CASE Engineering Inc.
796 Merus Ct
St. Louis, MO 63026
OFFICE: 636-349-1600
FAX: 636-349-1730
Website: www.caseengineeringinc.com

Structural Calculations

Client:

Wingstop Restaurants, Inc

Project:

Wingstop Restaurant

1041 NE Sam Walton Lane
Lee's Summit, MO 64086

Project # WIL-MO-04-21

Date: June 16, 2021

STATE OF MISSOURI
STEPHEN J. SACCO
NUMBER
PE-21871
PROFESSIONAL ENGINEER

6/16/21

Hazards by Location

Search Information

Address: 1041 NE Sam Walton Lane Lee's Summit, MO 64086

Coordinates: 38.9312921, -94.36108949999999

Elevation: 993 ft

Timestamp: 2021-06-08T18:26:54.111Z

Hazard Type: Wind

ASCE 7-16

MRI 10-Year	76 mph
MRI 25-Year	83 mph
MRI 50-Year	88 mph
MRI 100-Year	94 mph
Risk Category I	103 mph
Risk Category II	109 mph
Risk Category III	117 mph
Risk Category IV	122 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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Hazards by Location

Search Information

Address: 1041 NE Sam Walton Lane Lee's Summit, MO 64086

Coordinates: 38.9312921, -94.36108949999999

Elevation: 993 ft

Timestamp: 2021-06-08T18:28:00.410Z

Hazard Type: Snow

ASCE 7-16

Ground Snow Load ---------- 20 lb/sqft

ASCE 7-10

Ground Snow Load ------ 20 lb/sqft

ASCE 7-05

Ground Snow Load ---------- 20 lb/sqft

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Hazards by Location

Search Information

Address: 1041 NE Sam Walton Lane Lee's Summit, MO 64086

Coordinates: 38.9312921, -94.36108949999999

Elevation: 993 ft

Timestamp: 2021-06-08T18:28:40.827Z

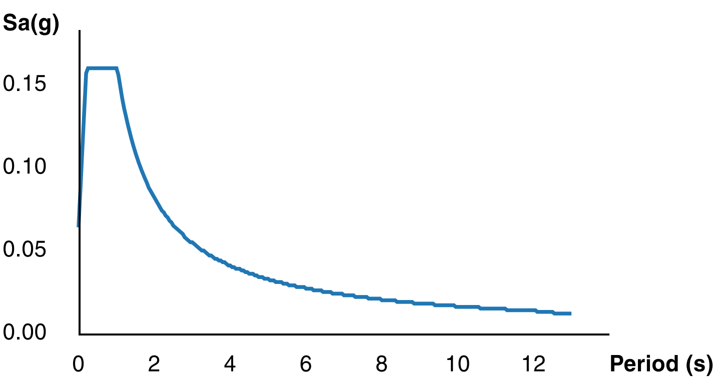
Hazard Type: Seismic

Reference Document: ASCE7-16

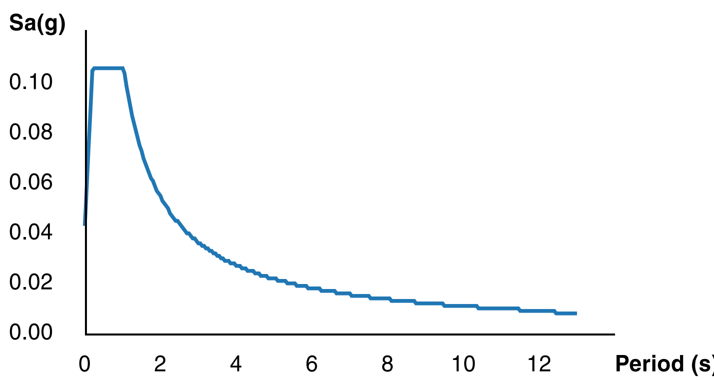
Risk Category: II

Site Class: D-default

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.1	MCE_R ground motion (period=0.2s)
S_1	0.068	MCE_R ground motion (period=1.0s)
S_{MS}	0.16	Site-modified spectral acceleration value
S_{M1}	0.164	Site-modified spectral acceleration value
S_{DS}	0.106	Numeric seismic design value at 0.2s SA
S_{D1}	0.109	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s

CR_S	0.927	Coefficient of risk (0.2s)
CR_1	0.876	Coefficient of risk (1.0s)
PGA	0.047	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.076	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	0.1	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.108	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.068	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.078	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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Code Search

Code: International Building Code 2018

Occupancy:

Occupancy Group = B Business

Risk Category & Importance Factors:

Risk Category =	II
Wind factor =	1.00
Snow factor =	1.00
Seismic factor =	1.00

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	0.25 / 12	1.2 deg
Building length	160.0 ft	
Least width	70.0 ft	
Mean Roof Ht (h)	17.0 ft	
Parapet ht above grd	23.0 ft	
Minimum parapet ht	3.0 ft	

Live Loads:

Roof

0 to 200 sf:	20 psf
200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
over 600 sf:	12 psf

Floor:

Typical Floor	N/A
Partitions	N/A

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JOB TITLE	Wingstop Restaurant		
	Lee's Summit, MO		
JOB NO.	WIL-MO-04-21	**SHEET NO.**	
CALCULATED BY	K. Woodard	**DATE**	6/10/21
CHECKED BY	CRH	**DATE**	

Wind Loads : ASCE 7- 16

Ultimate Wind Speed	109 mph
Nominal Wind Speed	84.4 mph
Risk Category	II
Exposure Category	B
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.701
Kh case 2	0.596
Type of roof	Monoslope

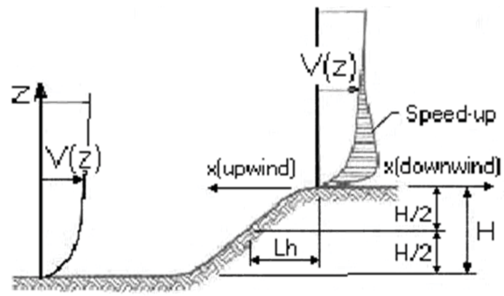
Topographic Factor (Kzt)

Topography	Flat
Hill Height (H)	80.0 ft
Half Hill Length (Lh)	100.0 ft
Actual H/Lh =	0.80
Use H/Lh =	0.50
Modified Lh =	160.0 ft
From top of crest: x =	50.0 ft
Bldg up/down wind?	downwind

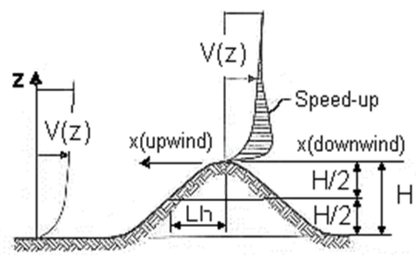
H/Lh= 0.50	K_1 = 0.000
x/Lh = 0.31	K_2 = 0.792
z/Lh = 0.11	K_3 = 1.000

At Mean Roof Ht:

$Kzt = (1+K_1K_2K_3)^2 =$ 1.00



ESCARPMENT



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h =	17.0 ft
B =	70.0 ft
/z (0.6h) =	30.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 0.24 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure formula

Rigid Structure

ē =	0.33
ℓ =	320 ft
z_{min} =	30 ft
c =	0.30
g_Q, g_v =	3.4
L_z =	310.0 ft
Q =	0.88
I_z =	0.30
G =	**0.86** use G = 0.85

Flexible or Dynamically Sensitive Structure

Natural Frequency (η_1) =	0.0 Hz				
Damping ratio (β) =	0				
/b =	0.45				
/α =	0.25				
Vz =	70.2				
N_1 =	0.00				
R_n =	0.000				
R_h =	28.282	η =	0.000	h =	17.0 ft
R_B =	28.282	η =	0.000		
R_L =	28.282	η =	0.000		
g_R =	0.000				
R =	0.000				
Gf =	0.000				

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JOB TITLE	Wingstop Restaurant
	Lee's Summit, MO
JOB NO.	WIL-MO-04-21
SHEET NO.	
CALCULATED BY	K. Woodard
DATE	6/10/21
CHECKED BY	CRH
DATE	

Snow Loads : ASCE 7- 16

Nominal Snow Forces

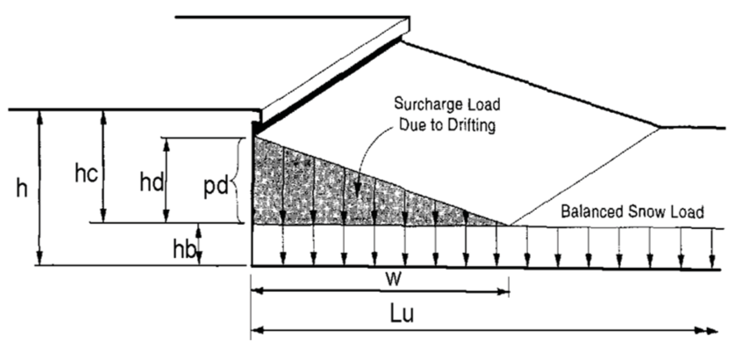
Roof slope		=	1.2 deg
Horiz. eave to ridge dist (W)		=	70.0 ft
Roof length parallel to ridge (L)		=	160.0 ft
Type of Roof			Monoslope
Ground Snow Load	Pg	=	20.0 psf
Risk Category		=	II
Importance Factor	I	=	1.0
Thermal Factor	Ct	=	1.00
Exposure Factor	Ce	=	1.0
Pf = 0.7*Ce*Ct*I*Pg		=	14.0 psf
Unobstructed Slippery Surface			no
Sloped-roof Factor	Cs	=	1.00
Balanced Snow Load		**=**	**14.0 psf**
Rain on Snow Surcharge Angle			1.40 deg
Code Maximum Rain Surcharge			5.0 psf
Rain on Snow Surcharge		=	5.0 psf
Ps plus rain surcharge		=	19.0 psf
Minimum Snow Load	Pm	=	20.0 psf
Uniform Roof Design Snow Load		=	**20.0 psf**

Near ground level surface balanced snow load = **20.0 psf**

NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs..

Windward Snow Drifts 1 - Against walls, parapets, etc

Up or downwind fetch	lu	=	0.0 ft
Projection height	h	=	0.0 ft
Projection width/length	lp	=	0.0 ft
Snow density	g	=	16.6 pcf
Balanced snow height	hb	=	0.84 ft
	hd	=	0.92 ft
	hc	=	-0.84 ft
hc/hb <0.2 = -1.0	**lp <15', drift not req'd**		
Drift height (hc)		=	0.00 ft
Drift width	w	=	-6.75 ft
Surcharge load:	pd = γ*hd	=	**0.0 psf**
Balanced Snow load:		=	14.0 psf
			14.0 psf



Note: If bottom of projection is at least 2 feet above hb then snow drift is not required.

Windward Snow Drifts 2 - Against walls, parapets, etc

Up or downwind fetch	lu	=	0.0 ft
Projection height	h	=	0.0 ft
Projection width/length	lp	=	0.0 ft
Snow density	g	=	16.6 pcf
Balanced snow height	hb	=	0.84 ft
	hd	=	0.92 ft
	hc	=	-0.84 ft
hc/hb <0.2 = -1.0	**lp <15', drift not req'd**		
Drift height (hc)		=	0.00 ft
Drift width	w	=	-6.75 ft
Surcharge load:	pd = γ*hd	=	**0.0 psf**
Balanced Snow load:		=	14.0 psf
			14.0 psf

CASE Engineering Inc. Office: 636-349-1600 Fax: 636-349-1730 Website: www.caseengineeringinc.com	Client:	Wingstop	Project Number:	WIL-MO-04-21
	Location:	Lee's Summit, MO	Date:	6/10/2021
	Subject:	Mechanical Rooftop Unit Weights and Reactions		
	Engineer:	K. Woodard		

Equipment Weight			Factor	Supports	Reaction Load
RTU1:	855	lb	1.3	6	**186**
RTU2:	785	lb	1.3	4	**256**
RTU3:		lb	1.3	4	
MAU:	525	lb	1.2	4	**158**
Hood:	955	lb	1.1	14	***76 or 138**
EF:	218	lb	1.0	4	**55**

*When using this load case a 300 lb additional load must be added to any single hanger rod to account for a worker/tools hanging from the hanger.

Steel Joist Analysis of Irregular Loads J-1:

Joist = 20K4
Length = 34 ft
Trib. Width = 5 ft
E = 29000 ksi
Live Δ_a = L/240

Strength Analysis Results:
Bending Passed
Max DCR = **0.97**
Shear Failed from 0to16, 23to40
Max DCR = **1.19**

Allowable Loads:
Total $Load_a$ = 212 plf (for stress)
Live $Load_a$ = 212 plf (for deflection)

DCR Limits without reinforcement:
Bending: 1.00
Shear: 1.00

Full Uniform Loads:
Dead Load = 15 psf
Live Load = 20 psf

Point Loads:
RTU-2 = 320 lbs
Live $Load_1$ = 0 lbs
$Location_1$ = 13.75 ft
RTU-2 = 320 lbs
Live $Load_2$ = 0 lbs
$Location_2$ = 19.33 ft
Dead $Load_3$= 0 lbs
Live $Load_3$ = 0 lbs
$Location_3$ = 0 ft
Dead $Load_4$= 0 lbs
Live $Load_4$ = 0 lbs
$Location_4$ = 0 ft
Dead $Load_5$= 0 lbs
Live $Load_5$ = 0 lbs
$Location_5$ = 0 ft
Dead $Load_6$= 0 lbs
Live $Load_6$ = 0 lbs
$Location_6$ = 0 ft
Dead $Load_7$= 0 lbs
Live $Load_7$ = 0 lbs
$Location_7$ = 0 ft
Add load = 0 lbs
Live $Load_8$ = 0 lbs
$Location_8$ = 0 ft

Partial Uniform Loads:
Dead $Load_1$= 0 psf
Live $Load_1$ = 0 psf
$Start_1$ = 0 ft
End_1 = 0 ft
Dead $Load_2$= 0 psf
Live $Load_2$ = 0 psf
$Start_2$ = 0 ft
End_2 = 0 ft

Triangular Snow Drift Loads:
+Drift $Load_1$= 0 psf
$Peak_1$ = 0 ft
0psf End_1 = 0 ft
+Drift $Load_2$= 0 psf
$Peak_2$ = 0 ft
0psf End_2 = 0 ft

Is reinforcing reasonable? No
Shear capacity at 1.5 x Depth = 3021

Note: Bar joists are typically constructed symmetrically with equal sized web members along the lengh. As a result the allowable capacity is constant, except at the ends where it decreases until the web is engaged. The accompanying shear diagram is based on the code minimums and does not reflect actual construction and design practices. For this instance, minimum allowable capacity decreases to about 3021 lbs before leveling off. Reinforcement is only required at locations where the applied shear is greater than these allowables.

Joist Reinforcement Design:
T&B Chords: 0.500 Dia. Rods
Fy = 36 ksi
K = 0.65
L = 12 in
DCR = **0.00**
Webs: L2X2X1/8
Fy = 36 ksi
K = 1
DCR = **0.03**

Chord Reinforcement:
Not Required

Web Reinforcement:
(1) - L2X2X1/8 at Each Member
From 8.25 ft to 13.5 ft
And From 20.5 ft to 25.25 ft

Chord Reinforcement Welds:
Not Required

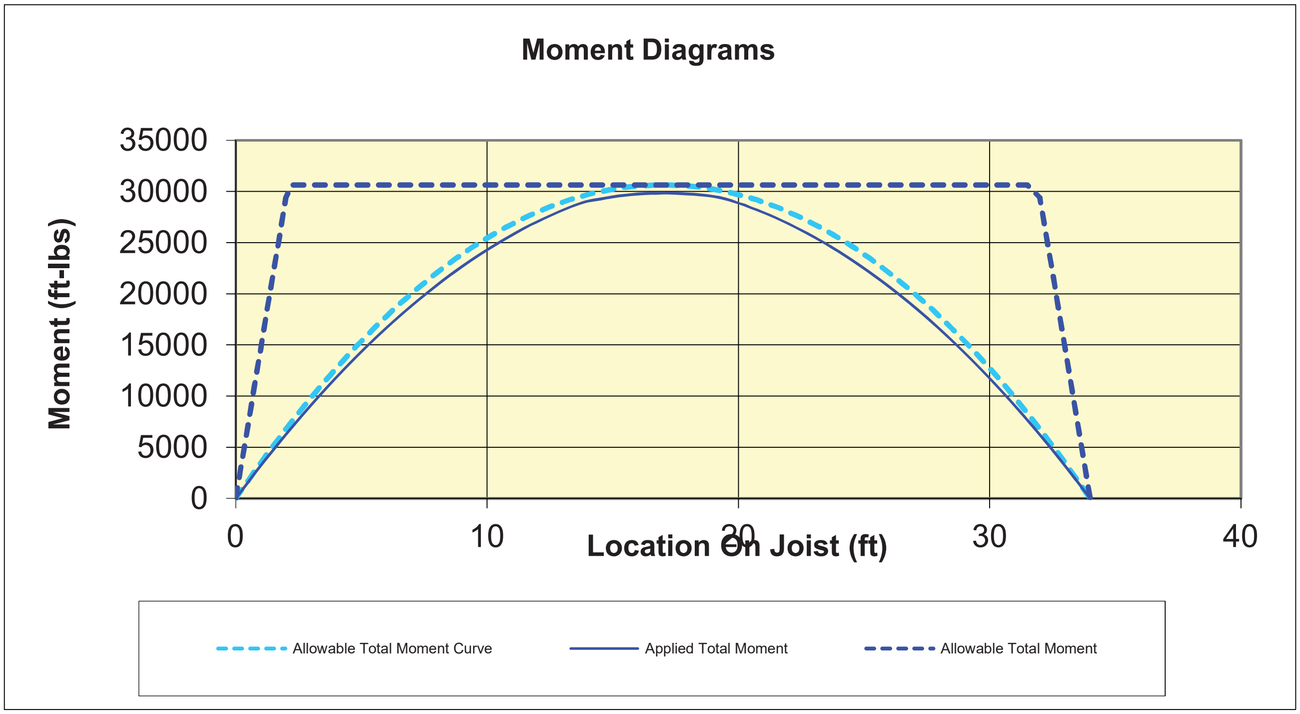
Web Reinforcement:
0.125" x 1" Total Weld at Each End

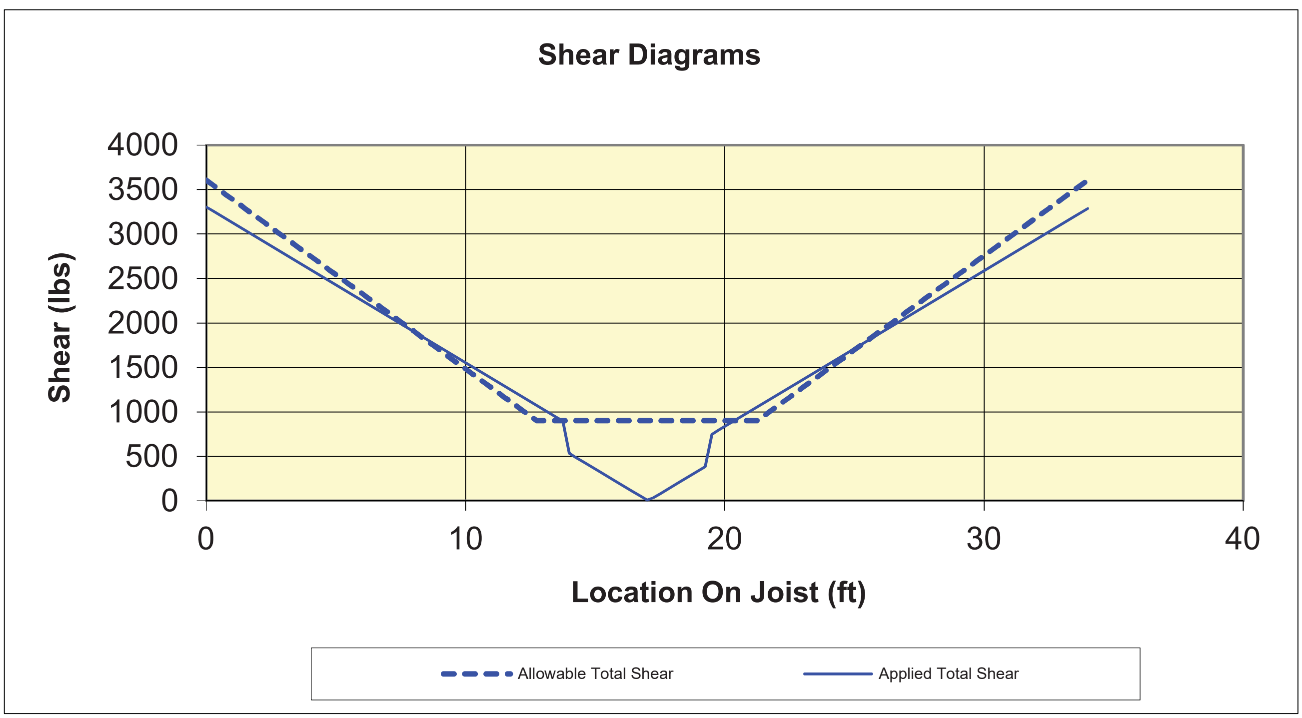
CASE Engineering Inc.
Office: 636-349-1600
Fax: 636-349-1730
Website: www.caseengineeringinc.com

CUSTOMER: Wingstop
LOCATION: Lee's Summit, MO
SUBJECT: Steel Joist Analysis for Mechanical Units
PREPARED BY: K. Woodard File: 0

PROJECT NUMBER: WIL-MO-04-21
DATE: 6/10/2021

Steel Joist Analysis of Irregular Loads J-1:





CASE Engineering Inc. Office: 636-349-1600 Fax: 636-349-1730 Website: www.caseengineeringinc.com	CUSTOMER: Wingstop LOCATION: Lee's Summit, MO SUBJECT: Steel Joist Analysis for Mechanical Units PREPARED BY: K. Woodard	File:	PROJECT NUMBER: WIL-MO-04-21 DATE: 6/10/2021

Steel Joist Analysis of Irregular Loads J-2:

Joist = 20K4
Length = 34 ft
Trib. Width = 5 ft
E = 29000 ksi
Live Δ_a = L/240

Strength Analysis Results:
Bending Failed from 9 to 31
Max DCR = **1.04**
Shear Failed from 0to16, 23to40
Max DCR = **1.31**

Allowable Loads:
Total $Load_a$ = 212 plf (for stress)
Live $Load_a$ = 212 plf (for deflection)

DCR Limits without reinforcement:
Bending: 1.00
Shear: 1.00

Full Uniform Loads:
Dead Load = 15 psf
Live Load = 20 psf

Point Loads:
RTU-2 = 320 lbs
Live $Load_1$ = 0 lbs
$Location_1$ = 13.75 ft
RTU-2 = 320 lbs
Live $Load_2$ = 0 lbs
$Location_2$ = 19.33 ft
EF = 91.85 lbs
Live $Load_3$ = 0 lbs
$Location_3$ = 0.67 ft
EF = 91.85 lbs
Live $Load_4$ = 0 lbs
$Location_4$ = 2.167 ft
Hood = 91.2 lbs
Live $Load_5$ = 0 lbs
$Location_5$ = 3.1 ft
Hood = 15.2 lbs
Live $Load_6$ = 0 lbs
$Location_6$ = 5.04 ft
Hood = 391.2 lbs
Live $Load_7$ = 0 lbs
$Location_7$ = 8.92 ft
Hood = 15.2 lbs
Live $Load_8$ = 0 lbs
$Location_8$ = 9.92 ft

Partial Uniform Loads:
Dead $Load_1$= 0 psf
Live $Load_1$ = 0 psf
$Start_1$ = 0 ft
End_1 = 0 ft
Dead $Load_2$= 0 psf
Live $Load_2$ = 0 psf
$Start_2$ = 0 ft
End_2 = 0 ft

Triangular Snow Drift Loads:
+Drift $Load_1$= 0 psf
$Peak_1$ = 0 ft
0psf End_1 = 0 ft
+Drift $Load_2$= 0 psf
$Peak_2$ = 0 ft
0psf End_2 = 0 ft

Is reinforcing reasonable? No
Shear capacity at 1.5 x Depth = 3021

Note: Bar joists are typically constructed symmetrically with equal sized web members along the lengh. As a result the allowable capacity is constant, except at the ends where it decreases until the web is engaged. The accompanying shear diagram is based on the code minimums and does not reflect actual construction and design practices. For this instance, minimum allowable capacity decreases to about 3021 lbs before leveling off. Reinforcement is only required at locations where the applied shear is greater than these allowables.

Joist Reinforcement Design:
T&B Chords: 0.500 Dia. Rods
Fy = 36 ksi
K = 0.65
L = 12 in
DCR = **0.13**
Webs: L1X1X1/8
Fy = 36 ksi
K = 1
DCR = **0.26**

Chord Reinforcement:
(2) - 0.5" Dia. 36 ksi Rods, T&B
From 13 ft to 19.75 ft

Web Reinforcement:
(1) - L1X1X1/8 at Each Member
From 0 ft to 8.75 ft
And From 11.75 ft to 13 ft
And From 19.75 ft to 28.75 ft

Chord Reinforcement Welds:
(2) - 0.125" x 1" Welds at 12"oc

Web Reinforcement:
0.125" x 1" Total Weld at Each End

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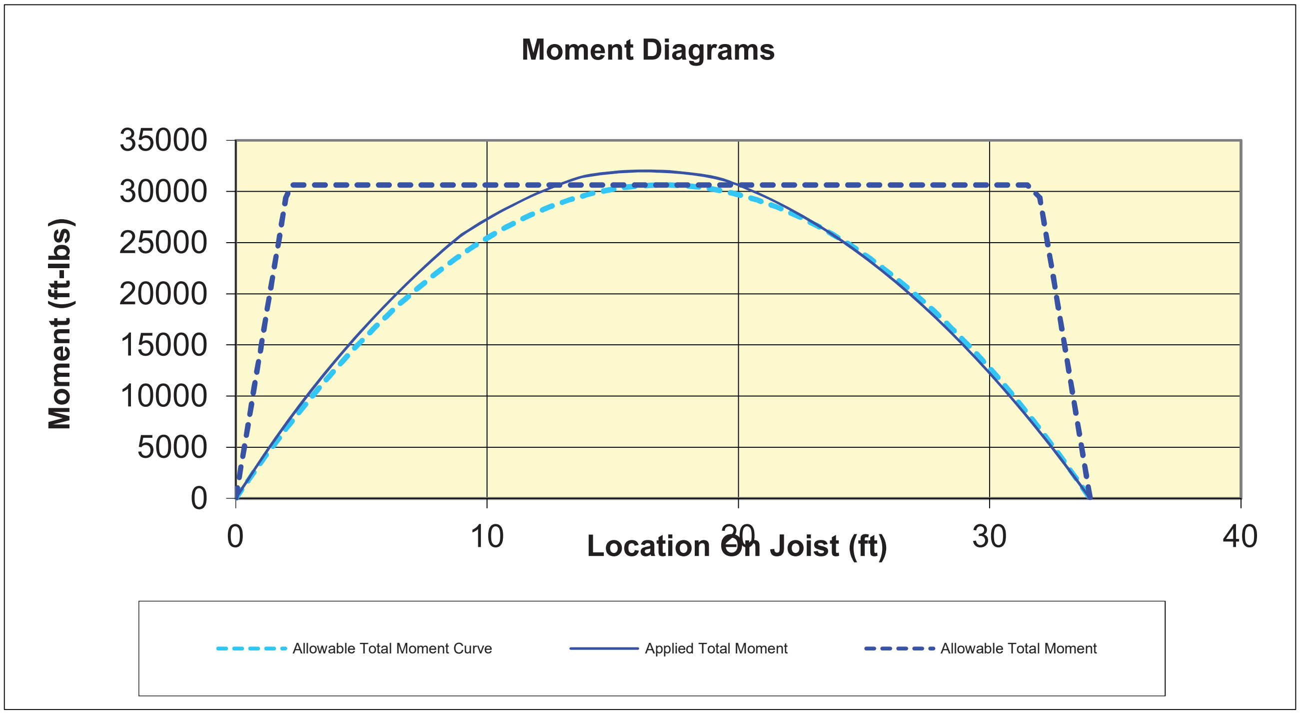
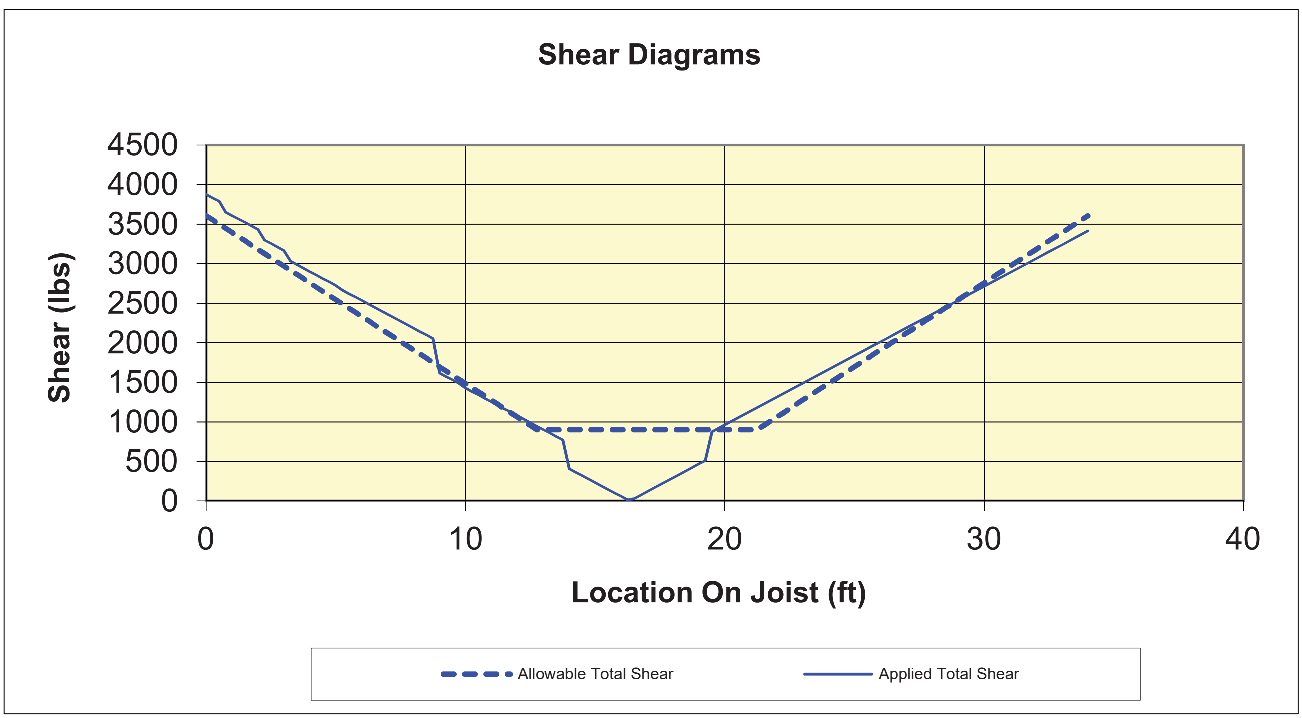
Steel Joist Analysis of Irregular Loads J-2:





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Steel Joist Analysis of Irregular Loads J-3:

Joist = 20K4
Length = 34 ft
Trib. Width = 5 ft
E = 29000 ksi
Live Δ_a = L/240

Strength Analysis Results:
Bending Failed from 9 to 31
Max DCR = **1.05**
Shear Failed from 0to16, 23to40
Max DCR = **1.30**

Allowable Loads:
Total $Load_a$ = 212 plf (for stress)
Live $Load_a$ = 212 plf (for deflection)

DCR Limits without reinforcement:
Bending: 1.00
Shear: 1.00

Full Uniform Loads:
Dead Load = 15 psf
Live Load = 20 psf

Point Loads:
MAU = 263.9 lbs
Live $Load_1$ = 0 lbs
$Location_1$ = 13.25 ft
MAU = 263.9 lbs
Live $Load_2$ = 0 lbs
$Location_2$ = 19.17 ft
EF = 18.15 lbs
Live $Load_3$ = 0 lbs
$Location_3$ = 0.67 ft
Hood+EF = 145.1 lbs
Live $Load_4$ = 0 lbs
$Location_4$ = 2.167 ft
Hood = 76 lbs
Live $Load_5$ = 0 lbs
$Location_5$ = 3.1 ft
Hood = 126.9 lbs
Live $Load_6$ = 0 lbs
$Location_6$ = 5.04 ft
Hood = 76 lbs
Live $Load_7$ = 0 lbs
$Location_7$ = 8.92 ft
Hood = 426.9 lbs
Live $Load_8$ = 0 lbs
$Location_8$ = 9.92 ft

Partial Uniform Loads:
Dead $Load_1$= 0 psf
Live $Load_1$ = 0 psf
$Start_1$ = 0 ft
End_1 = 0 ft
Dead $Load_2$= 0 psf
Live $Load_2$ = 0 psf
$Start_2$ = 0 ft
End_2 = 0 ft

Triangular Snow Drift Loads:
+Drift $Load_1$= 0 psf
$Peak_1$ = 0 ft
0psf End_1 = 0 ft
+Drift $Load_2$= 0 psf
$Peak_2$ = 0 ft
0psf End_2 = 0 ft

Is reinforcing reasonable? No
Shear capacity at 1.5 x Depth = 3021

Note: Bar joists are typically constructed symmetrically with equal sized web members along the lengh. As a result the allowable capacity is constant, except at the ends where it decreases until the web is engaged. The accompanying shear diagram is based on the code minimums and does not reflect actual construction and design practices. For this instance, minimum allowable capacity decreases to about 3021 lbs before leveling off. Reinforcement is only required at locations where the applied shear is greater than these allowables.

Joist Reinforcement Design:
T&B Chords: 0.500 Dia. Rods
Fy = 36 ksi
K = 0.65
L = 12 in
DCR = **0.14**
Webs: L1X1X1/8
Fy = 36 ksi
K = 1
DCR = **0.33**

Chord Reinforcement:
(2) - 0.5" Dia. 36 ksi Rods, T&B
From 12.5 ft to 19.75 ft

Web Reinforcement:
(1) - L1X1X1/8 at Each Member
From 0 ft to 9.75 ft
And From 19.75 ft to 28.5 ft

Chord Reinforcement Welds:
(2) - 0.125" x 1" Welds at 12"oc

Web Reinforcement:
0.125" x 1" Total Weld at Each End

CASE Engineering Inc.
Office: 636-349-1600
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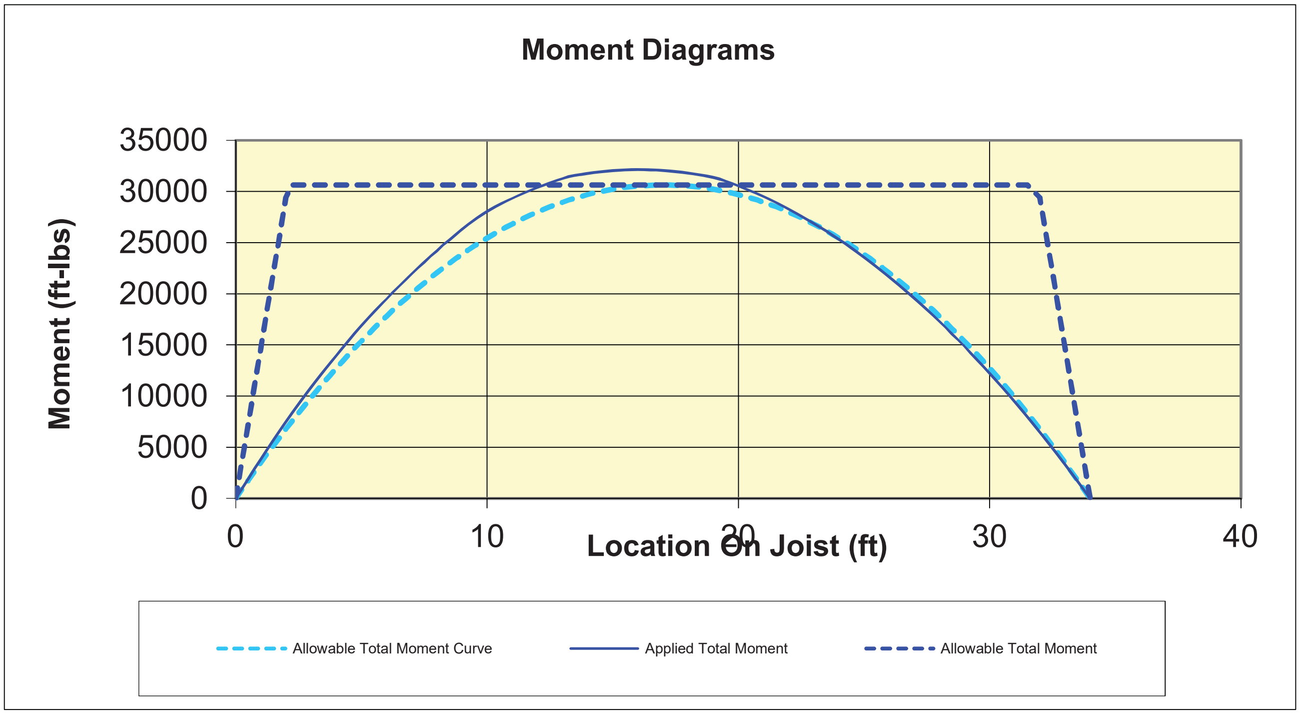
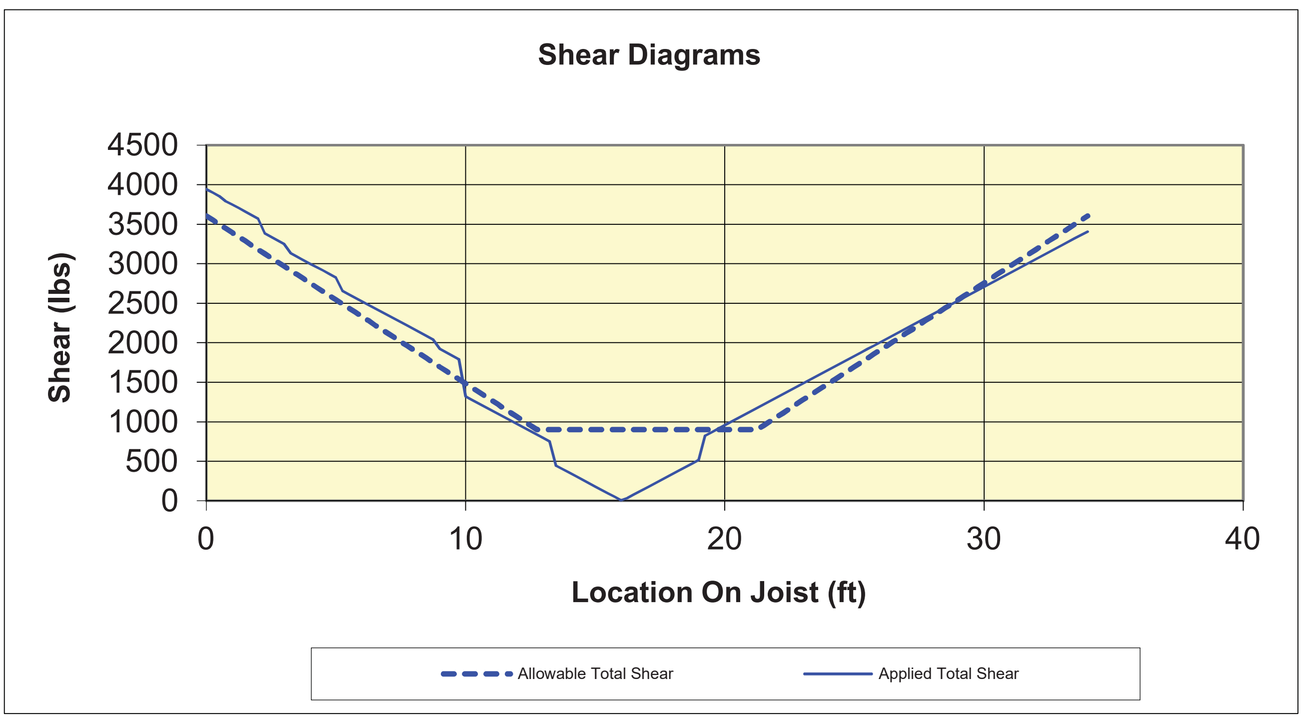
Website: www.caseengineeringinc.com

CUSTOMER:	Wingstop	PROJECT NUMBER:	WIL-MO-04-21
LOCATION:	Lee's Summit, MO	DATE:	6/11/2021
SUBJECT:	Steel Joist Analysis for Mechanical Units		
PREPARED BY:	K. Woodard	File:	0

Steel Joist Analysis of Irregular Loads J-3:





CASE
Engineering Inc.
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Website: www.caseengineeringinc.com

CUSTOMER:	Wingstop	PROJECT NUMBER:	WIL-MO-04-21
LOCATION:	Lee's Summit, MO	DATE:	6/11/2021
SUBJECT:	Steel Joist Analysis for Mechanical Units		
PREPARED BY:	K. Woodard	File:	

Steel Joist Analysis of Irregular Loads J-4:

Joist =	20K4	
Length =	34	ft
Trib. Width =	5	ft
E =	29000	ksi
Live Δ_a =	L/240	

Strength Analysis Results:

Bending	Passed
Max DCR =	**0.89**
Shear	Passed
Max DCR =	**0.98**

Allowable Loads:

Total $Load_a$ =	212	plf (for stress)
Live $Load_a$ =	212	plf (for deflection)

DCR Limits without reinforcement:

Bending:	1.00
Shear:	1.00

Full Uniform Loads:

Dead Load =	15	psf
Live Load =	20	psf

Point Loads:

MAU =	79	lbs
Live $Load_1$ =	0	lbs
$Location_1$ =	13.25	ft
MAU =	79	lbs
Live $Load_2$ =	0	lbs
$Location_2$ =	19.17	ft
RTU-1 =	325.5	lbs
Live $Load_3$ =	0	lbs
$Location_3$ =	1.33	ft
Hood =	25.08	lbs
Live $Load_4$ =	0	lbs
$Location_4$ =	2.167	ft
Dead Load =	0	lbs
Live $Load_5$ =	0	lbs
$Location_5$ =	0	ft
Hood =	25.08	lbs
Live $Load_6$ =	0	lbs
$Location_6$ =	5.04	ft
Dead Load =	0	lbs
Live $Load_7$ =	0	lbs
$Location_7$ =	0	ft
Hood =	124.1	lbs
Live $Load_8$ =	0	lbs
$Location_8$ =	9.92	ft

Partial Uniform Loads:

Dead $Load_1$=	0	psf
Live $Load_1$ =	0	psf
$Start_1$ =	0	ft
End_1 =	0	ft
Dead $Load_2$=	0	psf
Live $Load_2$ =	0	psf
$Start_2$ =	0	ft
End_2 =	0	ft

Triangular Snow Drift Loads:

+Drift $Load_1$=	0	psf
$Peak_1$ =	0	ft
0psf End_1 =	0	ft
+Drift $Load_2$=	0	psf
$Peak_2$ =	0	ft
0psf End_2 =	0	ft

Joist Reinforcement Design:

T&B Chords:	0.500	Dia. Rods
Fy =	36	ksi
K =	0.65	
L =	12	in
DCR =	**0.00**	
Webs:	L2X2X1/8	
Fy =	36	ksi
K =	1	
DCR =	**0.00**	

Chord Reinforcement:

Not Required

Web Reinforcement:

Not Required

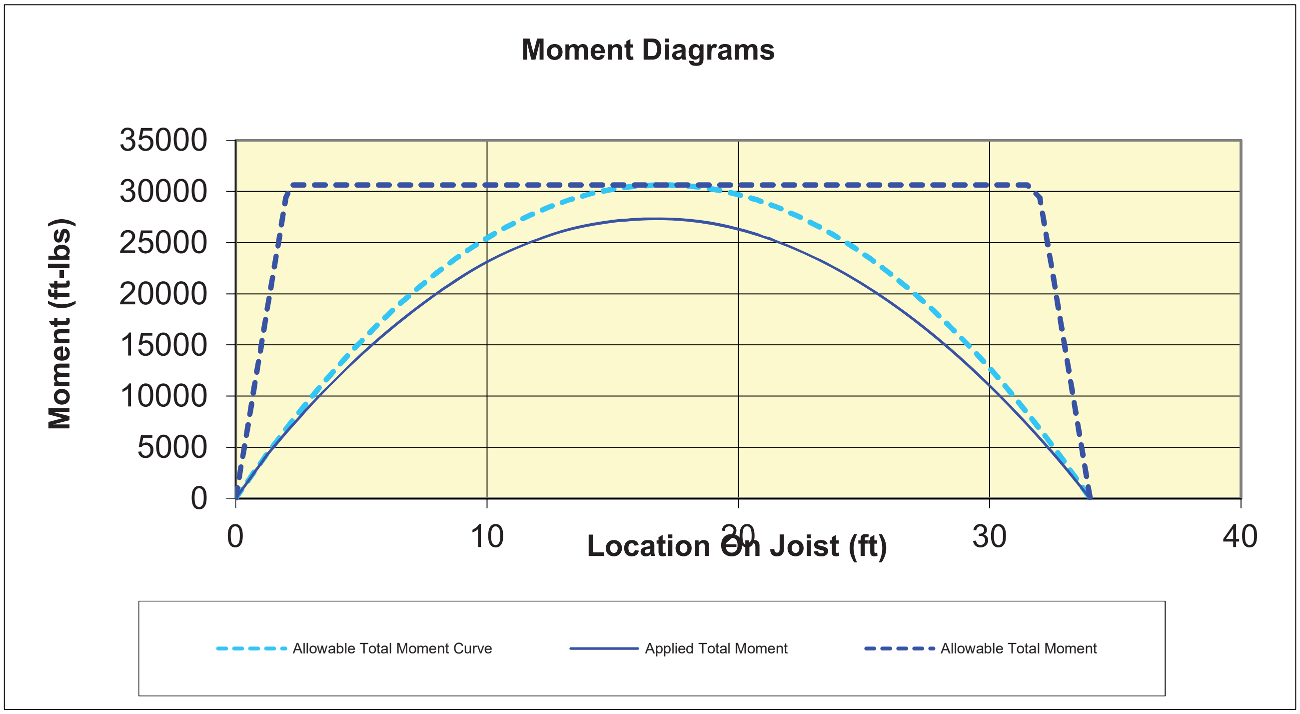
Chord Reinforcement Welds:

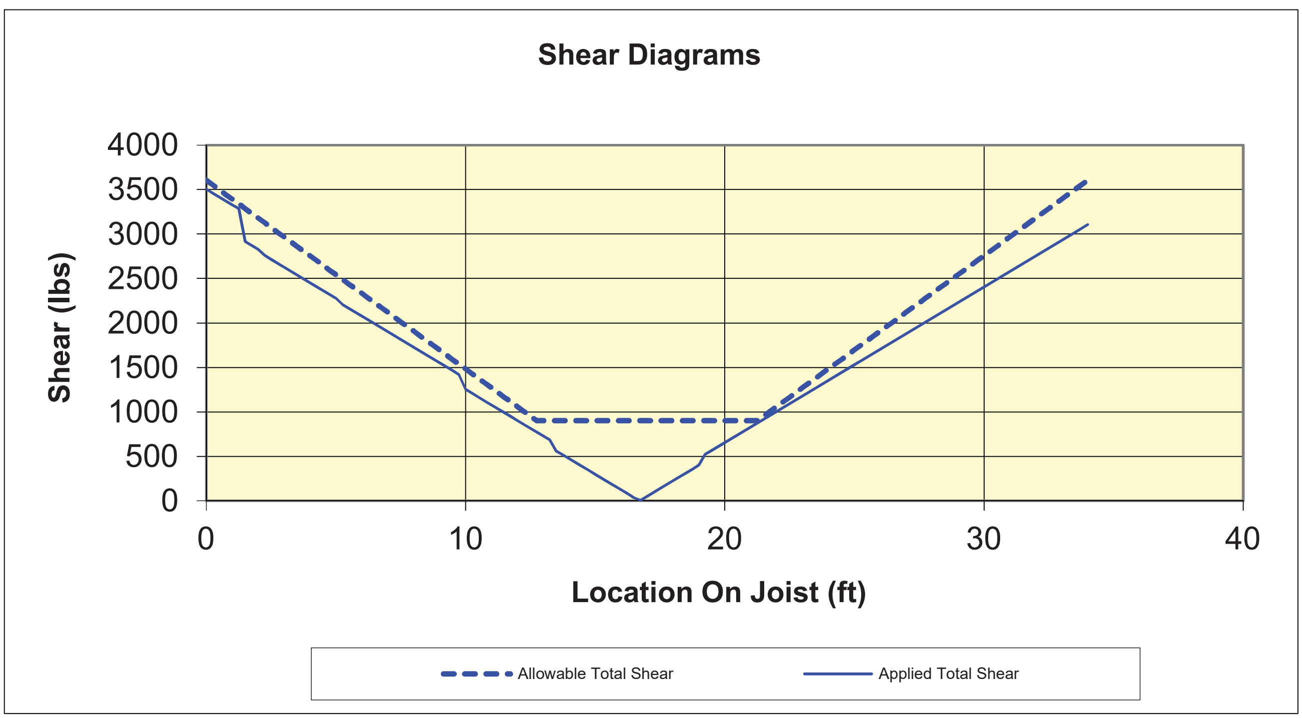
Not Required

Web Reinforcement:

Not Required

Steel Joist Analysis of Irregular Loads J-4:





CASE Engineering Inc.
Office: 636-349-1600
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Website: www.caseengineeringinc.com

CUSTOMER: Wingstop
LOCATION: Lee's Summit, MO
SUBJECT: Steel Joist Analysis for Mechanical Units
PREPARED BY: K. Woodard File:

PROJECT NUMBER: WIL-MO-04-21
DATE: 6/10/2021

Steel Joist Analysis of Irregular Loads J-5:

Joist = 20K4
Length = 34 ft
Trib. Width = 5 ft
E = 29000 ksi
Live Δ_a = L/240

Strength Analysis Results:
Bending ²assed
Max DCR = **0.86**
Shear Failed from 0to16, 23to40
Max DCR = **1.02**

Allowable Loads:
Total $Load_a$ = 212 plf (for stress)
Live $Load_a$ = 212 plf (for deflection)

DCR Limits without reinforcement:
Bending: 1.00
Shear: 1.00

Full Uniform Loads:
Dead Load = 15 psf
Live Load = 20 psf

Point Loads:
Hood = 509 lbs
Live $Load_1$ = 0 lbs
$Location_1$ = 2.33 ft
RTU-1 = 148.8 lbs
Live $Load_2$ = 0 lbs
$Location_2$ = 5.25 ft
Dead Load = 0 lbs
Live $Load_3$ = 0 lbs
$Location_3$ = 0 ft
Dead Load = 0 lbs
Live $Load_4$ = 0 lbs
$Location_4$ = 0 ft
Dead Load = 0 lbs
Live $Load_5$ = 0 lbs
$Location_5$ = 0 ft
Dead Load = 0 lbs
Live $Load_6$ = 0 lbs
$Location_6$ = 0 ft
Dead Load = 0 lbs
Live $Load_7$ = 0 lbs
$Location_7$ = 0 ft
Dead Load = 0 lbs
Live $Load_8$ = 0 lbs
$Location_8$ = 0 ft

Partial Uniform Loads:
Dead $Load_1$= 0 psf
Live $Load_1$ = 0 psf
$Start_1$ = 0 ft
End_1 = 0 ft
Dead $Load_2$= 0 psf
Live $Load_2$ = 0 psf
$Start_2$ = 0 ft
End_2 = 0 ft

Triangular Snow Drift Loads:
+Drift $Load_1$= 0 psf
$Peak_1$ = 0 ft
0psf End_1 = 0 ft
+Drift $Load_2$= 0 psf
$Peak_2$ = 0 ft
0psf End_2 = 0 ft

Is reinforcing reasonable? No
Shear capacity at 1.5 x Depth = 3021

Note: Bar joists are typically constructed symmetrically with equal sized web members along the lengh. As a result the allowable capacity is constant, except at the ends where it decreases until the web is engaged. The accompanying shear diagram is based on the code minimums and does not reflect actual construction and design practices. For this instance, minimum allowable capacity decreases to about 3021 lbs before leveling off. Reinforcement is only required at locations where the applied shear is greater than these allowables.

Joist Reinforcement Design:
T&B Chords: 0.500 Dia. Rods
Fy = 36 ksi
K = 0.65
L = 12 in
DCR = **0.00**
Webs: L1X1X1/8
Fy = 36 ksi
K = 1
DCR = **0.05**

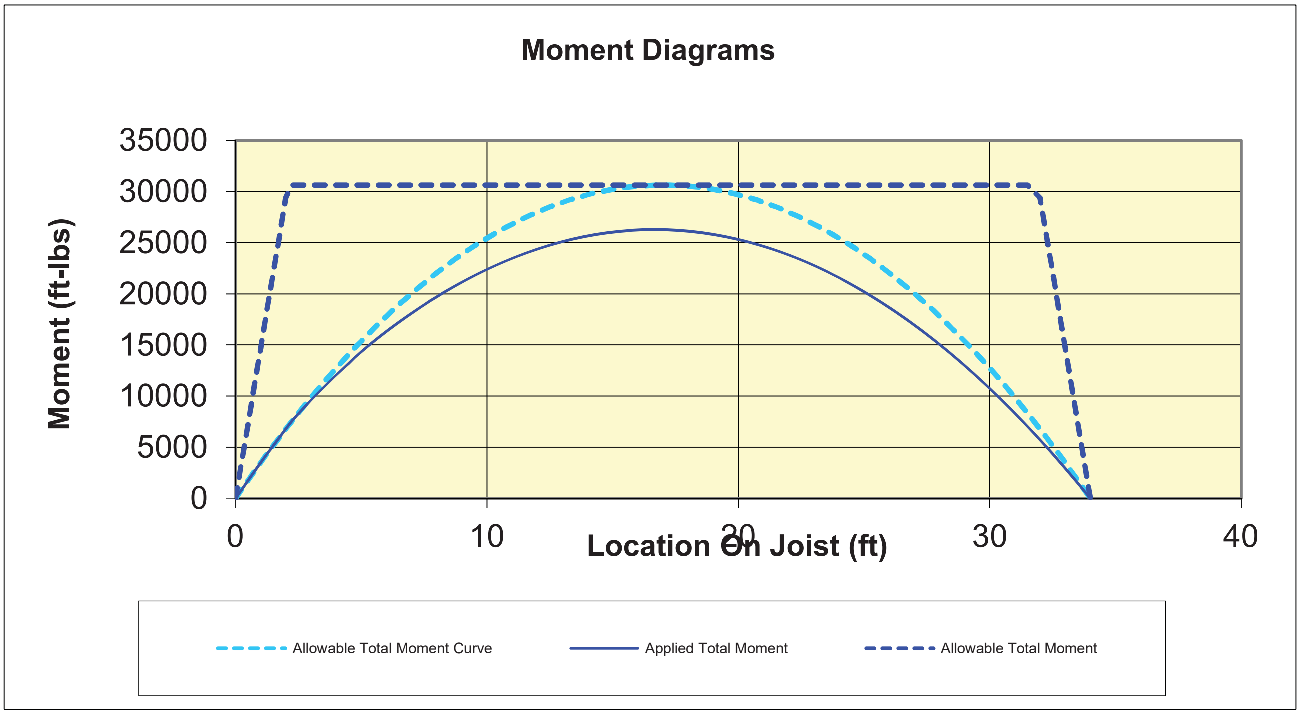
Chord Reinforcement:
Not Required

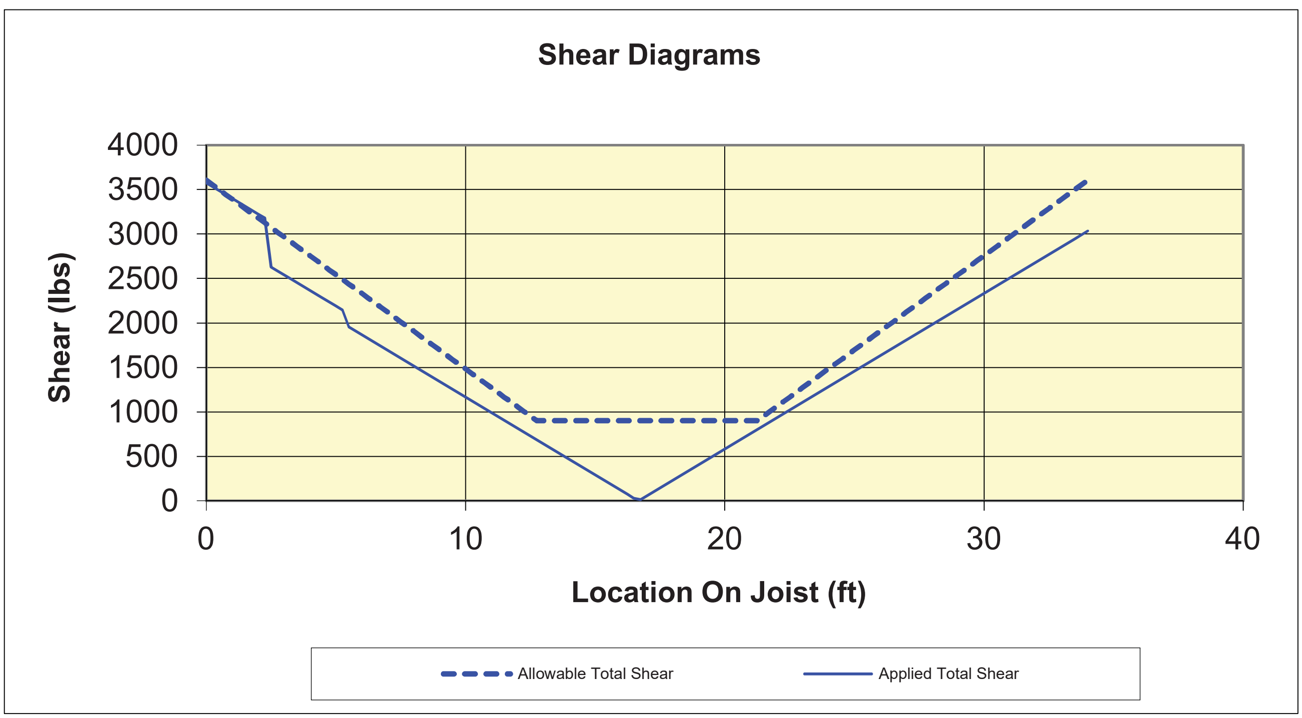
Web Reinforcement:
(1) - L1X1X1/8 at Each Member
From 1 ft to 2.25 ft

Chord Reinforcement Welds:
Not Required

Web Reinforcement:
0.125" x 1" Total Weld at Each End

Steel Joist Analysis of Irregular Loads J-5:





CASE Engineering Inc.

PROJECT NAME:	Wingstop	PROJ # WIL-MO-04-21	
LOCATION:	Lees Summit, MO		
SUBJECT:	Unit Overturning Check RTU-2	Updated	8/8/2019
PREPARED BY:	K. Woodard	DATE:	6/10/2021

Unit Overturning per ASCE 7-16 Wind Design

Input / Output

Risk Category		II	(Table 1.5-1)
Exposure Category		B	(Section 26.7.3)
Height Above Ground Level, z (ft)		17	
Velocity Pressure Exposure Coefficient K_z		0.6125	(Table 26.10-1)
Topographic Factor, K_{zt}		1.0	(Section 26.8-2 and Figure 26.8-1)
Wind Directionality Factor, K_d		0.85	(Section 26.6 and Table 26.6-1)
Ground Elevation Factor, K_e		1	(Section 26.9 and Table 26.9-1)
Ultimate Wind Speed 3-s Gust, V (mph)		109	(Figures 26.5-1 and 26.5-2)
Velocity Pressure, q_z (psf)	$q_z = .00256 K_z K_{zt} K_d K_e V^2$	15.8	(Equation 26.10-1)

Equipment Parameters	
Depth, D (in) (Least horizontal dimension at the elevation considered)	44.3
Width, W (in)	70.0
Height, H (in) (add curb height to this, curb is typically 14" MIN)	46.9
Weight, W_p (lb)	785

Gust Factor, GC_r		1.9	(Section 29.4.1)
Projected Area Normal to the Wind, A_f (Ft^2)	$A_f = \frac{W \times H}{144}$	22.8	
Design Wind Force, F_h (lb), (ASD)	$F_h = q_z(GC_r)A_f$	411	(Equation 29.4-1)

Overturning Moment, M_O (lb-in)	$M_O = F_h x .67H$	12919
Restoring Moment, M_R (lb-in)	$M_R = .6W_p\, x \frac{D}{2}$	10421

Resulant Force, (lb)	$= \frac{M_O - M_R}{D}$	56
Uplift?	$Resultant\ Force < 0 \rightarrow NO$ $Resultant\ Force > 0 \rightarrow YES$	YES

Curb to Roof Fastener Strength			
3/8" Ø Steel Bolt			
Tensile Strength, lbs	$= \frac{F_{nt} \times A_b}{\Omega} \times 1000$	2485	Ω = 2.0
(2) Bolts per side O.K.?		BOLTS O.K.	Fnt (ksi) = 45.0

$\frac{Resultant\ Force}{2} < Tensile\ Strength \rightarrow O.K.$ $\frac{Resultant\ Force}{2} > Tensile\ Strength \rightarrow N.G.$

Shear Strength, lbs	$= \frac{F_{nv} \times A_b}{\Omega} \times 1000$	1491	Fnv (ksi) = 27.0
(2) Bolts per side O.K.?		BOLTS O.K.	Ab (in^2) = 0.11

$\frac{Design\ Wind\ Force}{4} < Shear\ Strength \rightarrow O.K.$ $\frac{Design\ Wind\ Force}{4} > Shear\ Strength \rightarrow N.G.$

#10 Wood Screws	
Allowable Tensile Strength, lbs	216
(4) Screws per side O.K.?	**SCREWS O.K.**

$$\frac{Resultant\ Force}{4} < Allowable\ Tensile\ Strength \rightarrow O.K. \qquad \frac{Resultant\ Force}{4} > Allowable\ Tensile\ Strength \rightarrow N.G.$$

Allowable Shear Strength, lbs	144
(4) Screws per side O.K.?	**SCREWS O.K.**

$$\frac{Design\ Wind\ Force}{8} < Allowable\ Shear\ Strength \rightarrow O.K. \qquad \frac{Design\ Wind\ Force}{8} > Allowable\ Shear\ Strength \rightarrow N.G.$$

Curb Screw Strength	
#10 Self-Tapping Screws into 18ga Curb	
Allowable Tensile Strength (per SSMA), lbs	109
(4) Screws per side O.K.?	**SCREWS O.K.**

$$\frac{Resultant\ Force}{4} < Allowable\ Tensile\ Strength \rightarrow O.K. \qquad \frac{Resultant\ Force}{4} > Allowable\ Tensile\ Strength \rightarrow N.G.$$

Allowable Shear Strength (per SSMA), lbs	263
(4) Screws per side O.K.?	**SCREWS O.K.**

$$\frac{Design\ Wind\ Force}{8} < Allowable\ Shear\ Strength \rightarrow O.K. \qquad \frac{Design\ Wind\ Force}{8} > Allowable\ Shear\ Strength \rightarrow N.G.$$

CASE Engineering Inc.

PROJECT NAME:	Wingstop	PROJ # WIL-MO-04-21		
LOCATION:	Lees Summit, MO			
SUBJECT:	Unit Overturning Check MAU		Updated	8/8/2019
PREPARED BY:	K. Woodard		DATE:	6/10/2021

Unit Overturning per ASCE 7-16 Wind Design

Input / Output

Risk Category		II	(Table 1.5-1)
Exposure Category		B	(Section 26.7.3)
Height Above Ground Level, z (ft)		17	
Velocity Pressure Exposure Coefficient K_z		0.6125	(Table 26.10-1)
Topographic Factor, K_{zt}		1.0	(Section 26.8-2 and Figure 26.8-1)
Wind Directionality Factor, K_d		0.85	(Section 26.6 and Table 26.6-1)
Ground Elevation Factor, K_e		1	(Section 26.9 and Table 26.9-1)
Ultimate Wind Speed 3-s Gust, V (mph)		110	(Figures 26.5-1 and 26.5-2)
Velocity Pressure, q_z (psf)	$q_z = .00256 K_z K_{zt} K_d K_e V^2$	16.1	(Equation 26.10-1)

Equipment Parameters	
Depth, D (in) (Least horizontal dimension at the elevation considered)	24.0
Width, W (in)	100.0
Height, H (in) (add curb height to this, curb is typically 14" MIN)	29.0
Weight, W_p (lb)	525

Gust Factor, GC_r		1.9	(Section 29.4.1)
Projected Area Normal to the Wind, A_f (Ft^2)	$A_f = \frac{W \times H}{144}$	20.1	
Design Wind Force, F_h (lb), (ASD)	$F_h = q_z(GC_r)A_f$	370	(Equation 29.4-1)

Overturning Moment, M_O (lb-in)	$M_O = F_h x.67H$	7194
Restoring Moment, M_R (lb-in)	$M_R = .6W_p\, x\frac{D}{2}$	3780

Resulant Force, (lb)	$= \frac{M_O - M_R}{D}$	142
Uplift?	$Resultant\ Force < 0 \rightarrow NO$ $Resultant\ Force > 0 \rightarrow YES$	YES

Curb to Roof Fastener Strength				
3/8" Ø Steel Bolt				
Tensile Strength, lbs	$= \frac{F_{nt} \times A_b}{\Omega} \times 1000$	2485	Ω =	2.0
(2) Bolts per side O.K.?		BOLTS O.K.	Fnt (ksi) =	45.0
Shear Strength, lbs	$= \frac{F_{nv} \times A_b}{\Omega} \times 1000$	1491	Fnv (ksi) =	27.0
(2) Bolts per side O.K.?		BOLTS O.K.	Ab (in^2) =	0.11

$\frac{Resultant\ Force}{2} < Tensile\ Strength \rightarrow O.K.$ $\qquad \frac{Resultant\ Force}{2} > Tensile\ Strength \rightarrow N.G.$

$\frac{Design\ Wind\ Force}{4} < Shear\ Strength \rightarrow O.K.$ $\qquad \frac{Design\ Wind\ Force}{4} > Shear\ Strength \rightarrow N.G.$

#10 Wood Screws	
Allowable Tensile Strength, lbs	216
(4) Screws per side O.K.?	**SCREWS O.K.**

$$\frac{Resultant\ Force}{4} < Allowable\ Tensile\ Strength \rightarrow O.K. \qquad \frac{Resultant\ Force}{4} > Allowable\ Tensile\ Strength \rightarrow N.G.$$

Allowable Shear Strength, lbs	144
(4) Screws per side O.K.?	**SCREWS O.K.**

$$\frac{Design\ Wind\ Force}{8} < Allowable\ Shear\ Strength \rightarrow O.K. \qquad \frac{Design\ Wind\ Force}{8} > Allowable\ Shear\ Strength \rightarrow N.G.$$

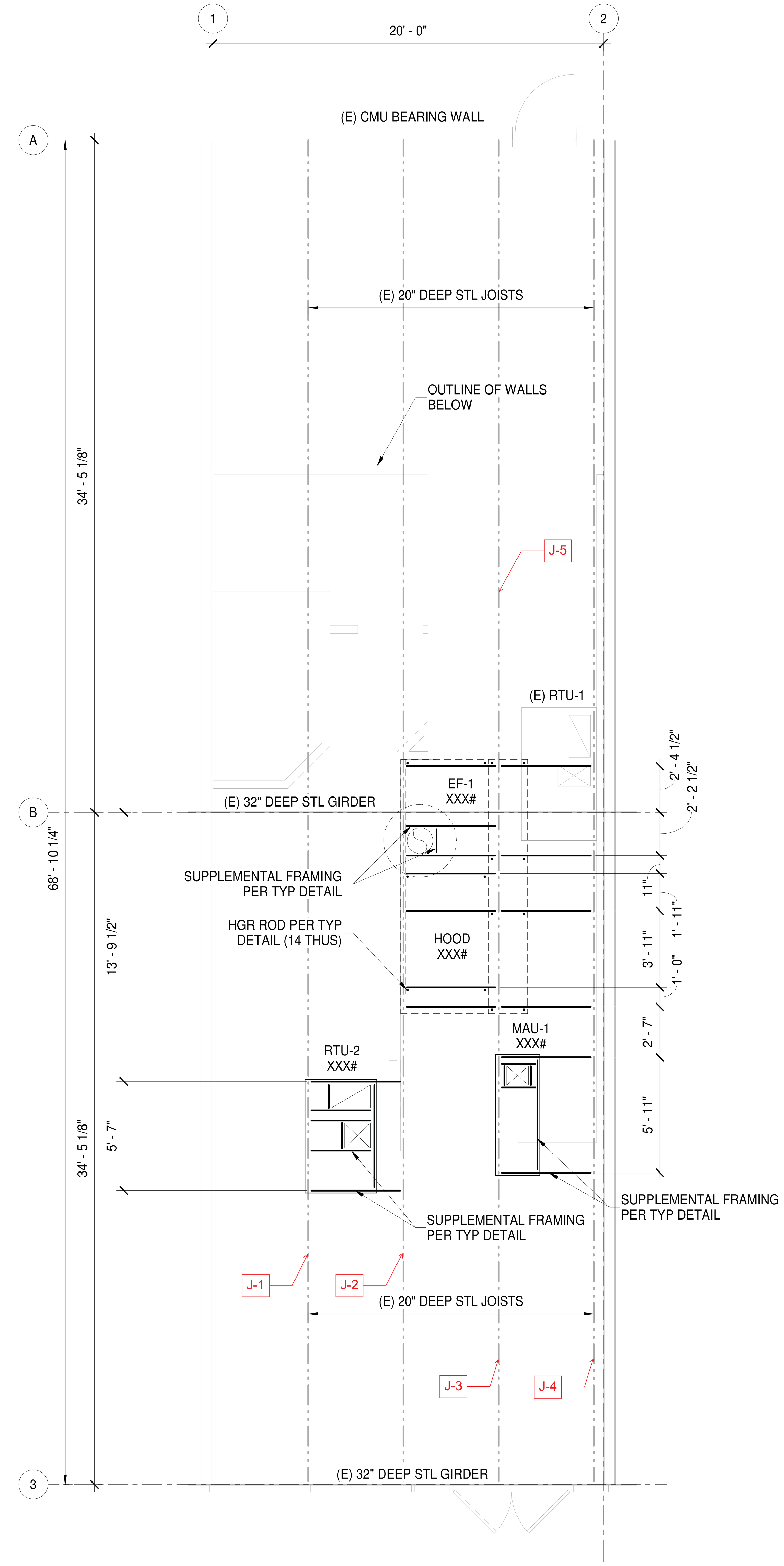
Curb Screw Strength	
#10 Self-Tapping Screws into 18ga Curb	
Allowable Tensile Strength (per SSMA), lbs	109
(4) Screws per side O.K.?	**SCREWS O.K.**

$$\frac{Resultant\ Force}{4} < Allowable\ Tensile\ Strength \rightarrow O.K. \qquad \frac{Resultant\ Force}{4} > Allowable\ Tensile\ Strength \rightarrow N.G.$$

Allowable Shear Strength (per SSMA), lbs	263
(4) Screws per side O.K.?	**SCREWS O.K.**

$$\frac{Design\ Wind\ Force}{8} < Allowable\ Shear\ Strength \rightarrow O.K. \qquad \frac{Design\ Wind\ Force}{8} > Allowable\ Shear\ Strength \rightarrow N.G.$$

Joist Reference for Calcs



PARTIAL EXISTING ROOF FRAMING PLAN

PLAN NOTES: SCALE: 1/4" = 1'-0"

1. SEE SHEET S1 FOR GENERAL NOTES AND TYPICAL DETAILS.
2. IT SHALL BE THE CONTRACTOR'S RESPONSIBILITY TO VERIFY ALL DIMENSIONS AND ELEVATIONS PRIOR TO BEGINNING CONSTRUCTION.
3. REFERENCE MECHANICAL DRAWINGS FOR EXACT WEIGHTS AND LOCATIONS OF MECHANICAL EQUIPMENT.
4. SEE ARCHITECTURAL DRAWINGS FOR DIMENSIONS, SECTIONS, AND ELEVATIONS NOT SHOWN HEREON.
5. ALL NEW AND EXISTING MECHANICAL EQUIPMENT MOUNTED TO OR HUNG FROM EXISTING **ROOF** FRAMING STRUCTURE THAT HAS BEEN ACCOUNTED FOR IN STRUCTURAL CAPACITY ANALYSIS IS SHOWN ON FRAMING PLAN. GENERAL CONTRACTOR SHALL NOTIFY ARCHITECT AND ENGINEER OF RECORD IMMEDIATELY IF EQUIPMENT EXISTS THAT IS NOT SHOWN ON PLAN.

CONSULTANT:

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St. Louis, MO 63026
CERTIFICATE OF AUTHORITY NO. 001498
T 636.349.1600
F 636.349.1730

CLIENT:

5501 LBJ FREEWAY, 5TH FLOOR
DALLAS, TX 75240
TELEPHONE: (972) 686-6500
FAX: (972) 686-6502

PROJECT INFORMATION:

WING•STOP
STORE NUMBER: GL#Y1750
HALLBUSTER SHOPS
1041 N.E. SAM WALTON LANE
LEE'S SUMMIT, MO 64086

SEAL:

DATE

PROJECT INFO:	WIL-MO-04-21
DRAWN BY:	JS
CHECKED BY:	KW

ISSUE:	DATE:
FOR PERMIT & BID	06-10-21

REVISION:	DATE:

PROJECT LOCATION:
LEE'S SUMMIT, MO

SHEET NUMBER / TITLE:
S2
ROOF FRAMING PLAN

STANDARD LOAD TABLE

Based on a Maximum Allowable Tensile Stress of 30 ksi

OPEN WEB STEEL JOISTS, K- SERIES

Adopted by the Steel Joist Institute November 4, 1985; Revised to May 2, 1994 - Effective September 1, 1994

The black figures in the following table give the TOTAL safe uniformly distributed load-carrying capacities, in pounds per linear foot, of K-Series Steel Joists. The weight of DEAD loads, including the joists, must be deducted to determine the LIVE load-carrying capacities of the joists. The load table may be used for parallel chord joists installed to a maximum slope of ½ inch per foot.

The figures shown in RED in this load table are the LIVE loads per linear foot of joist which will produce an approximate deflection of 1/360 of the span. LIVE loads which will produce a deflection of 1/240 of the span may be obtained by multiplying the figures in RED by 1.5. In no case shall the TOTAL load capacity of the joists be exceeded.

The approximate joist weights per linear foot shown in these tables do not include accessories.

The approximate moment of inertia of the joist, in inches^4 is; $I_j = 26.767(W_{LL})(L^3)(10^{-6})$, where W_{LL}= RED figure in the Load Table and L = (Span - .33) in feet.

For the proper handling of concentrated and/or varying loads, see Section 5.5 in the Recommended Code of Standard Practice.

Where the joist span exceeds the unshaded area of the table, the row of bridging nearest the mid span shall be diagonal bridging with bolted connections at chords and midspan.

Joist Designation	8K1	10K1	12K1	12K3	12K5	14K1	14K3	14K4	14K6	16K2	16K3	16K4	16K5	16K6	16K7	16K9
Depth (In.)	8	10	12	12	12	14	14	14	14	16	16	16	16	16	16	16
Approx. Wt. (lbs./ft.)	5.1	5.0	5.0	5.7	7.1	5.2	6.0	6.7	7.7	5.5	6.3	7.0	7.5	8.1	8.6	10.0
Span (ft.) ↓																
8	550 550															
9	550 550															
10	550 480	550 550														
11	532 377	550 542														
12	444 288	550 455	550 550	550 550	550 550											
13	377 225	479 363	550 510	550 510	550 510											
14	324 179	412 289	500 425	550 463	550 463	550 550	550 550	550 550	550 550							
15	281 145	358 234	434 344	543 428	550 434	511 475	550 507	550 507	550 507							
16	246 119	313 192	380 282	476 351	550 396	448 390	550 467	550 467	550 467	550 550	550 550	550 550	550 550	550 550	550 550	550 550
17		277 159	336 234	420 291	550 366	395 324	495 404	550 443	550 443	512 488	550 526	550 526	550 526	550 526	550 526	550 526
18		246 134	299 197	374 245	507 317	352 272	441 339	530 397	550 408	456 409	508 456	550 490	550 490	550 490	550 490	550 490
19		221 113	268 167	335 207	454 269	315 230	395 287	475 336	550 383	408 347	455 386	547 452	550 455	550 455	550 455	550 455
20		199 97	241 142	302 177	409 230	284 197	356 246	428 287	525 347	368 297	410 330	493 386	550 426	550 426	550 426	550 426
21			218 123	273 153	370 198	257 170	322 212	388 248	475 299	333 255	371 285	447 333	503 373	548 405	550 406	550 406
22			199 106	249 132	337 172	234 147	293 184	353 215	432 259	303 222	337 247	406 289	458 323	498 351	550 385	550 385
23			181 93	227 116	308 150	214 128	268 160	322 188	395 226	277 194	308 216	371 252	418 282	455 307	507 339	550 363
24			166 81	208 101	282 132	196 113	245 141	295 165	362 199	254 170	283 189	340 221	384 248	418 269	465 298	550 346
25						180 100	226 124	272 145	334 175	234 150	260 167	313 195	353 219	384 238	428 263	514 311
26						166 88	209 110	251 129	308 156	216 133	240 148	289 173	326 194	355 211	395 233	474 276
27						154 79	193 98	233 115	285 139	200 119	223 132	268 155	302 173	329 188	366 208	439 246
28						143 70	180 88	216 103	265 124	186 106	207 118	249 138	281 155	306 168	340 186	408 220
29										173 95	193 106	232 124	261 139	285 151	317 167	380 198
30										161 86	180 96	216 112	244 126	266 137	296 151	355 178
31										151 78	168 87	203 101	228 114	249 124	277 137	332 161
32										142 71	158 79	190 92	214 103	233 112	259 124	311 147

STANDARD LOAD TABLE/OPEN WEB STEEL JOISTS, K-SERIES
Based on a Maximum Allowable Tensile Stress of 30 ksi

Joist Designation	18K3	18K4	18K5	18K6	18K7	18K9	18K10	20K3	20K4	20K5	20K6	20K7	20K9	20K10	22K4	22K5	22K6	22K7	22K9	22K10	22K11
Depth (In.)	18	18	18	18	18	18	18	20	20	20	20	20	20	20	22	22	22	22	22	22	22
Approx. Wt. (lbs./ft.)	6.6	7.2	7.7	8.5	9.0	10.2	11.7	6.7	7.6	8.2	8.9	9.3	10.8	12.2	8.0	8.8	9.2	9.7	11.3	12.6	13.8
Span (ft.) ↓																					
18	550 550	550 550	550 550	550 550	550 550	550 550	550 550														
19	514 494	550 523	550 523	550 523	550 523	550 523	550 523														
20	463 423	550 490	550 490	550 490	550 490	550 490	550 490	517 517	550 550	550 550	550 550	550 550	550 550	550 550							
21	420 364	506 426	550 460	550 460	550 460	550 460	550 460	468 453	550 520	550 520	550 520	550 520	550 520	550 520							
22	382 316	460 370	518 414	550 438	550 438	550 438	550 438	426 393	514 461	550 490	550 490	550 490	550 490	550 490	550 548	550 548	550 548	550 548	550 548	550 548	550 548
23	349 276	420 323	473 362	516 393	550 418	550 418	550 418	389 344	469 402	529 451	550 468	550 468	550 468	550 468	518 491	550 518	550 518	550 518	550 518	550 518	550 518
24	320 242	385 284	434 318	473 345	526 382	550 396	550 396	357 302	430 353	485 396	528 430	550 448	550 448	550 448	475 431	536 483	550 495	550 495	550 495	550 495	550 495
25	294 214	355 250	400 281	435 305	485 337	550 377	550 377	329 266	396 312	446 350	486 380	541 421	550 426	550 426	438 381	493 427	537 464	550 474	550 474	550 474	550 474
26	272 190	328 222	369 249	402 271	448 299	538 354	550 361	304 236	366 277	412 310	449 337	500 373	550 405	550 405	404 338	455 379	496 411	550 454	550 454	550 454	550 454
27	252 169	303 198	342 222	372 241	415 267	498 315	550 347	281 211	339 247	382 277	416 301	463 333	550 389	550 389	374 301	422 337	459 367	512 406	550 432	550 432	550 432
28	234 151	282 177	318 199	346 216	385 239	463 282	548 331	261 189	315 221	355 248	386 269	430 298	517 353	550 375	348 270	392 302	427 328	475 364	550 413	550 413	550 413
29	218 136	263 159	296 179	322 194	359 215	431 254	511 298	243 170	293 199	330 223	360 242	401 268	482 317	550 359	324 242	365 272	398 295	443 327	532 387	550 399	550 399
30	203 123	245 144	276 161	301 175	335 194	402 229	477 269	227 153	274 179	308 201	336 218	374 242	450 286	533 336	302 219	341 245	371 266	413 295	497 349	550 385	550 385
31	190 111	229 130	258 146	281 158	313 175	376 207	446 243	212 138	256 162	289 182	314 198	350 219	421 259	499 304	283 198	319 222	347 241	387 267	465 316	550 369	550 369
32	178 101	215 118	242 132	264 144	294 159	353 188	418 221	199 126	240 147	271 165	295 179	328 199	395 235	468 276	265 180	299 201	326 219	363 242	436 287	517 337	549 355
33	168 92	202 108	228 121	248 131	276 145	332 171	393 201	187 114	226 134	254 150	277 163	309 181	371 214	440 251	249 164	281 183	306 199	341 221	410 261	486 307	532 334
34	158 84	190 98	214 110	233 120	260 132	312 156	370 184	176 105	212 122	239 137	261 149	290 165	349 195	414 229	235 149	265 167	288 182	321 202	386 239	458 280	516 314
35	149 77	179 90	202 101	220 110	245 121	294 143	349 168	166 96	200 112	226 126	246 137	274 151	329 179	390 210	221 137	249 153	272 167	303 185	364 219	432 257	494 292
36	141 70	169 82	191 92	208 101	232 111	278 132	330 154	157 88	189 103	213 115	232 125	259 139	311 164	369 193	209 126	236 141	257 153	286 169	344 201	408 236	467 269
37								148 81	179 95	202 106	220 115	245 128	294 151	349 178	198 116	223 130	243 141	271 156	325 185	386 217	442 247
38								141 74	170 87	191 98	208 106	232 118	279 139	331 164	187 107	211 119	230 130	256 144	308 170	366 200	419 228
39								133 69	161 81	181 90	198 98	220 109	265 129	314 151	178 98	200 110	218 120	243 133	292 157	347 185	397 211
40								127 64	153 75	172 84	188 91	209 101	251 119	298 140	169 91	190 102	207 111	231 123	278 146	330 171	377 195
41															161 85	181 95	197 103	220 114	264 135	314 159	359 181
42															153 79	173 88	188 96	209 106	252 126	299 148	342 168
43															146 73	165 82	179 89	200 99	240 117	285 138	326 157
44															139 68	157 76	171 83	191 92	229 109	272 128	311 146

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