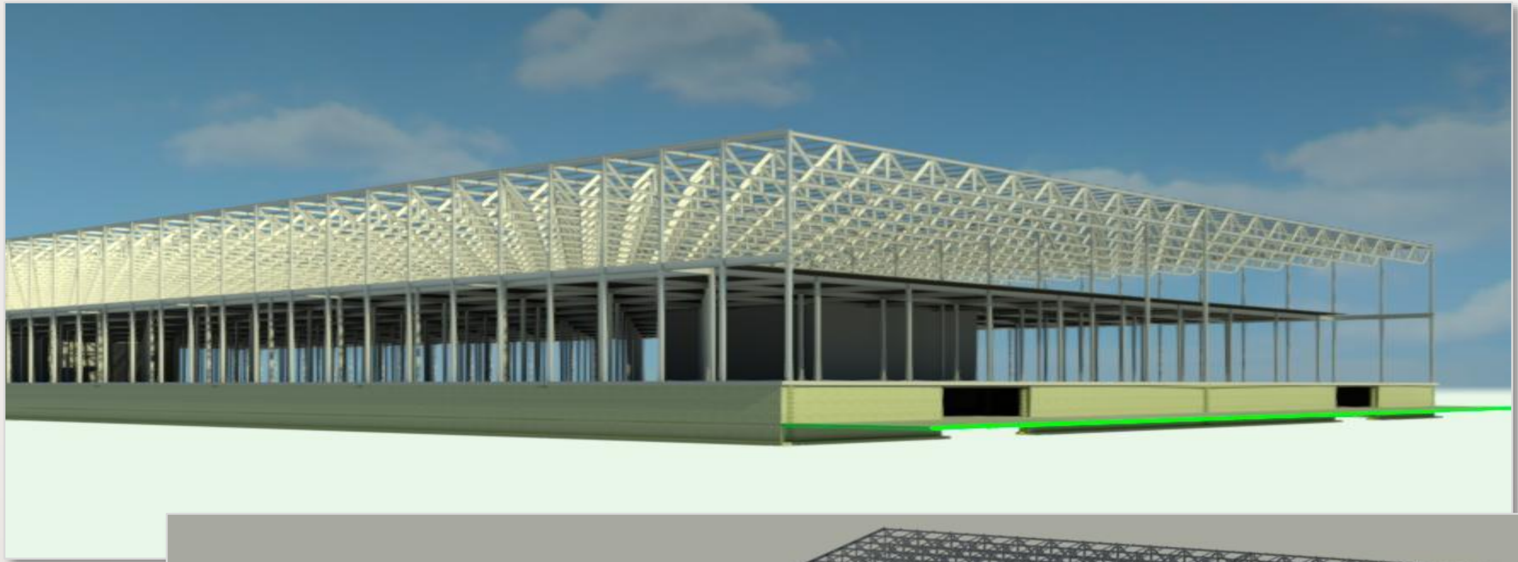


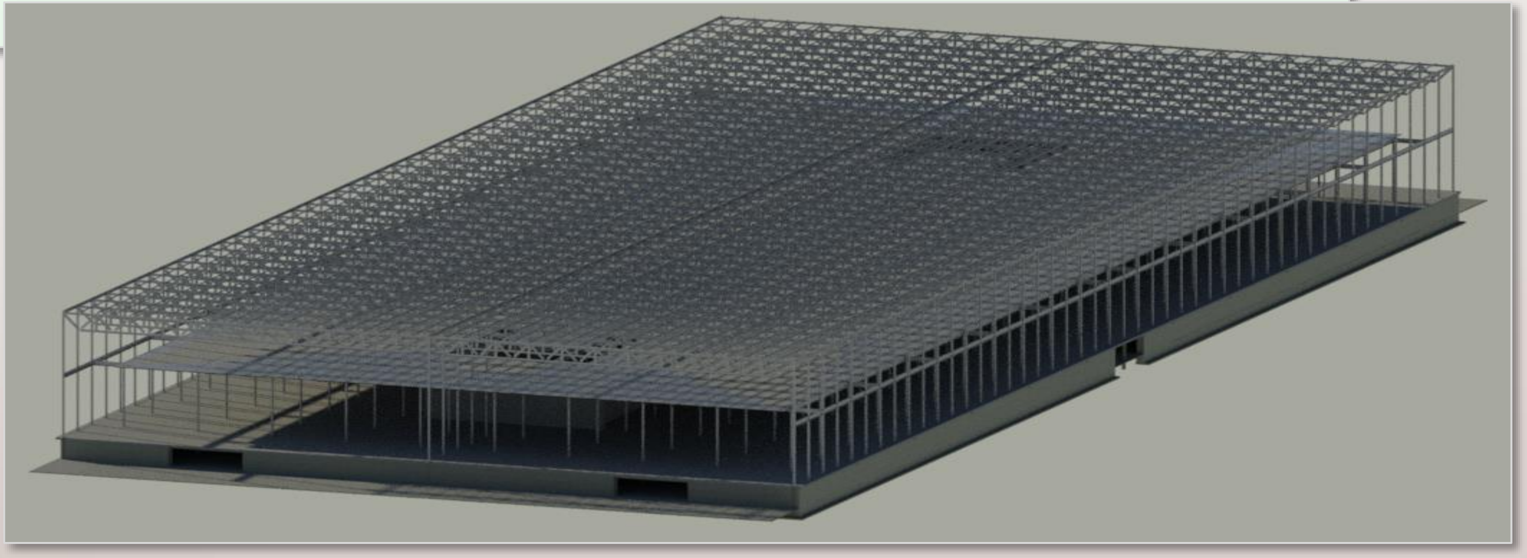
Ft: Feet

General Overview (2/3)



General Overview (3/3)





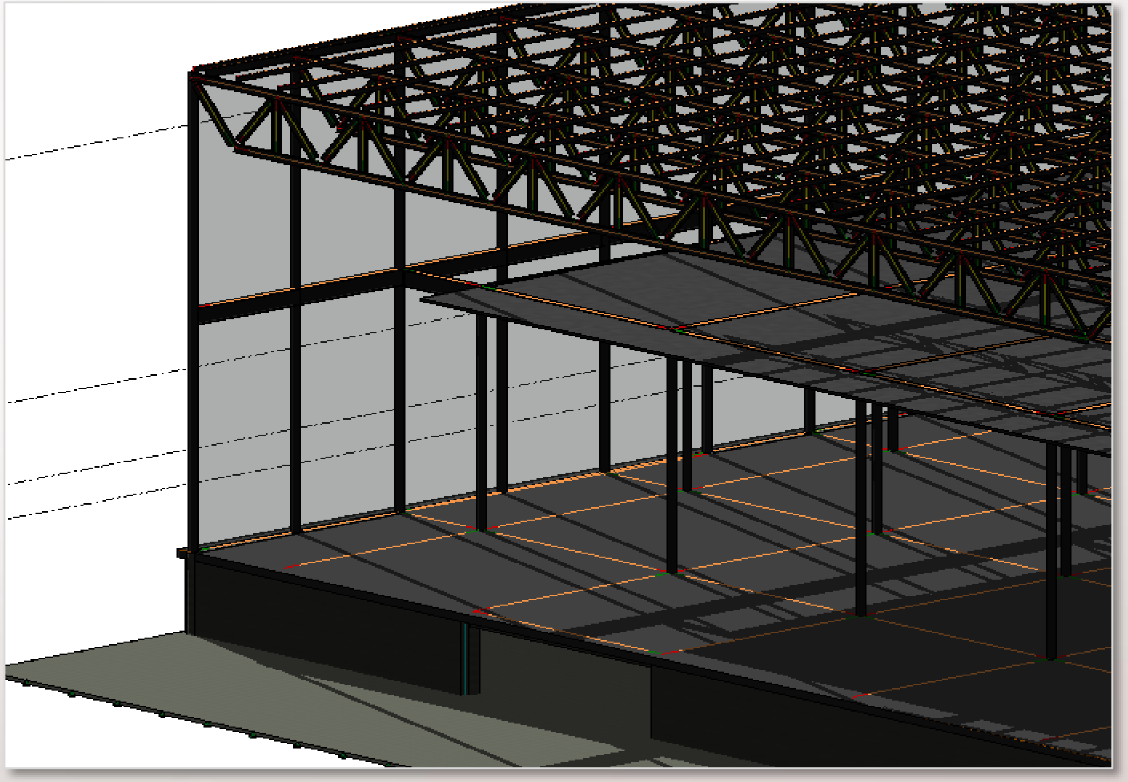
Software Used

- **Autodesk Revit:**
  - Drawing the model
- **Autodesk Robot:**
  - Structural analysis
- **Tekla Tedds:**
  - Element Specific Structural analysis

Materials used

- **Concrete: 5 KSI**
  - Reinforcing bars: Fy = 60 KSI
  - Underground parking & slabs

- **Steel members: Fy = 50 KSI**
  - Columns & beams
    - Ground and 1st floor
  - Square HSS & W shape



Fy: Yield strength
KSI: Kilo pound per square inch

Loads used

- **Load and Resistance Factor Design (LRFD) method**

Solar Panels	Rain load	Roof Live load
13 Lbsf	25 Lbsf	20 Lbsf
		

Lbsf: Pound per square foot

Loads used

- **Train floor and Light rail floors:**
  - Live load = 40 lbsf
    - Table 4.3-1
  - Super imposed dead load=166 lbsf
    - International Research Journal of Engineering and Technology
      - Structural Engineering Aspects of Metro Rail Projects
      - Volume: 06 Issue: 06 | June 2019

2) *Superimposed Dead Load*
As per the design guideline of the metro rail project the superimpose load shall not be taken less than 8.0 Ton/m.

- **Parking floor loads:**
  - Live load = 40 lbsf
    - Table 4.3-1

Garages (See Section 4.10)	
Passenger vehicles only	40 (1.92)

Lbsf: Pound per square foot

Wind Load (1/2)

- **Coastal Structure**
  - Powerful winds
    - Devastating effects on the structure

- **Libnor was a dead end**
  - Catalog for calculation of wind loads on structures
    - No response

LIBNOR
Lebanese Standards Institution
Co-creating Responsibility

Catalogs

Calculation of wind loads on structures
2018

Document Number: NL 137:2018 3rd ed
Sector: Construction Materials and Building
TC: NL TC 3015
ICS: 91.040

Wind Load (2/2)

- **Procedure in table 27.2-1 in ASCE 7**
  - Risk category: Type 2
    - Failure probability = $5*10^{-6}$ / year
  - Basic wind speed = 100 Km/hr

- **Wind load = 11.7 lbsf**

Table 27.2-1 Steps to Determine MWFRS Wind Loads for Enclosed, Partially Enclosed, and Open Buildings of All Heights

Step 1: Determine Risk Category of building; see Table 1.5-1.

Step 2: Determine the basic wind speed, V, for the applicable Risk Category; see Figs. 26.5-1 and 26.5-2.

Step 3: Determine wind load parameters:
- Wind directionality factor, K_d; see Section 26.6 and Table 26.6-1.
- Exposure category; see Section 26.7.
- Topographic factor, K_{zt}; see Section 26.8 and table in Fig. 26.8-1.
- Ground elevation factor, K_e; see Section 26.9
- Gust-effect factor, G or G_f; see Section 26.11.
- Enclosure classification; see Section 26.12.
- Internal pressure coefficient, (GC_{pi}); see Section 26.13 and Table 26.13-1.

Step 4: Determine velocity pressure exposure coefficient, K_z or K_h; see Table 26.10-1.

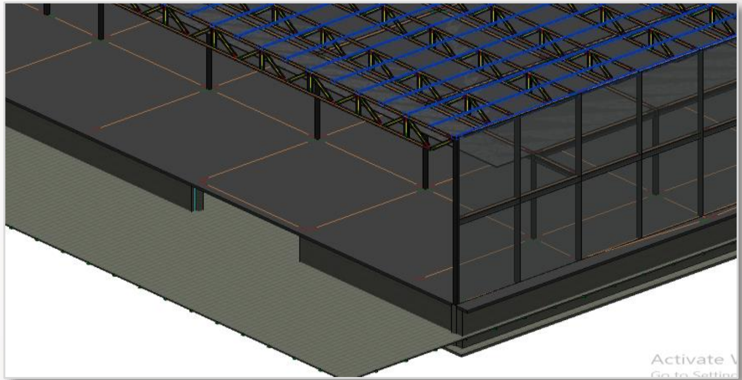
Step 5: Determine velocity pressure q_z or q_h, Eq. (26.10-1).

Step 6: Determine external pressure coefficient, C_p or C_N:
- Fig. 27.3-1 for walls and flat, gable, hip, monoslope, or mansard roofs.
- Fig. 27.3-2 for domed roofs.
- Fig. 27.3-3 for arched roofs.
- Fig. 27.3-4 for monoslope roof, open building.
- Fig. 27.3-5 for pitched roof, open building.
- Fig. 27.3-6 for troughed roof, open building.
- Fig. 27.3-7 for along-ridge/valley wind load case for monoslope, pitched, or troughed roof, open building.

Step 7: Calculate wind pressure, p, on each building surface:
- Eq. (27.3-1) for rigid and flexible buildings.
- Eq. (27.3-2) for open buildings.

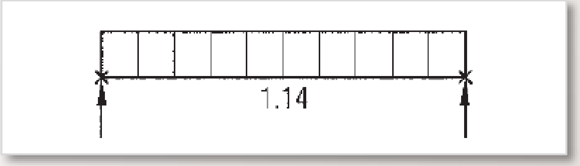
Lbsf: Pound per square foot

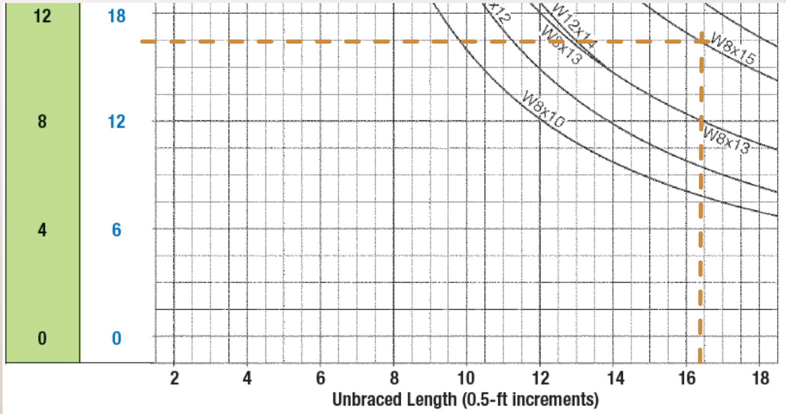
Calculations and software results (1/8)



- **Purlin design**

1. **Load** = 55.6 lbsf
   - $1.2 * D + 1.6(Lr \text{ or } R) + 1 \,(L \text{ or } W)$
2. **Tributary Area** = $16.4 * 7.3 = 120.38\ ft^2$
3. **Wu** = $408.1\ lb/ft$
4. **Mu** = $\frac{0.4081 * 16.4\text{^}2}{8} = 13.72\ k.ft$
5. **Cb** = 1.14 ➔ $\frac{Mu}{Cb} = 12.04\ k.ft$
6. **Section selected:**
   - Try W8*15 and add Self weight : $\phi Mn = 16.42 * 1.14 = 18.42\ k.ft$
     - New Wu = $426.1\ lb/ft$
     - New Mu = $14.32\ K.ft < \phi Mn$
7. **Deflection check:**
   - Allowable: $L/360 = 0.546\ in.$
   - Max deflection =$\frac{W*L\text{^}4}{384*EI} = 0.057\ in.$
8. **Shear check:**
   - φVn = $59.6\ kips$
   - Vu = $3.5\ kips < \phi Vn$





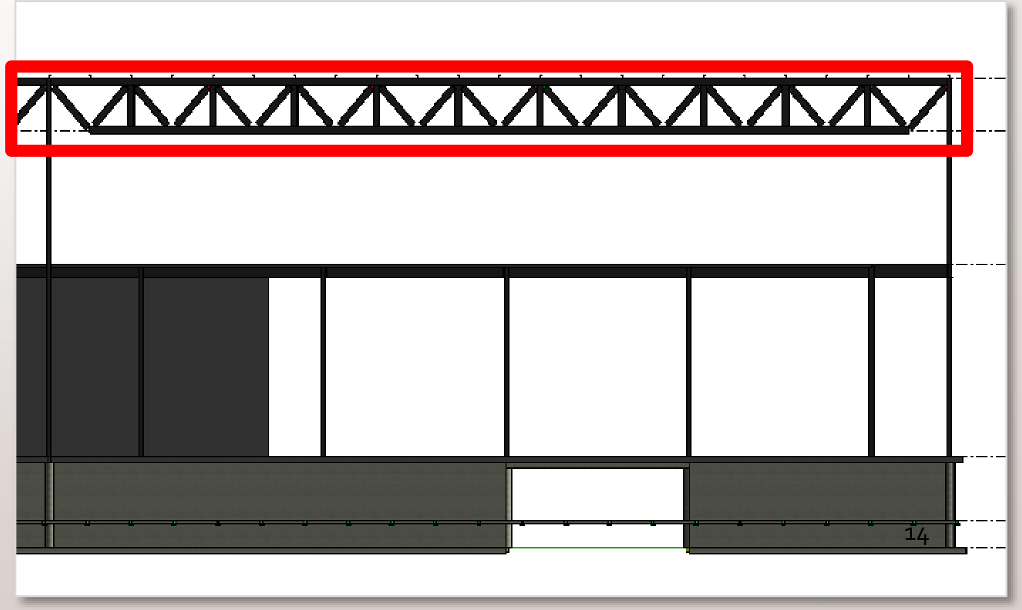
$$\frac{fiMn}{Mu} \geq 1$$

D: Dead load
Lbsf: Pound per square foot
Lr: Roof live load
R: Rain load
L: Live load
W: Wind Load

Calculations and software results (2/8)

- **Steel Joist selection**
  - Load of 1227 lb/ft (including purlin weight)
  - Span of 161.7 ft
  - Using SJI tables
    - Select 104 DLH 23
      - Capacity = 1332 lb/ft
      - Depth of Joist = 104 in.

			< 105	105-138	139	142	145	148	151	155	160	165
104DLH18	59	104	1100	115470	831 426	798 400	768 375	734 353	708 332	674 307	635 279	601 255
104DLH19	67	104	1337	140430	1011 484	971 453	933 426	897 401	861 377	819 349	770 317	727 289
104DLH20	75	104	1504	157890	1146 548	1107 513	1071 483	1032 453	992 427	944 395	886 359	833 327
104DLH21	90	104	1890	198480	1434 673	1376 632	1322 593	1271 558	1220 525	1160 486	1091 442	1028 403
104DLH22	104	104	2119	222540	1607 783	1551 734	1499 689	1449 648	1401 610	1340 564	1261 513	1189 468
104DLH23	109	104	2334	245100	1772 819	1712 768	1644 721	1578 678	1514 638	1437 590	1348 536	1267 489

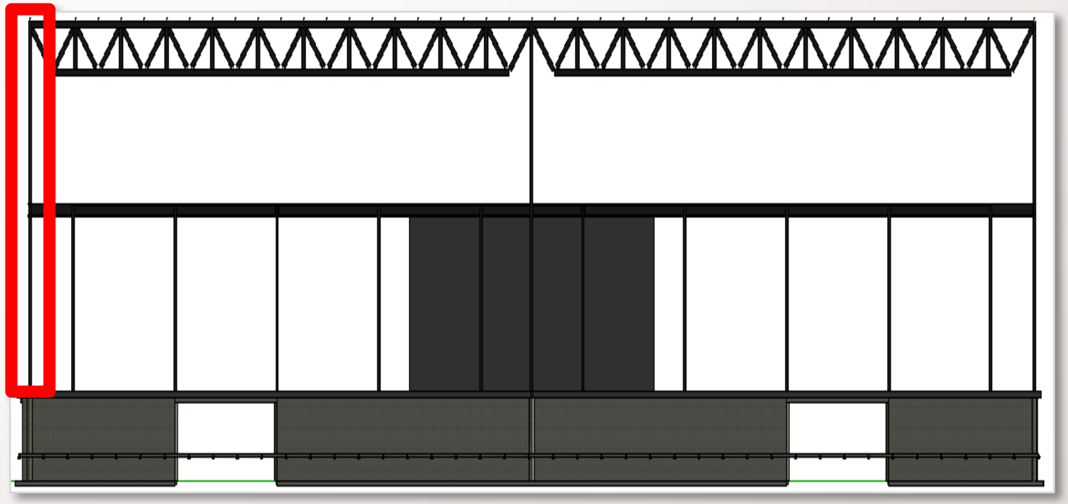


DLH: Type of Steel Joist
In: Inch
Lb/Ft: Pound per linear Foot

Calculations and software results (3/8)

- **Beam-Column design**
  - Length = 33.28 ft
  - Wind load = 192 lb/ft
  - Pu = 107.4 kips

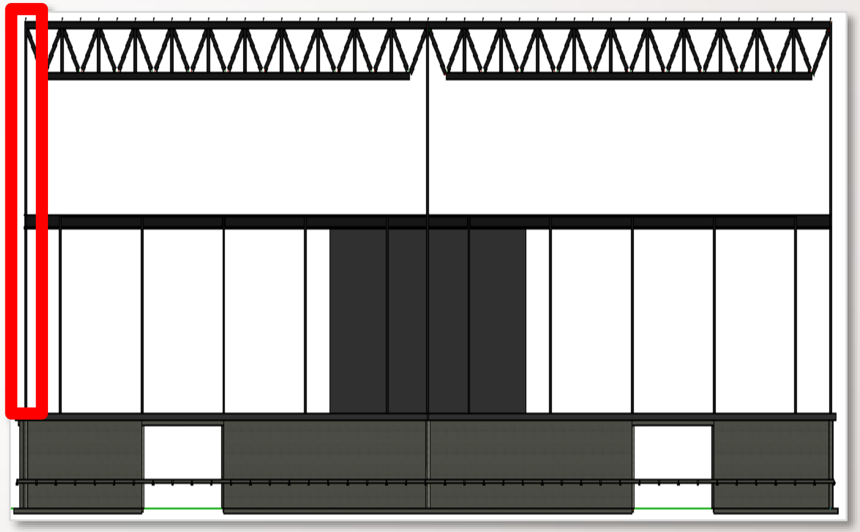
    Includes: weight of steel joist and purlin
  - Moment due wind load = 27.12 K.ft
  - $B_1 = \frac{1}{1-\frac{Pu}{Pex}}$; $Pex = \frac{\pi^2 * E * Ix}{(0.65*33.28*12)^2} = 4.24Ix$; $\boldsymbol{Try\ HSS\ 10 * 10 * \frac{5}{8}}$; $Ix = 304\ in^4$
  - $B_{1\,=\,1.09\,>\,1}$ ⇨ Mu = 1.09*27.12 = 29.58 K.ft & Pu = 107.4 kips



B_1: Moment amplification factor
Ft: Feet
Kips: Thousand pound

K.ft: Thousand pound per foot
Lb/Ft: Pound per linear Foot
Pex: Elastic Euler buckling load about the *x*-axis

Calculations and software results (4/8)



- **Beam-Column design**
  - φMn calculation:
    1. **Yielding**
       - Mp = Mn = Fy∗Z = 50 ∗ 73.2 = 3660 K.in = 305 k.ft
    2. **Local Buckling**
       - Ht = Bt = 14.2
       - $\lambda = 2.42 * \sqrt{\frac{E}{Fy}} = 58.28 > 14.2 \Rightarrow No\ Local\ Buckling$
    3. **Lateral Torsional Buckling:** Lb = 33.28
       - $Lp = 0.13 * E * ry \frac{\sqrt{J*Ag}}{Mp} = 33.35 > Lb \Rightarrow No\ LTB$
         - Mn = 347 > Mp; Use Mp
    4. **Result:** φMn = 274.5 > 29.58 ; $\frac{Pu}{\varphi Pn} = 0.258 > 0.2$
       - $\frac{Pu}{\varphi Pn} + \frac{8*Mu}{9*\varphi Mn} = 0.258 + \frac{8*29.58}{9*274.5} = 0.36 < 1$; not very efficient

- **Select HSS 6*6*5/8, $\frac{Pu}{\varphi Pn} + \frac{8*Mu}{9*\varphi Mn} = 1.83 > 1$!!**

$$\frac{Pu}{\varphi Pn} + \frac{8 * Mu}{9 * \varphi Mn} \leq 1$$

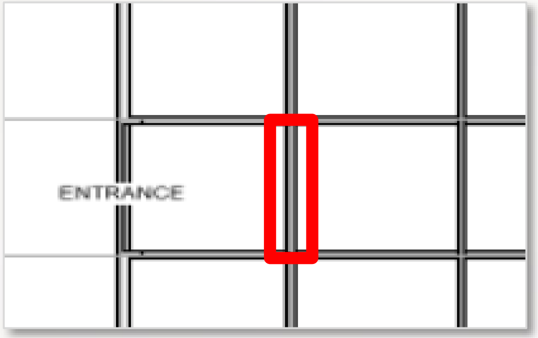
or

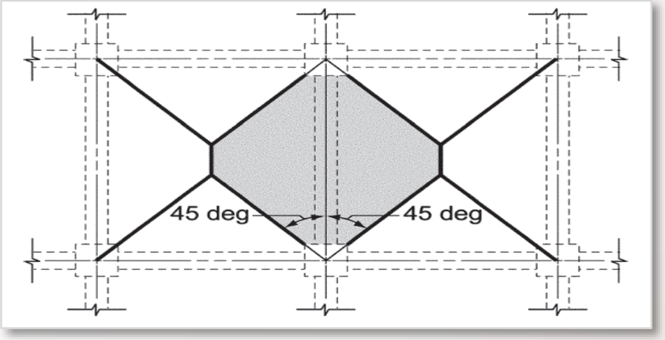
$$\frac{Pu}{2\varphi Pn} + \frac{Mu}{\varphi Mn} \leq 1$$

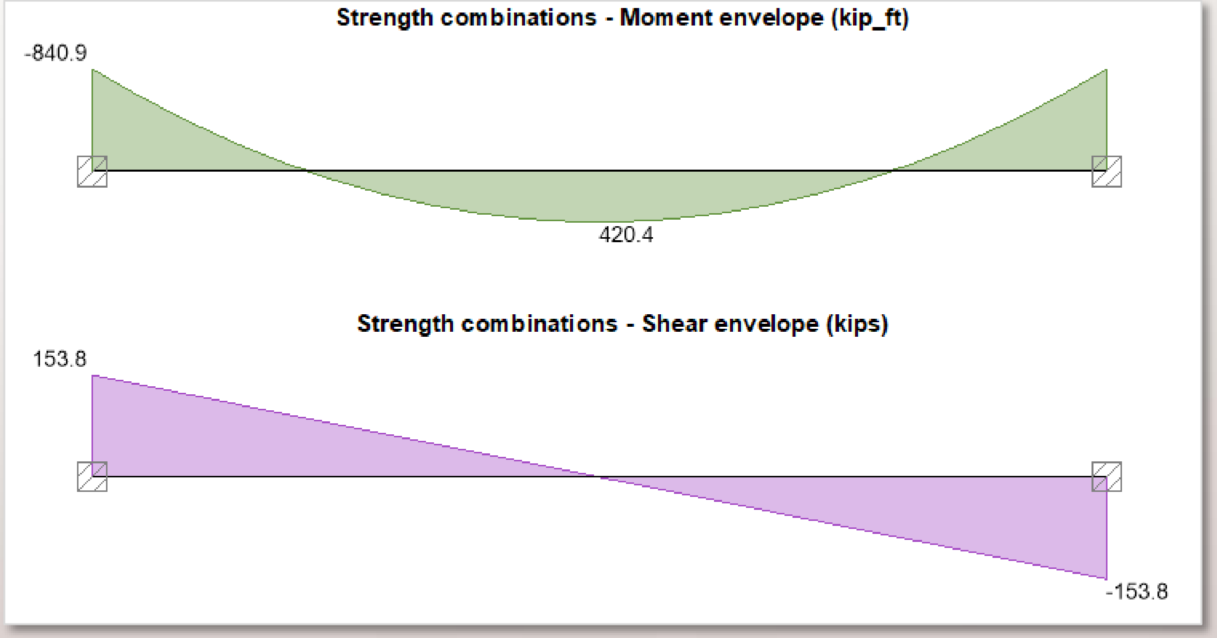
Fy: Yield strength
Z: Gross plastic section modulus

Calculations and software results (5/8)

- **Concrete Beam design**
  - **Interior Beams**
    - Tributary area = 760 ft²
    - Length = 32.8 ft
    - Dimensions = 20 * 35 in.
    - Loads:
      - Dead = 5.34 K/ft
      - Live = 1.31 K/ft







Load combination: 1.2D + 1.6L (Strength)

Node	Force Fx (kips)	Force Fz (kips)	Moment My (kip_ft)
1	0	153.816	-840.859
2	0	153.816	840.859

Ft: Feet
Ft²: Foot square

Calculations and software results (6/8)

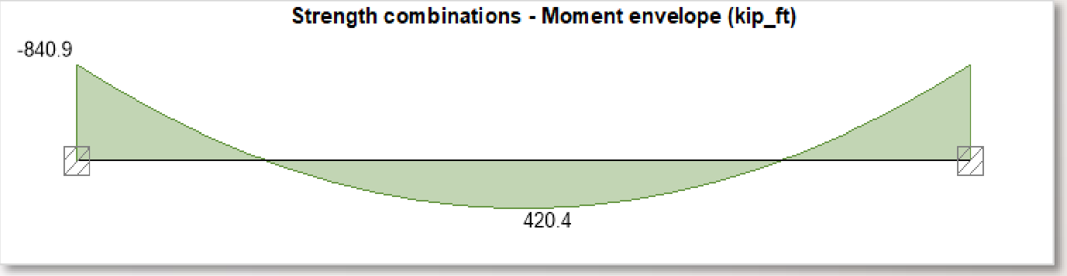
Procedure

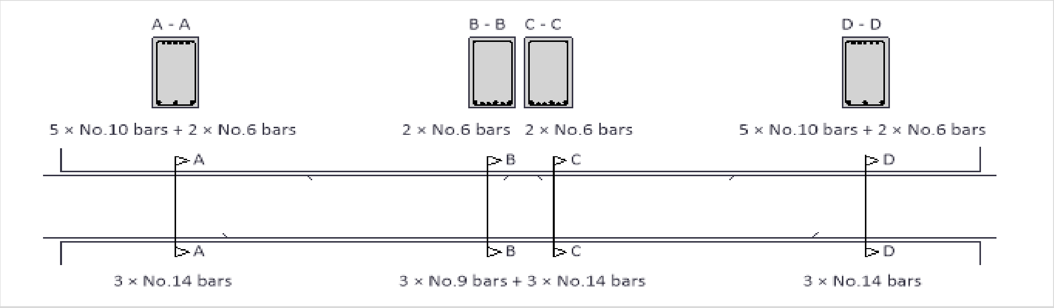
1. Calculate Mu
2. ρ required : $\frac{Mu}{\varphi}$ = ρ req∗ b ∗ d² ∗ Fy ∗ (1 – 0.59∗ρ req ∗ $\frac{Fy}{F'c}$)
3. Area of steel required > As min
   - As required = 6.124 in²
   - As prov = 7.217 in²
   - As min = 2.3 in²
4. Depth of equivalent rectangular stress block: $a = \frac{As\ prov * Fy}{0.85 * F'c * b}$ = 5.095 in.
5. Depth to neutral axis: $c = \frac{a}{\beta 1}$ = 6.3 in.
6. ϕMn = $A_{s,prov}$ ∗ fy ∗ (d – a / 2) = 973.5 K.ft > Mu = 840 K.ft
7. Spacing check: Sprov = 2.67 in (c/c) ; Smax = 10.31 in.

Table 24.3.2—Maximum spacing of bonded reinforcement in nonprestressed and Class C prestressed one-way slabs and beams

Reinforcement type	Maximum spacing s	
Deformed bars or wires	Lesser of:	$15\left(\frac{40,000}{f_s}\right)-2.5c_c$
		$12\left(\frac{40,000}{f_s}\right)$

Results

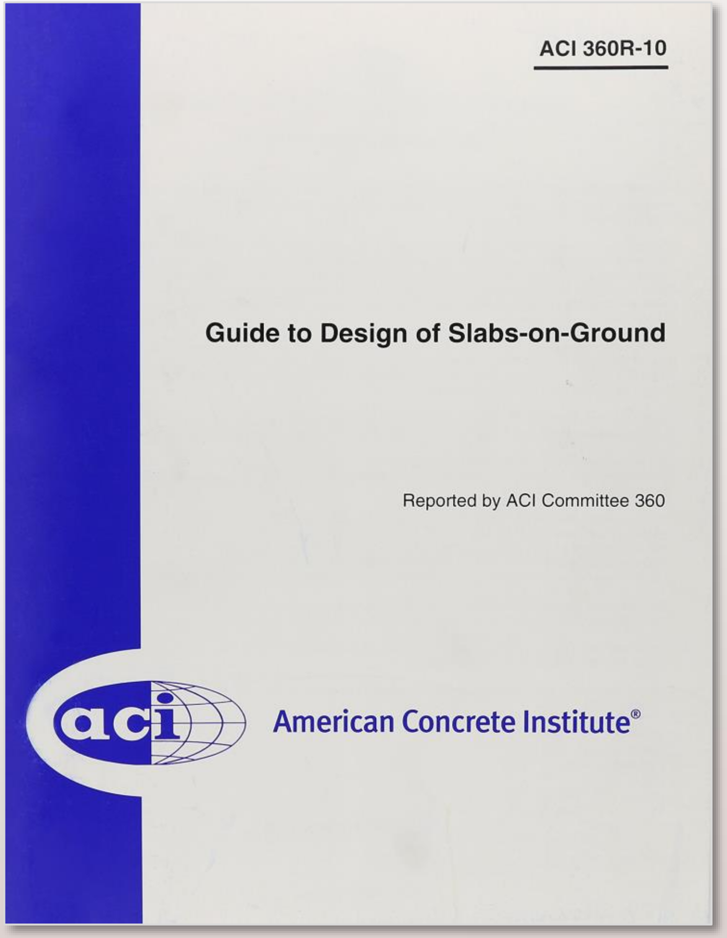




In²:Inch square

Calculations and software results (7/8)

- **Slab on ground design (ACI 360R-06)**
  - Parking Floor
  - Portland Cement Association (PCA) design method
  - Wheel specifications
    - Single axle
    - Single wheel
    - Wheel spacing = 40 in.
  - Loading detail
    - Wheel contact area = 25 in^2
    - Axle load = 25 kips
    - Factor of safety = 2
  - Material properties
    - Concrete F'c = 5 KSI
    - MOR = $9 * \sqrt{F'c} = 636.4\ psi$
  - Allowable stress in concrete = $\frac{636.1}{2} = 318.2\ psi$



In²: Inch square
KSI: 1000 psi
Psi: Pound per square inch
MOR: Modulus of Rupture

Calculations and software results (8/8)

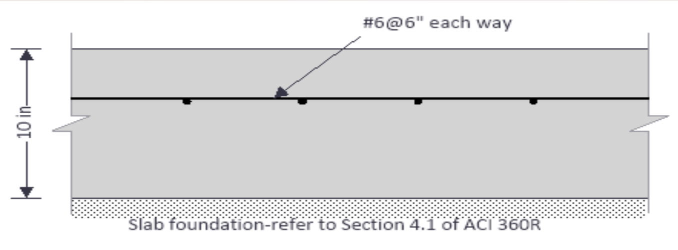
1 .**Stress/1000 lb of axle load** = $\frac{Allowable\ stress}{contact\ area} = \frac{318.2}{25} = 12.7\ psi$

2. **Wheel contact area** = 25 in^2

3. **Wheel spacing** = 40 in.

4. **Result**:
   - **H min = 8.75 in.**
   - **H actual = 10 in.**

5. **Reinforcement:**
   - **Section 2.2.2** *Slabs reinforced for crack width control*
     - Reinforcement in top third of the slab to control shrinkage cracks
   - **Minimum steel**
     - PCA requires 0.5% of slab cross-section for crack width control
     - Area of steel min = 0.005*h = 0.525 in^2
     - Area of steel provided: #6 at 6 inch each way. Aprov = 0.88 in^2



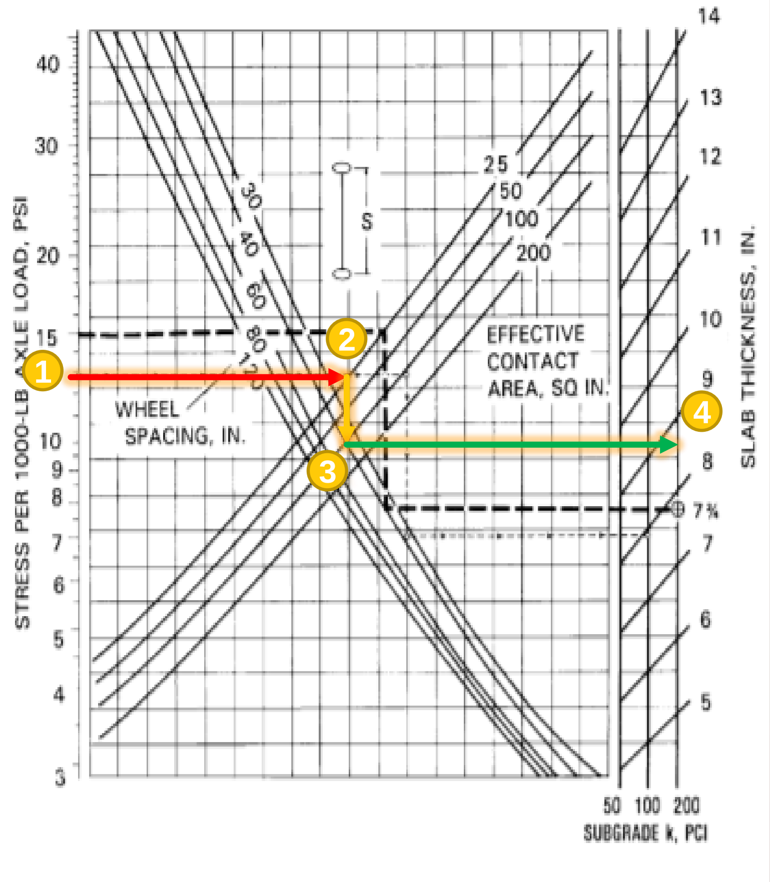


Fig. A1.1—PCA design chart for axles with single wheels.

In: Inch
In²: Inch square
Psi: Pound per square inch

Part 2: Weight Optimization using Genetic Algorithm

- Algorithm vs. Genetic Algorithm
- Genetic Algorithm: How it works ?
- Terminologies in GA
- How does it relate to our project ?
- Running the Program
- Results
- Conclusion

Algorithm versus Genetic Algorithm

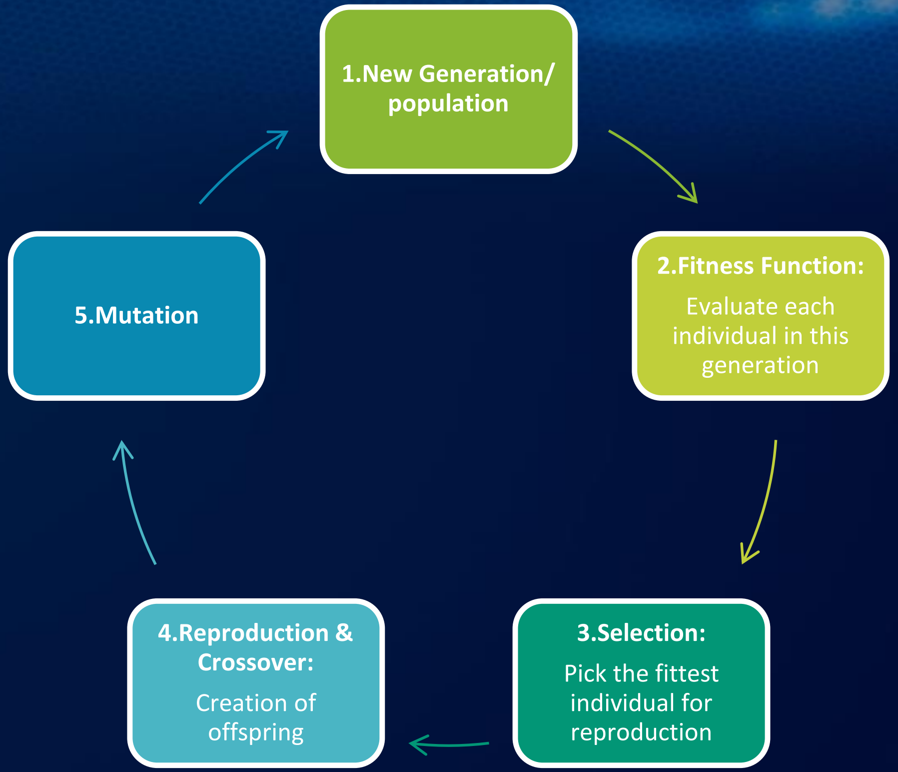
- **Algorithm**
  - List of step by step instructions in order to solve a problem.

- **Genetic Algorithms**
  - a search technique
  - inspired by the theory of natural evolution.
  - natural selection → only the fittest individuals are selected for reproduction.

“If a population wants to thrive, it must improve by itself constantly, it’s the survival of the fittest”

Charles Darwin

Genetic Algorithms: How it works ?



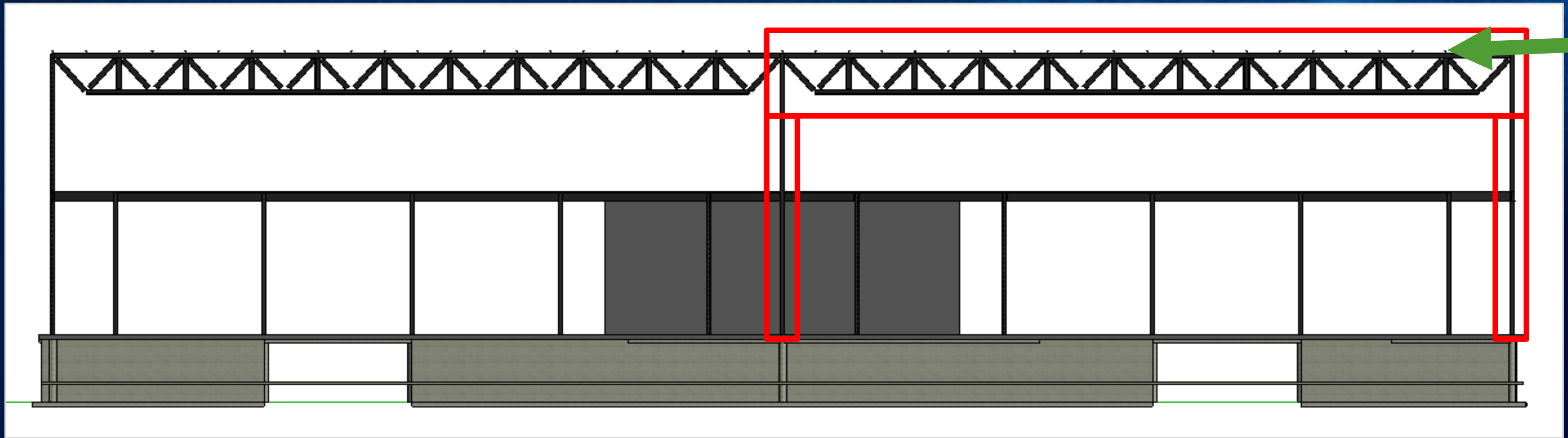
Terminologies in GA

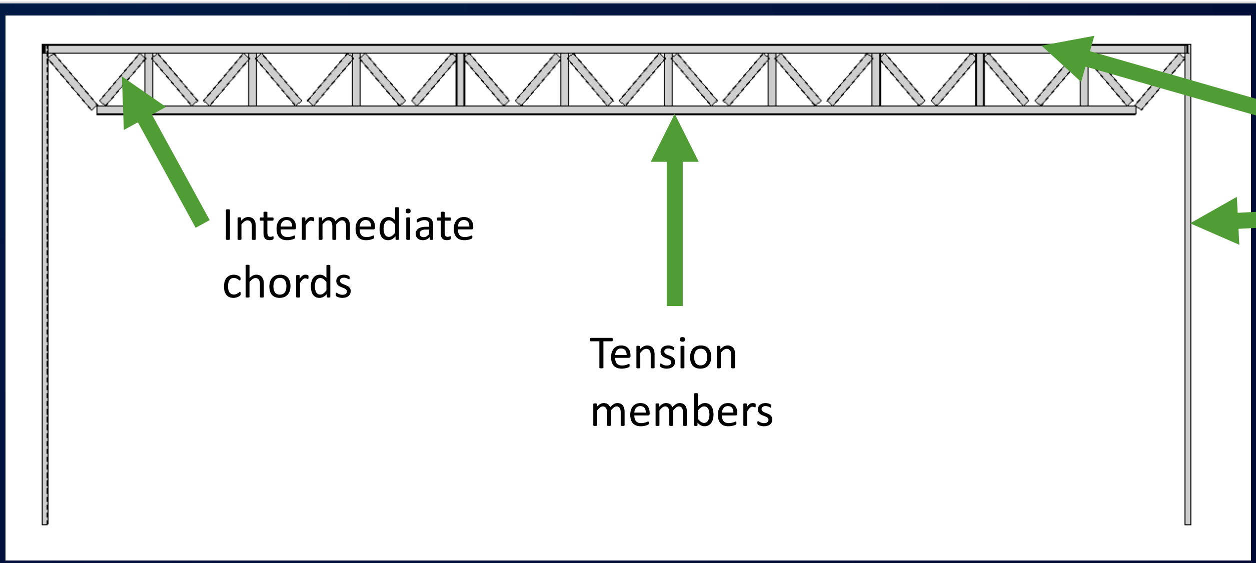
Objects			Function	Operations		
Generation	**Initial population**	**Individual (Chromosome)**	**Fitness function**	**Selection**	**Crossover**	**Mutation**
Contains 2 populations	Set of individuals	Set of variables, known as genes	Gives a fitness score to each individual	Try to select the fittest individuals and let them pass their genes to the next generation.	Exchanging the genes of parents among themselves	Some of the genes can be subjected to a mutation: They can be flipped
Each population is different	Each being a solution to the problem	Ind1 (G1,G2,G3,G4…)	An individual will be selected for reproduction is based on its fitness score.	Individuals with high fitness have more chance to be selected for reproduction.	Ind1 (G1,G2,G3) Ind2 (G4,G5,G6) ↓ Ind1 (G4,G5,G3)	Before Mutation: Ind1 (G1,G2,G3) After Mutation: Ind1 (G1,G4,G5)

G: Gene

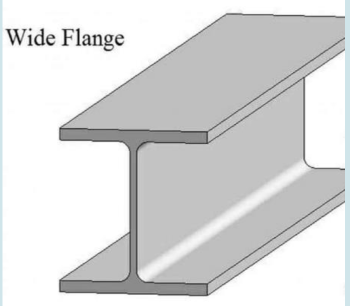
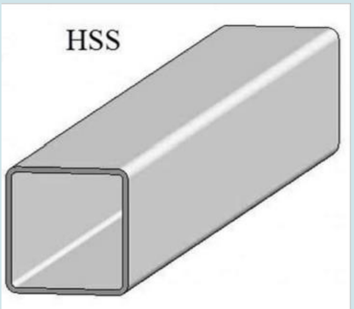
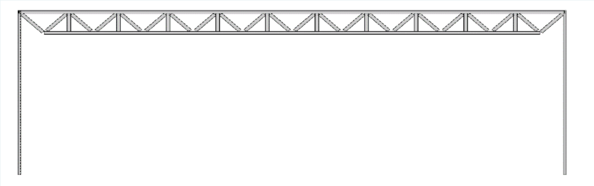
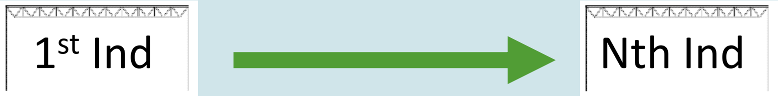
How does it relate to the project? (1/2)

What are we optimizing ?





How does it relate to the project? (2/2)

Genetic Algorithm terminology	Engineering terminology	Illustration	Description
Gene	1 Steel Section (W-shape OR square HSS)	 or 	5 types of genes: 1. Purlin 2. Tension member 3. Compression member 4. Chord 5. Column (HSS)
Individual	Steel Structure (Group of W-Shape and HSS)		Each individual is composed of 5 genes which are a combination of the above genes
Population	A list of different individuals		Each individual with its own Fitness Value
Fitness function	Total weight of the Structure (of 1 individual)	$\sum Length * Weight\ of\ each\ member$	Minimize the weight

Running the Program (1/4)

1. Creating the initial Generation
2. Assigning its Fitness Value

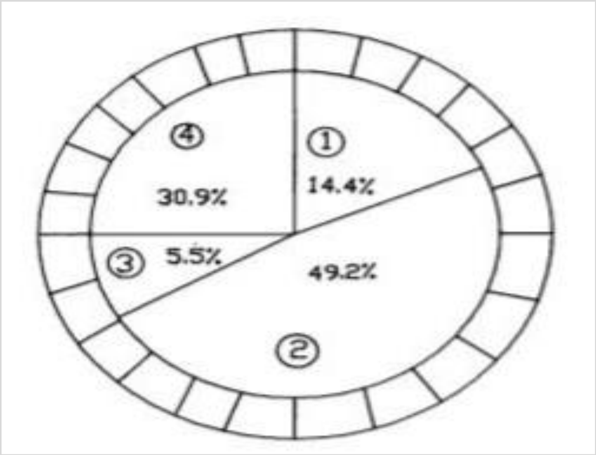
			Genes / Steel section					
			Gene: Purlin	Gene: Tension member	Gene: Compression member	Gene: Chord	Gene: HSS Column	Fitness (weight, lbs)
Generation	Population 1	Structure /Individual 1	W4*13	W8*10	W16*57	W12*16	HSS7*7*1/2	23,979.742
		Structure /Individual 2	W10*19	W4*13	W6*12	W6*12	HSS9*9*5/16	18,090.404
		Structure /Individual 3	W36*330	W8*15	W6*12	W16*57	HSS9*9*5/16	141,128.17
	Population 2	Structure /Individual 1	W8*31	W8*15	W6*12	W4*13	HSS8*8*1/8	20,741.604
		Structure /Individual 2	W24*84	W12*16	W8*15	W18*65	HSS8*8*1/8	58,410.582
		Structure /Individual 3	W24*84	W10*19	W16*45	W12*16	HSS12*12*3/8	49,100.465

Color coding Key: Structure / Individual pop 1 | Structure / Individual pop 2 | Gene/Steel section | Gene/Steel section example | Fitness: Weight of individual

Running the Program (2/4)

3. Selecting the fittest individuals for reproduction

– Roulette Wheel Selection



Individual 1: W4*13, W8*10, W16*57, W12*16, HSS7*7*1/2

Weight: 23,979.742 lbs.

Individual 2: W24*84, W12*16, W8*15, W18*65, HSS8*8*1/8

Weight: 58,410.582 lbs.

Selection Probability of individual 1 > Selection Probability of individual 2

30 % > 20%

Running the Program (3/4)

4. Reproduction

	Gene	Purlin	Tension Member	Compression Member	Chord	HSS Column	Fitness (weight, lbs)
	Factor	$\frac{fiMn}{Mu} \geq 1$	$Ag + Ry \geq 3.38$	$\frac{fiPn}{Pu} \geq 1$	$Ag \geq 0.34$	$\frac{Pu}{\varphi Pn} + \frac{8 * Mu}{9 * \varphi Mn} \leq 1$ or $\frac{Pu}{2\varphi Pn} + \frac{Mu}{\varphi Mn} \leq 1$	$\sum Length * Weight\ of\ each\ member$
Factors	Individual pop 1	1.163	3.801	2.95	4.71	1.263	23,979.742
	Individual pop 2	43.36	5.483	1.21	19.1	2.65	58,410.582
	Resulting Child Individual pop 1/2	1.163	3.801	1.21	4.71	1.263	17,196.742
Sections	Individual pop 1	W4*13	W8*10	W16*57	W12*16	HSS7*7*1/2	23,979.742
	Individual pop 2	W24*84	W12*16	W8*15	W18*65	HSS8*8*1/8	58,410.582
	Resulting Child Individual pop 1/2	W4*13	W8*10	W8*15	W12*16	HSS7*7*1/2	17,196.742

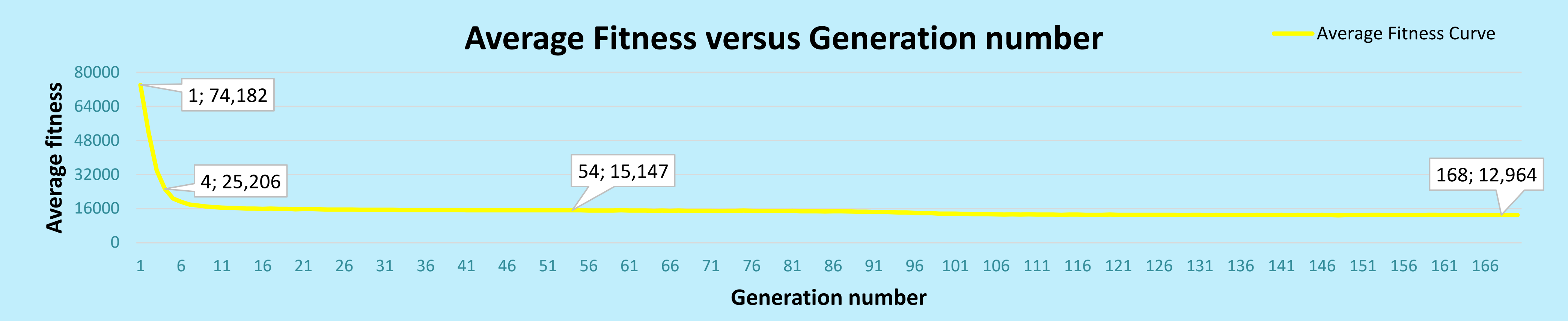
Color coding Key: Structure / Individual pop 1 | Structure / Individual pop 2 | Gene/Steel section | Gene/Steel section example | Fitness: Weight of individual

Running the Program (4/4)

5. Mutation

- **Mutation probability** = 0.1 %
- **Before mutation:**
  - W4*13, W8*10, W8*15, W12*16, HSS7*7*1/2
  - Fitness = 17196.742 lbs
- **After mutation** (if it does occur):
  - W4*13, W8*10, W8*15, W12*16, HSS10*10*5/8
  - Fitness = 19501.73 lbs

Study Results



	Purlin	Tension member	Compression member	Chords	HSS Column	Fitness (weight, lbs)
Hand Calculation	W8*15	W4*13	W8*31	W4*13	HSS 6*6*5/8	22,757.25
Checks Value	1.196	4.83	4.53	3.83	1.83 (>1 !!)	
Genetic Algorithm	W4*13	W8*10	W6*12	W8*10	HSS9*9*5/16	17,557.729
Checks Value	1.16	3.80	1.035	2.96	0.85 (<1)	
					Improvement	**22.8%**

Color coding Key: Gene/Steel section | Gene/Steel section example | Fitness: Weight of individual

Conclusion

1. *GA creates pseudo-random combinations*

   → Calculates their *fitness*

   → Checks *structural integrity*

   → Returns *best individual* it has encountered

2. *Optimization of section efficiency* led to *weight optimization*

3. *Final structure has sufficient strength* with *minimum weight*

4. *The Weight of the structure is lower* due to the *pseudo-random assignments*

Thank you

References

- ASCE-7 minimum design loads and associated criteria for buildings and other structures
- AISC steel manual 15th edition
- Genetic Algorithms in search optimization and machine learning, David E. Goldberg
- International Research Journal of Engineering and Technology
- https://www.metalroofnet.com/metal-roofing-blog/bid/97731/The-Most-Common-Metal-Roof-Materials
- https://www.englertinc.com/blog/weighing-in-metal-roofs-versus-other-roof-choices/
- https://sunmetrix.com/is-my-roof-suitable-for-solar-panels-and-what-is-the-weight-of-a-solar-panel/
- https://up.codes/s/truck-and-bus-garages
- https://pavementinteractive.org/reference-desk/design/design-parameters/equivalent-single-axle-load/
- https://www.tesla.com/en_JO/blog/solar-roof?redirect=no

Appendices Part 1

Calculations and software results

- **Tension member design**

1. Check for yielding

   - $\varphi Pn = \varphi * Fy * Ag \Rightarrow Ag \geq \frac{98.7}{0.9 * 50} = 2.19\ in^2$

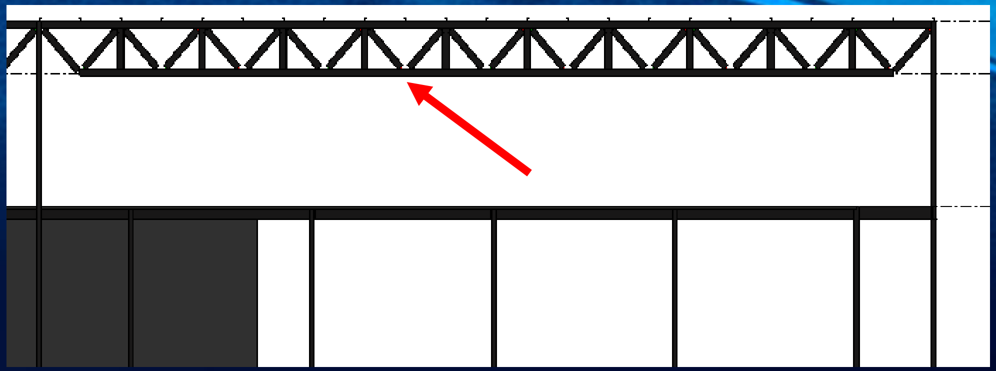
2. **Check for fracture**

   - $\varphi Pn = \varphi * Fu * U * Ag \Rightarrow Ag \geq \frac{98.7}{0.75*65*0.85} = 2.38\ in^2$

3. **Slenderness check**

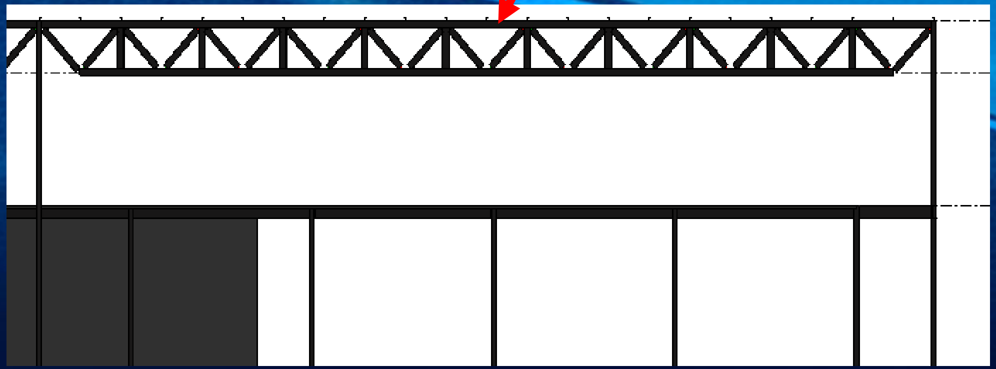
   - $\frac{L}{ry} \leq 300 \Rightarrow ry \geq \frac{11.45 * 12}{300} = 0.458$

– Choose W4*13 :
  - Ag = 3.83 in^2
  - Ry = 1 in.



$Ag \geq 2.38\ \&\ \text{ry} \geq 0.458$

Calculations and software results



- Compression member design
  - Distance between Purlins = 7.3 ft
  - Pu = 78.8 kips
  - Using Tables 4-1 in AISC steel construction manual
    - Select W8*31
      - φPn = 357 kips

F_y = 50 ksi

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W-Shapes

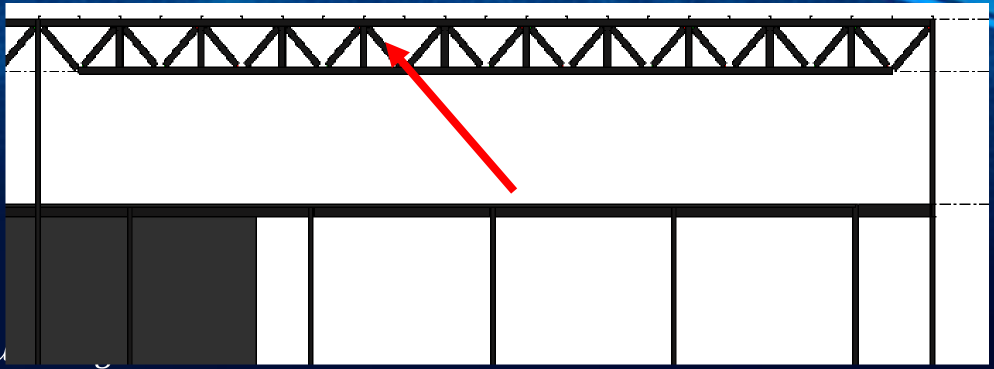
W8

Shape		W8×											
lb/ft		67		58		48		40		35		31	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
s of gyration, r_y	0	590	886	512	769	422	634	350	526	308	463	273	411
	6	542	815	470	706	387	581	320	481	281	423	249	374
	7	526	790	455	685	375	563	309	465	272	409	241	362
	8	508	763	439	660	361	543	298	448	262	394	232	348
	9	488	733	422	634	347	521	285	429	251	377	222	333
	10	467	701	403	606	331	497	272	409	239	359	211	317

$$\frac{fiPn}{Pu} \geq 1$$

Calculations and software results

- Chord at point load design
  - Length = 8.8 ft
  - Load = 6.7 kips
  - $Ag = \frac{6.7}{0.9 * 50} = 0.15\ in^2 \Rightarrow Try\ W4 * 13$
  - $4.71 * \sqrt{\frac{29000}{50}} = 113.4 > \frac{KL}{r} = \frac{1 * 8.78 * 12}{1} = 105.36 \Rightarrow Inelastic\ Flexural\ Bu$



  - $Fe = \frac{\pi^2 * 29000}{105.36^2} = 25.78\ ksi \Rightarrow Fcr = (0.658)^{\frac{50}{25.78}} * 50 = 22.2\ ksi$
  - $Ag = \frac{6.7}{0.9 * 22} = 0.34\ in^2 \Rightarrow Use\ W4 * 13$
  - Check LB and FTB
    - Lb<Lp<Lr → No Lateral Torsional Buckling
    - λφφ<λρφ
    - λφω<λρω

No Local Buckling

$Ag \geq 0.34\ in^2$

Calculations and software results

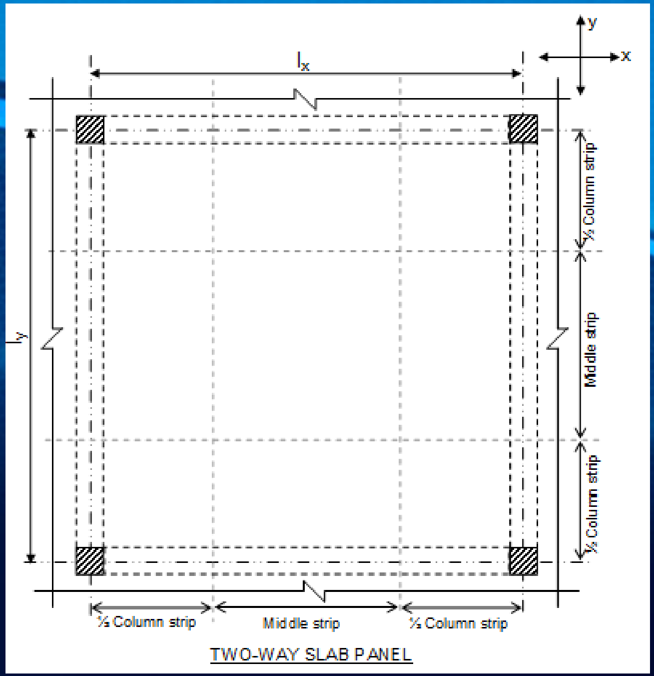
- Slab design: Ground & 1st floor
  - $\frac{L}{S} = 1 < 2 \Rightarrow Two\ way\ slab$
  - Minimum thickness:
    - Average Stiffness ratio = 0.2
    - $h \geq \begin{cases} \text{5 in.} \\ \text{or} \\ \text{11.6 in.} \end{cases}$
  - Calculate total factored load Wu: 263 lbsf
  - Determine the total factored static Moment
    - $Mu = \frac{Wu*L2*Ln^2}{8}$ = 1090 kips.ft

Table 8.3.1.2—Minimum thickness of nonprestressed two-way slabs with beams spanning between supports on all sides

α_{fm}[1]	Minimum h, in.		
$\alpha_{fm} \leq 0.2$	8.3.1.1 applies		(a)
$0.2 < \alpha_{fm} \leq 2.0$	Greater of:	$\frac{\ell_n\left(0.8+\frac{f_y}{200,000}\right)}{36+5\beta(\alpha_{fm}-0.2)}$	(b)[2],[3]
		5.0	(c)
$\alpha_{fm} > 2.0$	Greater of:	$\frac{\ell_n\left(0.8+\frac{f_y}{200,000}\right)}{36+9\beta}$	(d)[2],[3]
		3.5	(e)

Calculations and software results

- Determine column and middle strip widths
  - One column strip width = $\frac{L}{4} = 8.2\ ft$
  - Middle strip width = $\frac{L}{2} = 16.4\ ft$
- Distribute Moment into positive and negative moments
  - +ve Moment = 0.35*Mu = 376.95 $K.ft$
    - Portion of positive moment in column strip = $0.6 * 377 = 226.2\ K.ft$
    - Portion of positive moment in Middle strip = 150.8 $K.ft$
  - -ve Moment = 0.65*Mu = 700 $K.ft$
    - Portion of negative moment in column strip = $0.75 * 700 = 525\ K.ft$
    - Portion of negative moment in middle strip = 175 $K.ft$
- Design reinforcements for the moments
  - ρ
  - Area of steel required



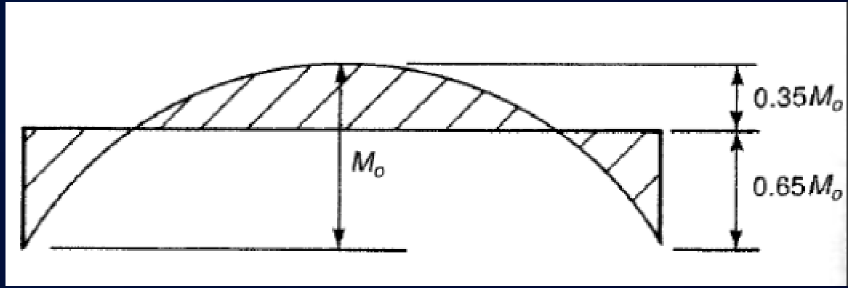


Table 8.10.5.5—Portion of positive M_u in column strip

$\alpha_{f1}\ell_2/\ell_1$	ℓ_2/ℓ_1 0.5	1.0	2.0
0	0.60	0.60	0.60
≥1.0	0.90	0.75	0.45

Table 8.10.5.1—Portion of interior negative M_u in column strip

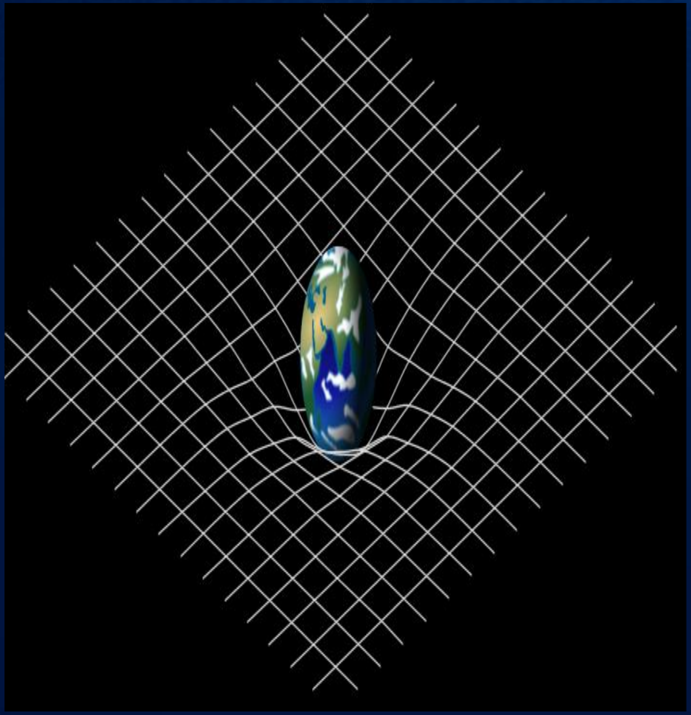
$\alpha_{f1}\ell_2/\ell_1$	ℓ_2/ℓ_1 0.5	1.0	2.0
0	0.75	0.75	0.75
≥1.0	0.90	0.75	0.45

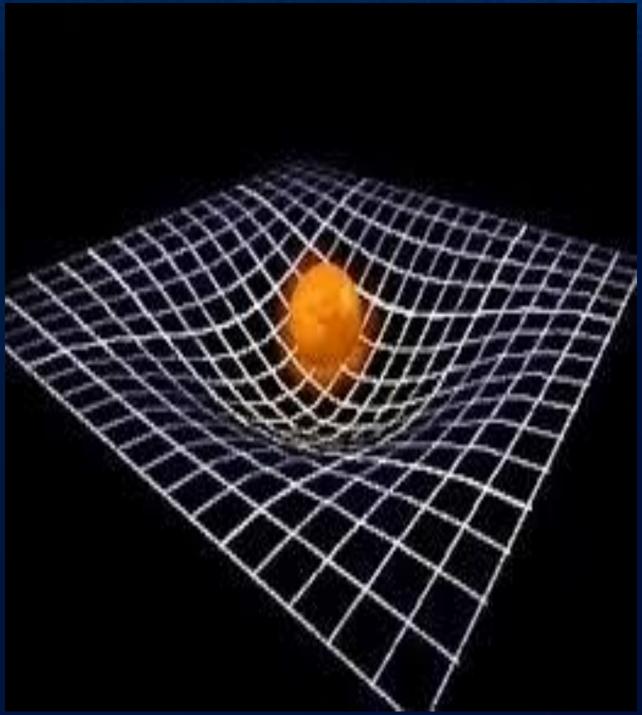
Calculations and software results

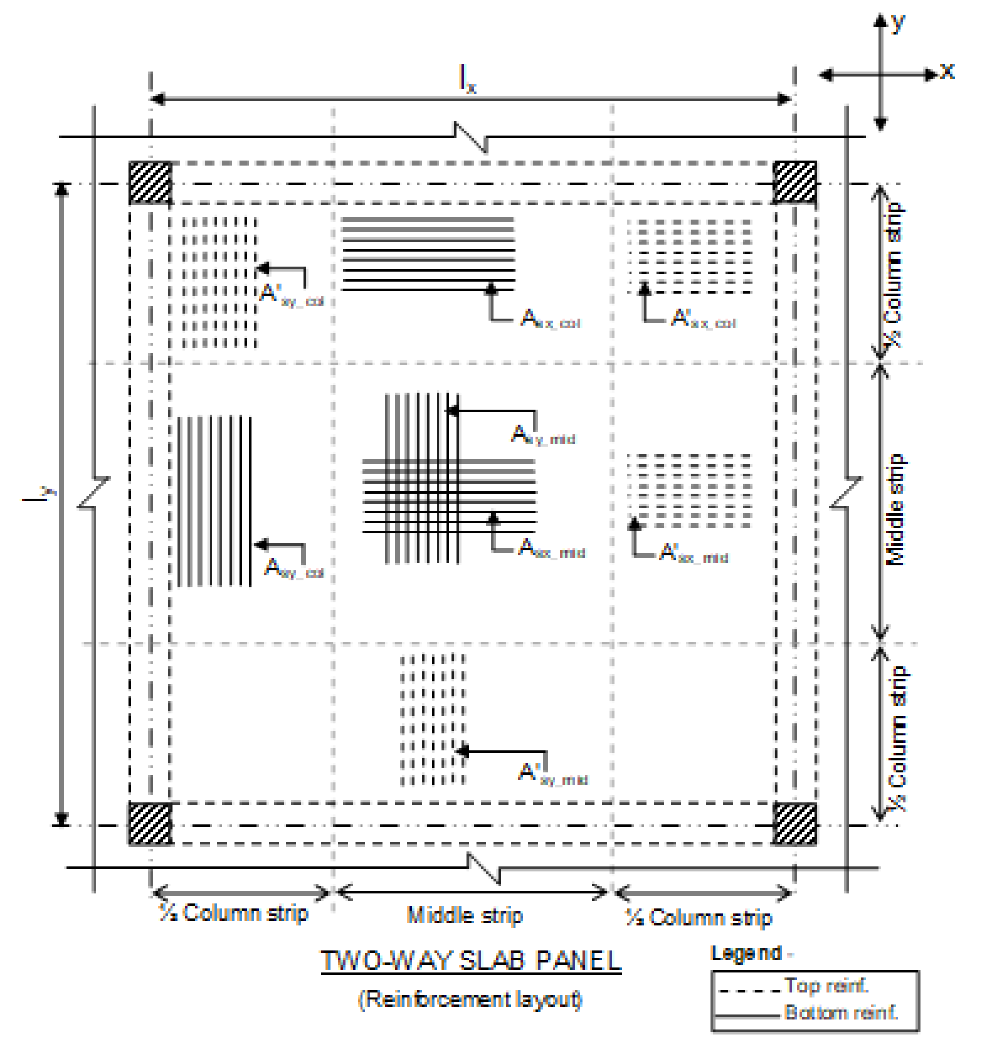
	Column Strip (X-direction)	Column Strip (Y-direction)	Middle Strip (X-direction)	Middle Strip (Y-direction)
Moment +ve (kip*ft)	227.46	227.46	149.8	149.8
Moment -ve (kip*ft)	525.51	525.51	175.17	175.17
ρ required +ve (Bottom)	0.00167	0.00180	0.00205	0.00214
ρ required –ve (Top)	0.004	0.0044	0.00123	0.00133
Area of steel required (Top)	0.605	0.638	0.16	0.167
Area of steel Provided (Top) in^2	0.614; #5 @ 6 in	0.736; #5 @ 5	0.337; #4 @ 7	0.337; #4 @ 7
Area of steel required (Bottom) in^2	0.256	0.267	0.188	0.196
Area of steel Provided (Bottom) in^2	0.337; #4 @ 7 in	0.337; #4 @ 7	0.337; #4 @ 7	0.337; #4 @ 7

Calculations and software results

- Visualizing the results

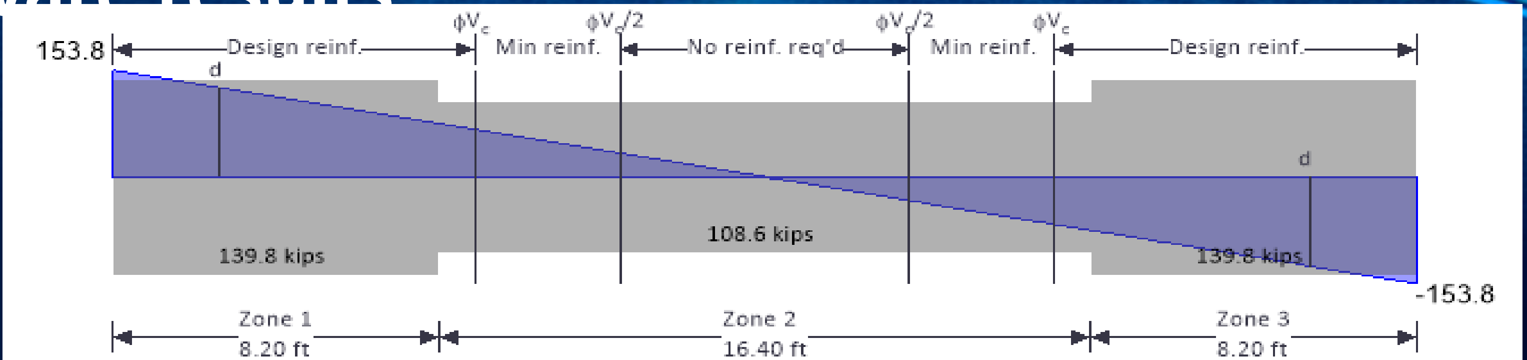


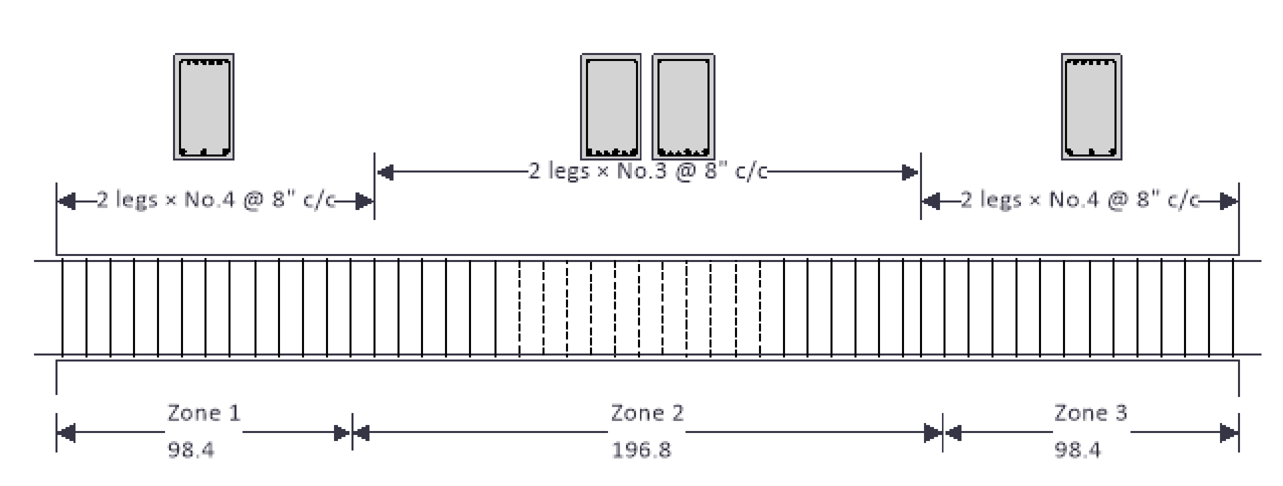




Calculations and software results

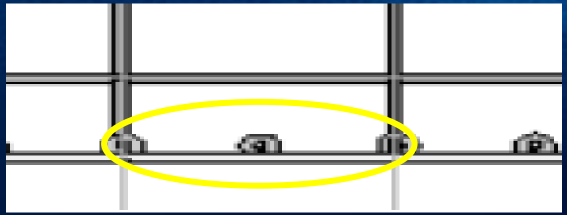
- **Shear design of beam:**
  1. Shear force at “d” inches from support:
     - d = 32.28 in.
       – Shear Force at d:
         » $Vu = 128.587\ kips$
     - **Shear force in concrete:**
       – $\phi Vc = 0.75 * 2 * \sqrt{F'c} * b * d = 68.474\ kips$
     - **Reinforcement shear strength required:**
       – $\phi Vs = Vu - \phi Vc = 60.113\ kips$
     - **Area of shear reinforcement required:**
       – $\phi Vs = \frac{\phi * Asv * Fy * d}{s} \Rightarrow Asv = 0.497\ in^2$
       – $Asmin = 0.212\ in^2$
       – $As\ provided = 0.589\ in^2 \Rightarrow 2\#4$ @ 8 in
       – $Smax \le \min\left(12\ in, \frac{d}{2} = 16.14\right) = 12\ in$
       – Smin = $\frac{4}{3}$*D agg = 1.33 in.



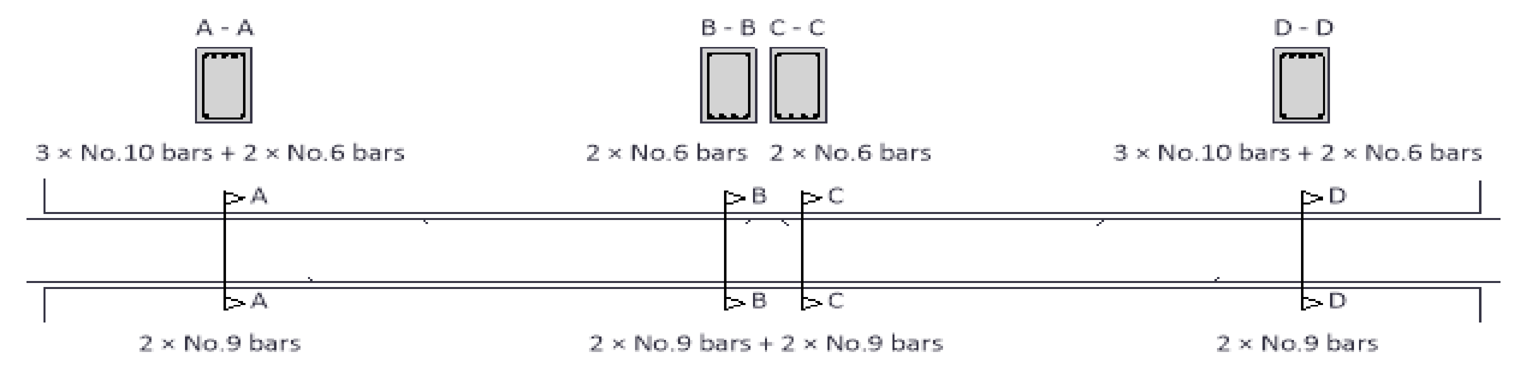


Calculations and software results

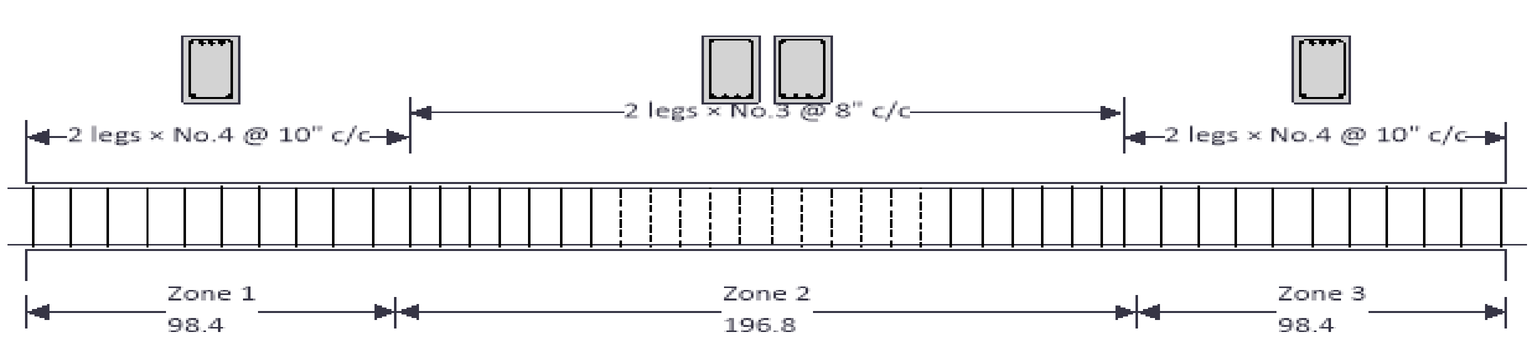
Reinforcement detail for edge beam:



Main reinforcement



Shear reinforcement

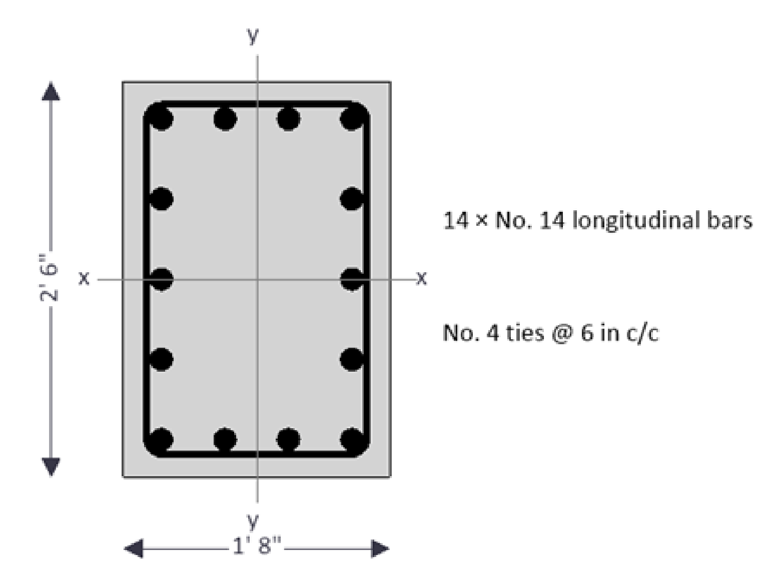
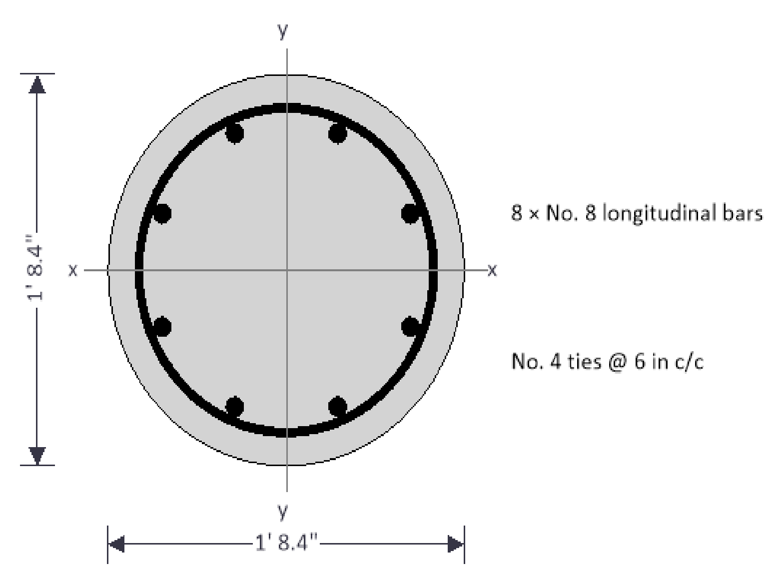
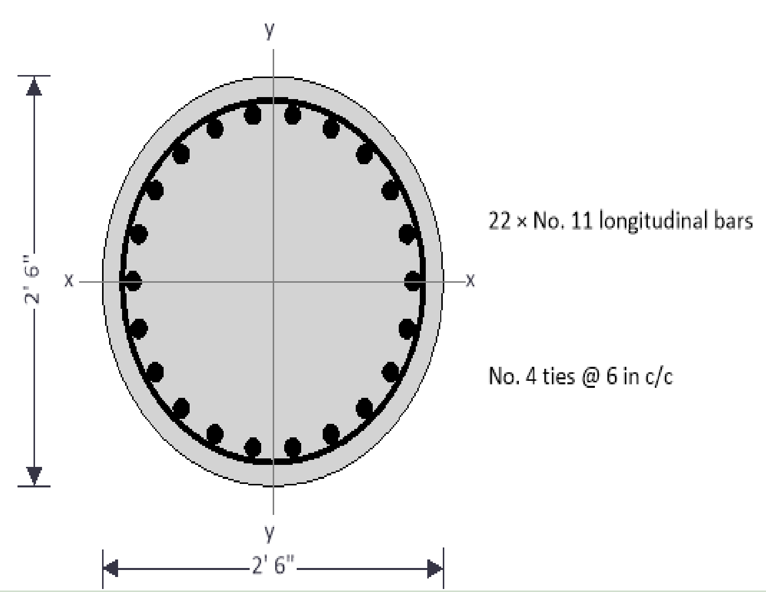


Calculations and software results

- **Column Design**
  - Side column

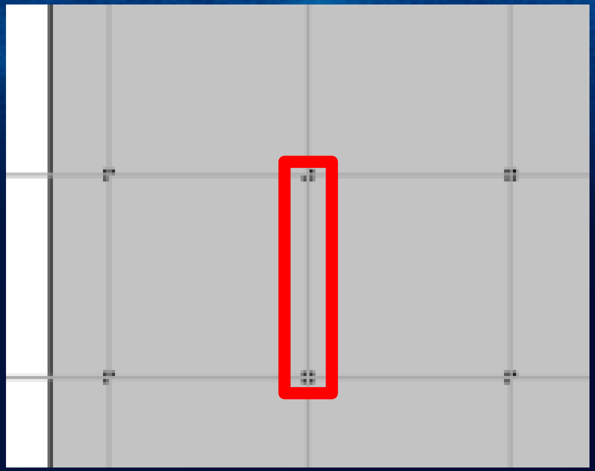
1. Assume short column
2. Calculate Pu = 673 kips
3. Assume ρ = $\frac{A\,steel}{A\,gross} = 0.02$
   1. $Pn = 0.85 * F'c * (Ag - Ast) + Fy * Ast\,; \Rightarrow get\ Ag = 600\ in^2$ (20 in $*$ 30 in)
4. Check if slender:
   1. $\frac{K*L}{r} = \frac{1*10*12}{0.3*30} = 13.34 \leq 22$
5. Find $\gamma h = h\ - 2 * cover\ - 2 * d.stirrup\ - d.bar = 24.625$
   1. $\gamma = \frac{24.625}{30} = 0.82$
   2. $\frac{\phi Mn}{b*h^2} = \frac{1262*12}{20*30^2} = 0.84\,; \frac{\phi Pn}{b*h} = \frac{826}{20*30} = 1.3767$
6. Using P-M diagrams
   - γ required = 0.049
   - Area of steel required = 29.4 in^2
7. Spacing requirement
   1. S provided = 4 in (center to center)
   2. S min = max (1.5* db = 2.06 in. , 1.5 in, 0.75*Daggregate = 1.33in.) = 2.06 < 4 in
8. Transverse reinforcement:
   1. #4 bars at 6 inches
   2. Max Spacing = 16 inches

Calculations and software results

Side column reinforcement detail	Center column reinforcement detail	Corner column reinforcement detail
		

Steel Beam design

- Length $= 32.8$ ft
- Wu $= 8.55$ Kips/ft
- Mu $= \frac{Wu*L^{\wedge}2}{8} = 1150$ K. ft; Cb =1.14
- Try W27*178 →$\phi Mn = 1435$ K. ft
  - Mu new = 1179 K.ft < ϕMn
- $\phi Vn = 605$ kips $>$ Vu $= 143.5$ kips
- Deflection check:
  - Allowable Live load deflection = $\frac{L}{360} = 1.1$ in.
  - Max. Live load deflection =$\frac{5*1.32*(32.812)^{\wedge}4}{384*29000*5660} = 0.21$ in. $<$ Allowable deflection



Steel Column design

- Tributary Area = 1076 ft^2 = 32.8 * 32.8
- Wu = 8.5 K/ft.
- Pu = 292 kips (including beam self weight)
- Column Height = 32.2 ft.
- Use HSS10*10*1/2
  - ϕPn = 361.8

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 50 ksi

HSS12–HSS10

Shape	HSS12×12×		HSS10×10×									
	3/16[c]		5/8		1/2		3/8		5/16		1/4[c]	
t_{des}, in.	0.174		0.581		0.465		0.349		0.291		0.233	
lb/ft	29.84		76.33		62.46		47.90		40.35		32.63	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
0	149	225	629	945	515	774	395	594	332	499	241	362
6	148	223	612	921	502	755	386	580	324	487	237	357
7	148	222	607	912	497	748	382	574	321	483	236	354
8	147	221	600	902	492	740	378	569	318	478	234	352
9	146	220	593	891	486	731	374	562	315	473	232	349
10	146	219	585	879	480	721	369	555	311	467	231	346
11	145	217	576	865	473	711	364	547	306	460	228	343
12	144	216	566	851	465	699	358	538	301	453	226	340
13	143	215	556	835	457	687	352	529	296	445	223	336
14	142	213	545	819	448	674	346	519	291	437	221	332
15	141	211	534	802	439	660	339	509	285	429	218	327
16	139	210	522	784	430	646	332	498	279	420	215	323
17	138	208	509	765	420	631	324	487	273	411	212	318
18	137	206	496	746	410	616	317	476	267	401	208	313
19	136	204	483	726	399	600	309	464	260	391	205	308
20	134	202	470	706	388	583	300	452	253	381	201	302
21	133	199	456	685	377	567	292	439	246	370	197	297
22	131	197	442	664	366	550	284	426	239	360	193	291
23	129	195	428	643	354	533	275	413	232	349	188	283
24	128	192	413	621	343	515	266	400	225	338	183	274
25	126	190	399	599	331	498	258	387	218	327	177	266
26	124	187	384	577	319	480	249	374	210	316	171	257
27	123	184	370	555	308	462	240	360	203	305	165	248
28	121	181	355	534	296	445	231	347	195	293	159	239
29	119	179	341	512	284	427	222	334	188	282	153	230
30	117	176	326	490	273	410	213	321	180	271	147	221
32	113	170	298	448	250	375	196	294	166	249	135	203
34	109	163	271	407	228	342	179	269	152	228	124	186
36	105	157	244	367	206	[illegible]	163	244	138	207	113	170
38	100	151	219	329	185	278	147	220	125	187	102	153
40	95.9	144	198	297	167	251	132	199	112	169	92.1	138

Effective length, L_c (ft), with respect to least radius of gyration, r_y

Properties

Base Plate Design

Pu = 214.8 kips

1. **Check for concrete crushing**

– $Pn = \phi * 0.85 * F'c * A1 * \sqrt{\frac{A2}{A1}} = 0.65 * 0.85 * 5 * A1 * \sqrt{\frac{326.851}{A1}}$

- A2: Concrete column area
- A1: Plate area = 18.49 in^2

– $\sqrt{\frac{A2}{A1}} = 4.2 > 2 \,!! \Rightarrow use \sqrt{\frac{A2}{A1}} = 2$

- 214.8 = $0.65 * 0.85 * 5 * A1 * 2 \Rightarrow A1 = 38.87\ in^2 \Rightarrow B = N = 6.23\ in. (Not\ good, HSS\ 10 * 10 * \frac{1}{2}, use\ B = N = 12\ in.)$

2. **Check for plate bending**

1. $m = \frac{N-0.95*d}{2} = \frac{12-0.95*10}{2} = 1.25$
2. $n = \frac{B-0.8*b}{2} = \frac{12-0.8*10}{2} = 2$
3. $n' = \frac{\sqrt{d*b}}{4} = \frac{\sqrt{10*10}}{4} = 2.5$

$l = 2.5$

Plate Thickness t = $\sqrt{\frac{2*214.8*2.5\text{^}2}{0.9*50*144}} = 0.6437\ in$

Use $PL\ 12 * 12 * {}^{3}/_{4}$

Appendices Part 2

Software and Coding Language

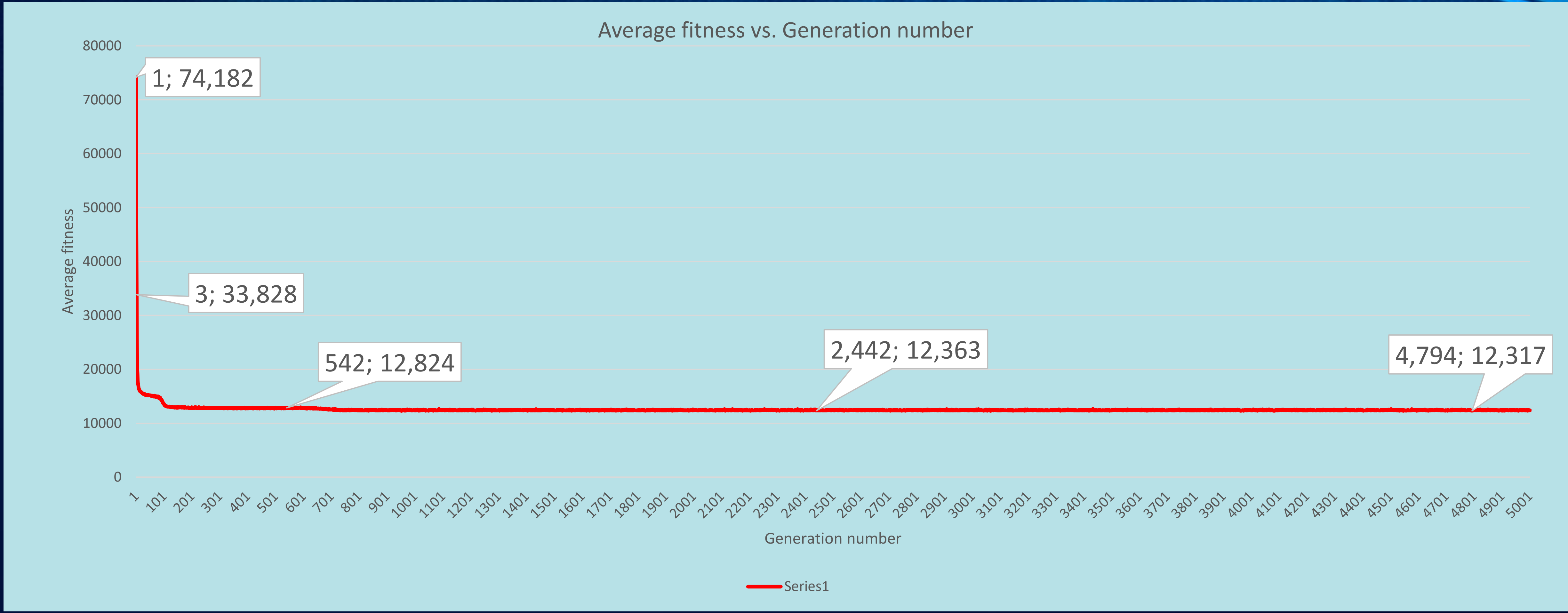
- Software used
  - IntelliJ Idea
- Coding language
  - Java

Elitism

Forcing the best of the previous population into the next population

# of Generations	# of individuals	Crossover rate	Mutation Prob.	Elitism	Average fitness
500	250	100%	0.1%	Best 20%	13,402.2312
500	250	100%	0.1%	Best 10%	13,114.412
500	250	100%	0.1%	Best individual	12,687.0103

Results


Average fitness vs. Generation number
80000
70000
60000
50000
40000
30000
20000
10000
0
Average fitness
1; 74,182
3; 33,828
542; 12,824
2,442; 12,363
4,794; 12,317
1 101 201 301 401 501 601 701 801 901 1001 1101 1201 1301 1401 1501 1601 1701 1801 1901 2001 2101 2201 2301 2401 2501 2601 2701 2801 2901 3001 3101 3201 3301 3401 3501 3601 3701 3801 3901 4001 4101 4201 4301 4401 4501 4601 4701 4801 4901 5001
Generation number
Series1