STRUCTURAL ANALYSIS OF ROOF SUPPORT STRUCTURE TO DETERMINE SAFE LOAD CARRYING CAPACITY

FOR:

TOWN OF EXETER
DEPARTMENT OF PUBLIC WORKS
13 NEWFIELDS ROAD
EXETER, NEW HAMPSHIRE

JANUARY 2016

The H.L. Turner Group Inc.

ARCHITECTS ▪ ENGINEERS ▪ BUILDING SCIENTISTS

Town of Exeter

Department of Public Works

13 Newfields Road

Exeter, New Hampshire

Structural Analysis of Roof Support Structure to Determine Safe Load Carrying Capacity

Main Department of Public Works Garage and the Water Department Garage

January 2016

Introduction

On December 22, 2015, structural engineers, Paul M. Becht, P.E and Jonathan George, representatives of The H.L. Turner Group Inc. (TTG), visited the Exeter Department of Public Works located at 13 Newfields Road, Exeter, New Hampshire. We met with Kevin Smart, Maintenance Superintendent, for the DPW. The purpose of this site visit was to review the existing roof structure over the Main DPW Garage, as well as the building used by the Town's Water Department.

The Main DPW Garage

The Main DPW Garage was constructed around 1969 and is a typical, pre-engineered, metal building supported by a perimeter wall that extends 4 feet above grade. The interior of the building has a concrete slab-on-grade. The building is approximately 15,000 square feet measuring 250 feet by 60 feet overall. The original section of the building is about 130 feet long and consists of an area with ground level offices, a break room, locker room and rest rooms, a mezzanine with additional office space and storage, and four truck bay doors to the west. The building was expanded in the mid-1970's with the addition of about 120 feet with five more truck bays on the east side. The two bays directly east of the central office area are typically used for maintenance of Town vehicles and the last three bays to the east are used for a parts room, vehicle and equipment storage, and a mezzanine with additional storage for equipment. Both sections of the building are structurally similar and are comprised of steel bents spaced at 20 feet on center, with 8-inch deep roof purlins spaced approximately 4 feet on center spanning between the bents. The building is clad with metal roofing panels and metal siding. There is minimal insulation in the walls and roof.

The metal roof panels are attached with exposed fasteners and it appears that the roof panels were coated with an elastomeric, membrane type coating designed to lengthen the life expectancy of an existing roof. During our evaluation and measurements we observed signs of leakage, which explains the installation of the coating system. The roof edge detail on the Main DPW Garage appears to be incomplete. The closure piece is set about 12 inches in from the edge of the roof panel such that it is slightly inboard of the plane of the wall. It appears that water can penetrate the area between the top of the wall and the bottom of the roofing and easily find its way down the inside face of the wall. The closure piece appears to have a significant number of openings. The roof edge detail combined with the lack of insulation and venting makes this roof very susceptible to the formation of ice dams.

<u>The Water Department Garage</u>

The Water Department Garage was constructed during the late 1980's and is a typical, pre-engineered, metal building built on a perimeter frost wall with a concrete slab-on-grade in the interior of the building. The building is approximately 6,000 square feet, measuring 100 feet long by 60 feet wide. There are five bays, each with an overhead door. The bay on the east side of the building serves as a drive-thru wash bay and therefore, has an overhead door at each side of the building. The building is comprised of steel bents spaced at 20 feet on center with 9-inch roof purlins spaced approximately 4 feet on center spanning between the bents. The building is clad with metal roofing panels and metal siding. The low slope roof is typical of a pre-engineered metal building. The lack of insulation and venting in these types of buildings makes them very susceptible to the formation of ice dams.

<u>Site Investigation</u>

While on-site we took measurements of the columns, framing members, girts, girt and bent spans, and spacing at various locations throughout the garage. This information was then used to perform a structural analysis of the existing roof framing system and determine the safe live load carrying capacity of the specific roof areas. The purpose of this study is to evaluate the general condition of the existing structure and to determine if the structure meets the current International Building Code (IBC) requirements for snow load. This study will also assist the Town in adopting a snow monitoring and removal plan, should the code required loads be exceeded during a particularly snowy winter.

Knowing the live load capacity of the roof structure is critical in terms of determining the allowable snow build-up that can occur on the roof and when snow removal may be required. If new mechanical units are being planned for the roof, knowing the live load capacity of the roof can provide preliminary input on whether structural upgrades will be needed. However, any time any equipment is placed on the roof of a building, a detailed engineering study is always recommended.

Building Codes and Snow Loads

Building codes have changed over the years. The current building code for New Hampshire is the 2009 International Building Code (IBC). Snow accumulation on the roofs of buildings have been studied and updated, and have been changed to better reflect actual snow that falls in specific towns. Furthermore, the current code accounts for the occurrence of blowing and drifting snow. For ground snow loads, on a town-by-town basis within the State of New Hampshire, the IBC references ASCE 07 for certain structural loading. For New Hampshire, many areas of the state are recognized as special study zones and defers to the authority having jurisdiction. Local building code officials in turn refer to a publication entitled Technical Report ERDC/CRREL TR-02-6, "Ground Snow Loads for New Hampshire" dated February 2002, published by the Cold Regions Research and Engineering Laboratory in conjunction with the US Army Corps of Engineers. For the Town of Exeter, the ground snow load is listed as 50 pounds per square foot (psf). This translates to a low slope roof load of 35 psf, not including snow drift load. This value was used throughout this report and compared against the calculated capacity for each section of the roof. Whenever there is a height change of 4 feet or more, wind can cause snow to drift against the vertical wall between the low roof and the high roof. In this case, there are no changes in height that would require consideration of drift loads.

A word about snow loads. A snow load of 35 pounds per square foot is equivalent to about 3 feet of moderately heavy snow. Moderately heavy snow is considered to be somewhere between "light/dry" snow and "heavy/wet" snow. The moisture content of moderately heavy snow is about 20%, with a unit weight of 12 pounds per square foot. Light/dry snow, which can have as little 5% water content, would have to accumulate to over 11 feet to reach the code prescribed value, or at the other end of the spectrum, it would take only 20 inches of heavy wet snow, which can have a water content of 33%, to equal the 35 pounds per square foot. So in deciding if snow removal from a roof is required, it is not enough to measure the depth. The weight of the snow in terms of its pounds per square foot must also be measured. For reference, a snow density chart has been included in the appendices.

Assumptions used for the Analysis

In calculating the allowable load carrying capacity of the roof, we included a dead load for the existing building materials of approximately six pounds per square foot. This includes the weight of the existing roof panels, insulation, steel framing, miscellaneous piping, and electrical conduits. The dead load was then separated from the total load carrying capacity of the roof to give the allowable live load (or snow load) capacities.

To ascertain the grade of steel used for each building it would require obtaining a steel coupon from one of the members, bringing it to a materials laboratory, and having it tested to determine the yield strength of the metal. For this analysis we relied on our past experience with buildings of this vintage. For structures built pre-1980, which includes the entire Main DPW Garage, we assumed a steel yield strength of 36,000 pounds per square inch (psi). For post-1980 structures, in this case the Water Department Garage, we assumed a yield strength for the steel members of 50,000 psi. This grade of steel was typically specified for building construction starting in the 1980's.

The results of our analysis for each section of the building are summarized below.

Main DPW Garage – 1969 Section

The structural analysis of the 8-inch deep girts, which span from bent-to-bent, indicates that the girts are capable of safely supporting a live load of approximately 20 pounds per square foot or about 57% of the current code mandated snow load of 35 psf.

The horizontal bents which span across the full 60-foot width of the building are 42 inches deep at centerline and taper down to 11-3/8" at the exterior walls. The top and bottom flange widths are constant at 10 inches wide. The structural analysis of this portion of the bent indicates that the member is capable of safely supporting the current code mandated snow load of 35 pounds per square foot.

The columns are 4" x 8" members and the structural analysis of the columns supporting the bents indicate that they are capable of safely supporting a live load of only 18 pounds per square foot or about 50% of the current code mandated snow load.

The horizontal portion of the bent could support up to 45 pounds per square foot, but the capacities of the girts and the columns limit the safe live load carrying capacity for the structure. The live load limit for the 1969 section of the DPW Garage roof is between 18 and 20 pounds per square foot. This is equivalent to about 18 inches of moderately heavy snow. The analysis results are included in the appendices.

Main DPW Garage – Mid-1970's Section

The structural analysis of the 8-inch deep girts, which span from bent-to-bent, indicates that the girts are capable of safely supporting a live load of approximately 20 pounds per square foot or about 57% of the current code mandated snow load of 35 psf.

The horizontal bents which span across the full 60-foot width of the building are 44 inches deep at centerline and taper down to 15-1/8" at the exterior walls. The top and bottom flange widths are constant at 8 inches wide. The structural analysis of this portion of the bent indicates that the member is capable of safely supporting the current code mandated snow load of 35 pounds per square foot.

The columns are 6" x 10" members and the structural analysis of the columns supporting the bents indicate that they are capable of safely supporting a live load of 25 pounds per square foot or about 70% of the current code mandated snow load.

The horizontal portion of the bent could support up to 40 pounds per square foot, but the capacities of the girts and the columns limit the safe live load carrying capacity for the structure. The live load limit for the 1975 section of the DPW Garage roof is about 20 pounds per square foot. This is equivalent to about 20 inches of moderately heavy snow. The analysis results are included in the appendices.

Water Department Garage

The structural analysis of the 9-inch deep girts, which span from bent-to-bent, indicates that the girts are capable of safely supporting a live load of approximately 30 pounds per square foot or about 85% of the current code mandated snow load of 35 psf.

The horizontal bents which span across the full 60-foot width of the building are 32 inches deep at centerline and remain constant at this depth at the exterior walls. The top and bottom flange widths are constant at 8 inches wide. The structural analysis of this portion of the bent indicates that the member is capable of safely supporting the current code mandated snow load of 35 pounds per square foot, with no excess capacity.

The columns are tapered. Starting at a depth of 32 inches at the top they taper down to a depth of 8-1/2 inches at the base. The column flanges are a constant 8 inches wide. The structural analysis of the columns supporting the bents indicates that these members are capable of safely supporting the current code mandated snow load of 35 pounds per square foot, with no excess capacity.

The safe live load carrying capacity of the Water Department Garage is dictated by the capacity of the purlins. The live load limit for the Water Department Garage is about 30 pounds per square foot. This is equivalent to about 30 inches of moderately heavy snow. The analysis results are included in the appendices.

Conclusion

This concludes our report on the structural evaluation of the DPW and Water Department Garage roofs to determine the safe load carrying capacities. Various members in both buildings fall short of meeting current code requirements for snow load. Partial structural failure may have been averted in the past for two reasons. First, the Town had a snow removal plan in place, requiring personnel to physically go up on the roof and shovel off the snow and two, the lack of insulation at the roof contributed to heat loss that resulted in less snow build-up on an otherwise cold roof.

The Water Department Garage, due to an insufficient girt capacity, falls about 15% short of meeting code. The Town could manage with this situation provided snowfalls are monitored and a snow removal plan remains in place. Besides, the Water Department Garage is in relatively good condition and still has many more years of useful life.

There is a much more pronounced gap between the load carrying capacity of the DPW Garage roof and current code requirements. The Town may consider building upgrades to achieve the desired live load capacity. These upgrades may include the installation of additional members (i.e. the installation of new girts between the existing girts) or strengthening the existing members by adding cover plates or doubler plates on existing flanges of columns to increase the cross-section and ultimately increase its load carrying capacity. If new girts are added to upgrade the roof's load carrying capacity, it would be an opportune time to replace the existing roof panels at the same time. The alternative is simply replacing the building with a new structure, designed to meet current building codes. There are other factors that enter into the decision to replace the building including: does the current building provide the needed space, does the building layout function as intended, and does it meet the needs of the Department; are the office and other spaces sufficient in meeting the needs of the staff and is the general overall condition of the building, including the electrical and mechanical systems, in good condition?

We understand that the Town does implement snow removal during times of heavy build-up on the roof. If there is not a formal plan in place, the Town should adopt a monitoring plan for checking snow loads and a subsequent plan for snow removal. The depth of snow on the roof of a building is important in assessing the loading; however, it is the total weight of the accumulated snow and ice that is the most critical element when assessing the vulnerability of the structure. As mentioned at the beginning of this report, the weight of the snow is dependent upon the water/moisture content. A wet, heavy snow can have as much as 33% water content, which translates to almost 21 pounds per square foot. Reference the chart in the appendices that gives density information and snow load based on accumulation depth.

The only practical and accurate way to determine the roof snow load is to collect actual samples of snow from the roof in question and weigh them. Other methods for keeping track of the weight of the snow on the roof are a Snow Scale for rooftop applications by Hydrological Services of America or the SnowSentry by 2KR Systems LLC of Barrington, NH. Other methods for monitoring snow load include measuring the deflection of the roof support structure at key points inside the building to determine if maximum load capacities are being approached. Once a formal plan has been generated for the DPW Garages, similar plans can be generated for all the municipal buildings.

Framing in 1969 section of DPW Garage.

Column-to-beam connection at exterior wall of DPW Garage. Note size of column.

1969 Section of DPW Garage: Roof panel support girt running over bent (note insulation).

Mid-1970’s Section of DPW Garage: Typical bent at centerline with girts and roof insulation.

Overview of roof on DPW Garage. Note exposed fasteners and metallic roof coating.

Close-up of ridge of DPW Garage.

DPW Garage roof edge detail.

Steel bent in Water Department Garage.

Water Department Garage: Steel bent with roof panel support girts and insulation.

Overview of Water Department Garage.

Density Information				
	Light/Dry Snow	Heavy/Wet Snow	Ice	Water
Snow Density (lb/cu ft)	3.12	20.81	57.25	62.43
% of water	5%	33%	92%	100%

Snow Load Based on Accumulation Depth			
Snow Depth (feet)	"Dry Snow" (lbs/sq ft)	"In between Snow" (lbs/sq ft)	"Wet Snow" (lbs/sq ft)
1	3	12	21
2	6.5	24	42
3	9.5	36	62
4	12.5	48	83
5	15.5	60	104

Collecting samples of snow/ice is the only practical and accurate way to determine the roof load. The first step is to collect a uniform vertical column of snow from the snow surface to the roof surface. This can be done by thrusting a 3-pound coffee can (6 inches in diameter) repeatedly into the snow until reaching the roof. Empty the snow into a bucket each time the coffee can is filled. After the snow is collected, it is melted and poured back into the coffee can and water depth measured in inches. This depth multiplied by 5.2 provides the snow load in pounds per square foot. For example, if your melted sample measures 4 inches deep, your roof snow load is approximately 21 lbs. per square foot (4 x 5.2 = 20.8).

The other option is to measure a 12" by 12" section from the top of the snow surface to the roof level and measure this column.

BASIS OF DESIGN/ANALYSIS

LOADING ON BENTS:

ASSUMING 6PSF UNIFORM DEAD LOAD:

$W_D = 6psf(20') = 120$ LB/FT

* SELF WIEGHT OF BENT INCLUDED IN ANALYSIS PROGRAM

GROUND SNOW FOR EXETER = 50 PSF

ROOF SNOW = 0.7(1.0)(1.0)(1.0)(50 psf)
= 35psf

$W_S = 35$ psf $(20') = 700$ LB/FT

$F_y = 36{,}000$ psi STEEL (FOR PRE 1980)
$\gamma_s = 490$ pcf $\Rightarrow$ MAIN DPW GARAGE

$F_y = 50{,}000$ psi STEEL (FOR POST 1980)
$\gamma_s = 490$ pcf $\Rightarrow$ WATER DEPT. GARAGE

Job # 4358

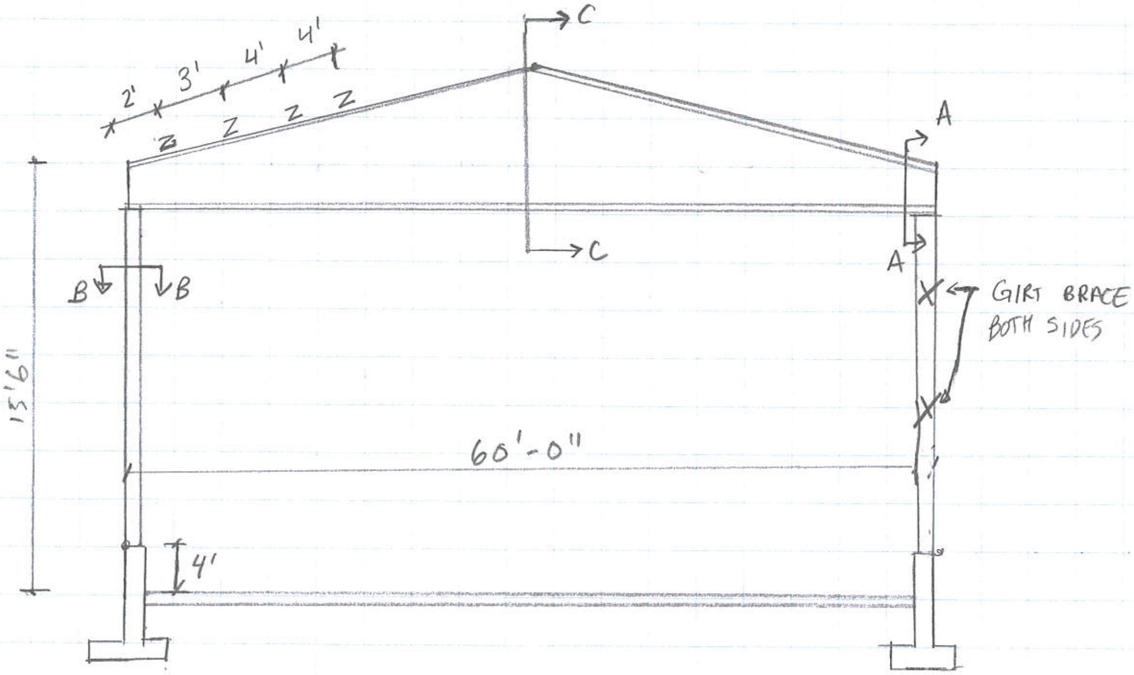
Subject TOWN OF EXETER - DPW ROOF EVALUATION

DEPARTMENT OF PUBLIC WORKS GARAGE

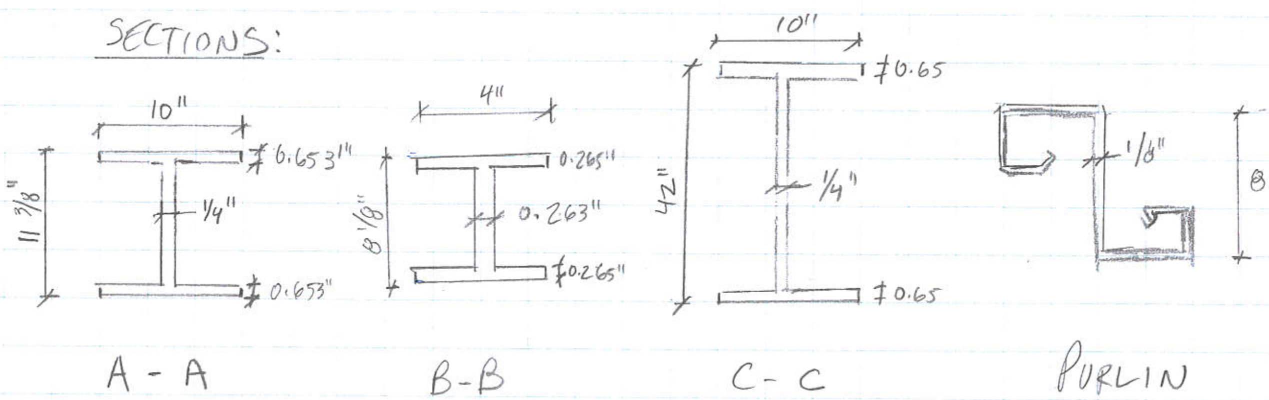
BAY 1: ORIGINAL BLDG (CONSTRUCTED ≈ LATE 60's - EARLY 1970's)

BUILDING SECTION:

BAY TO BAY SPACING = 20' C.C.



SECTIONS:



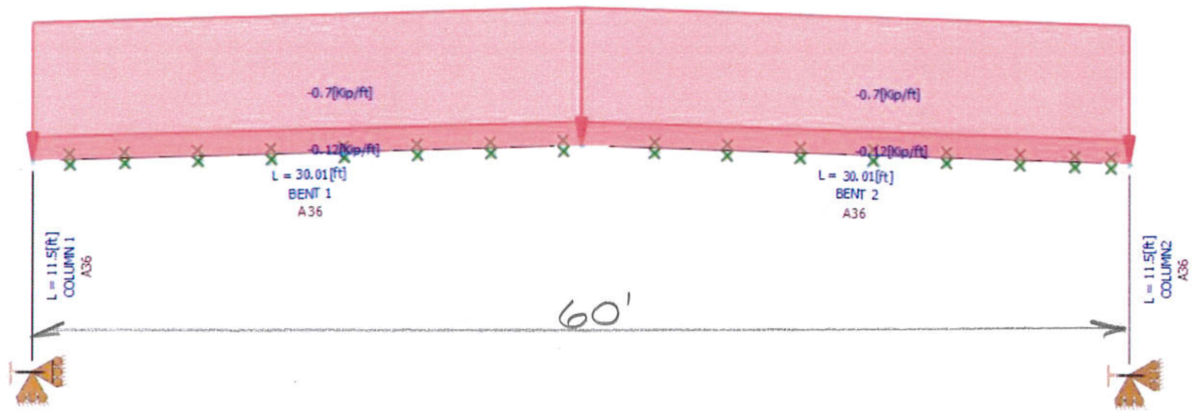
*COLUMNS DO NOT MEET CURRENT CODE.
STRUCTURE RATED FOR 10 PSF OF SNOW

DPW GARAGE

BAY 1: STRUCTURAL ANALYSIS RESULTS

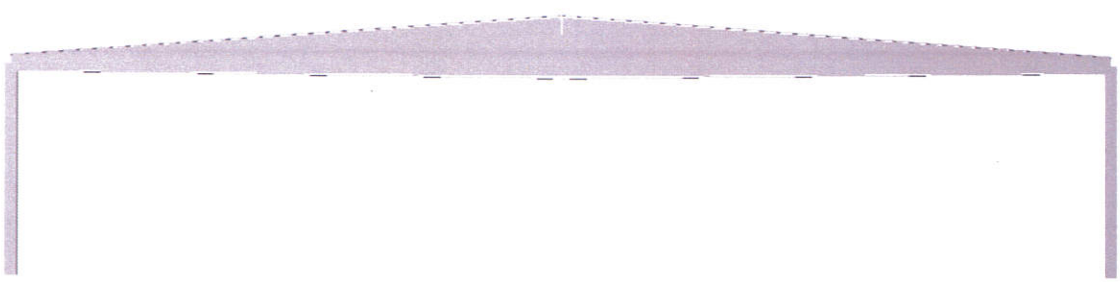
(CONSTRUCTED LATE 1960'S-EARLY 1970'S)

LOADING CONDITION:



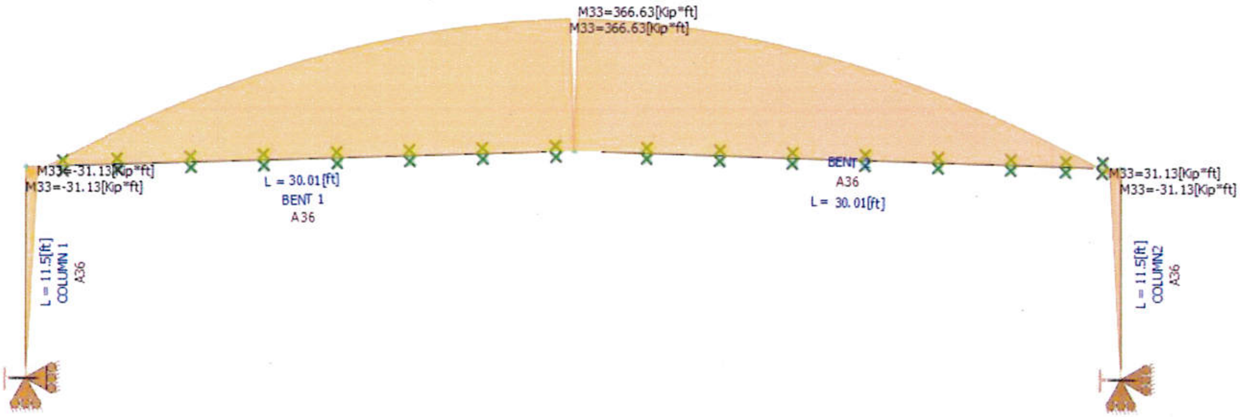
F_y = 36 ksi (TYP. FOR ERA)
W_D = 120 lb/ft = .12 k/ft
W_{SNOW}= 700 lb/ft = .70 k/ft (based on current NH snow load)
35psf SNOW LOAD

RENDERING OF BENT:



FOR 35psf LIVE LOAD

MOMENT DIAGRAM:



STEEL CODE CHECK:

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

a=DL+SL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
BENT 1	*Bay 1 Bent*	2	a at 62.50%	0.73	OK	Eq. H1-1b
BENT 2		3	a at 37.50%	0.73	OK	Eq. H1-1b
COLUMN 1	*Bay 1 Col*	1	a at 100.00%	1.68	N.G.	Eq. H1-1a
COLUMN2		4	a at 100.00%	1.68	N.G.	Eq. H1-1a

No good for
35psf live load

Bentley Microsoft

Current Date: 1/11/2016 10:28 AM
Units system: English
File name: P:\4358 Town of Exeter - DPW Roof Analysis\Reports\Structural Analysis\BAY 1\BAY 1exeter.etz\

Analysis result

Maximum forces at members

Condition : **a=DL+SL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-26.57	-2.71	0.00	0.00	0.00	0.00
Min	-26.73	-2.71	0.00	0.00	0.00	-31.13
MEMBER **2**						
Max	-2.71	26.47	0.00	0.00	0.00	366.63
Min	-3.54	-0.09	0.00	0.00	0.00	-31.13
MEMBER **3**						
Max	-2.71	0.09	0.00	0.00	0.00	366.63
Min	-3.54	-26.47	0.00	0.00	0.00	-31.13
MEMBER **4**						
Max	-26.57	2.71	0.00	0.00	0.00	31.13
Min	-26.73	2.71	0.00	0.00	0.00	0.00

MAX. MOMENT

Local deflections in members

(a) (b)

Condition : **a=DL+SL**

Station	Axis 1 [in]	Axis 2 [in]	Axis 3 [in]	Rotation11 [Rad]	Defl. (2) [in]	Defl. (3) [in]
MEMBER **1** Cantilever - type (a)						
0%	0.000	0.000	0.000	0.00000	-	-
25%	-0.008	0.257	0.000	0.00000	0.22083 (L/625)	-
50%	-0.015	0.425	0.000	0.00000	0.35333 (L/391)	-
75%	-0.023	0.417	0.000	0.00000	0.30916 (L/446)	-
100%	-0.030	0.144	0.000	0.00000	-	-
MEMBER **2** Cantilever - type (a)						
0%	-0.145	-0.026	0.000	0.00000	-	-
25%	-0.145	-1.231	0.000	0.00000	-0.61673 (L/584)	-
50%	-0.146	-2.113	0.000	0.00000	-0.91100 (L/395)	-
75%	-0.146	-2.411	0.000	0.00000	-0.62041 (L/581)	-
100%	-0.147	-2.378	0.000	0.00000	-	-
MEMBER **3** Cantilever - type (a)						
0%	0.004	-2.383	0.000	0.00000	-	-
25%	0.003	-2.275	0.000	0.00000	-0.48023 (L/750)	-

max. Δ

Local deflections in members

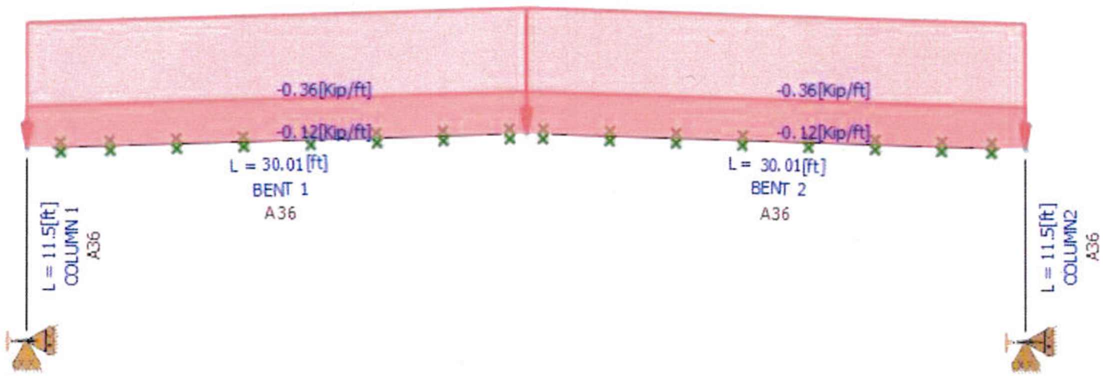
Condition : **a=DL+SL**

Station	Axis 1 [in]	Axis 2 [in]	Axis 3 [in]	Rotation11 [Rad]	Defl. (2) [in]	Defl. (3) [in]
MEMBER **3** Cantilever - type (a)						
50%	0.003	-1.858	0.000	0.00000	-0.65169 (L/553)	-
75%	0.002	-1.110	0.000	0.00000	-0.49145 (L/733)	-
100%	0.002	-0.030	0.000	0.00000	-	-
MEMBER **4** Cantilever - type (a)						
0%	0.000	0.000	0.000	0.00000	-	-
25%	-0.008	-0.221	0.000	0.00000	-0.22083 (L/625)	-
50%	-0.015	-0.354	0.000	0.00000	-0.35333 (L/391)	-
75%	-0.023	-0.310	0.000	0.00000	-0.30916 (L/446)	-
100%	-0.030	-0.001	0.000	0.00000	-	-

BAY 1: STRUCTURAL RATING ANALYSIS

(CONSTRUCTED LATE 1960'S-EARLY 1970'S)

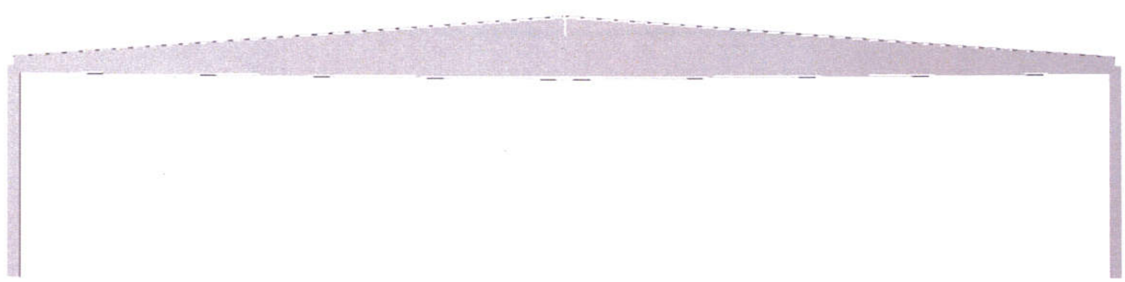
LOADING CONDITION:



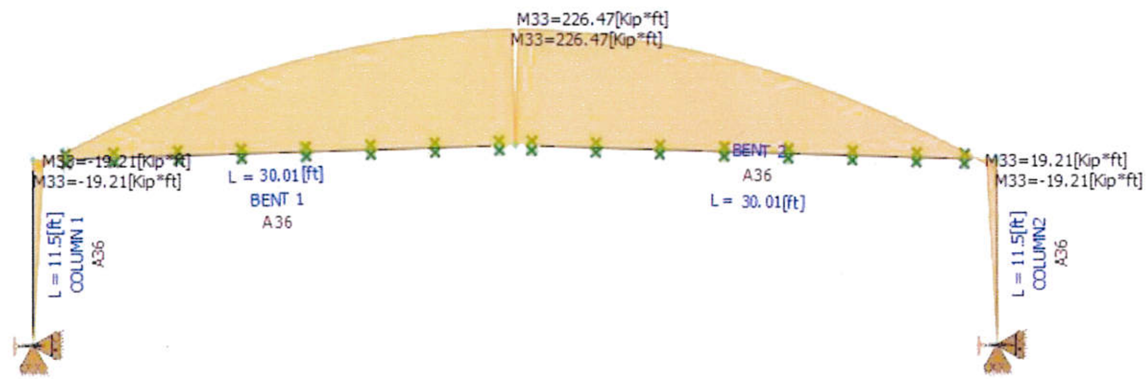
Fy = 36 ksi
W_D = 120 lb/ft = .12 k/ft
W_{SNOW} = 18 psf * (20') = 360 lb/ft

18psf Snow LOAD

RENDERING OF BENT:



MOMENT DIAGRAM:



STEEL CODE CHECK:

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :
a=DL+SL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
BENT 1	***Bay 1 Bent***	**2**	a at 62.50%	**0.45**	**OK**	Eq. H1-1b
BENT 2		**3**	a at 37.50%	**0.45**	**OK**	Eq. H1-1b
COLUMN 1	***Bay 1 Col***	**1**	a at 100.00%	**1.01**	**N.G.**	Eq. H1-2
COLUMN2		**4**	a at 100.00%	**1.01**	**N.G.**	Eq. H1-2

Close - Say ok

The H.L. Turner Group Inc.

Date 1/11/16

Job # 4358

Computed JEG

Subject TOWN OF EXETER - ROOF EVALUATION

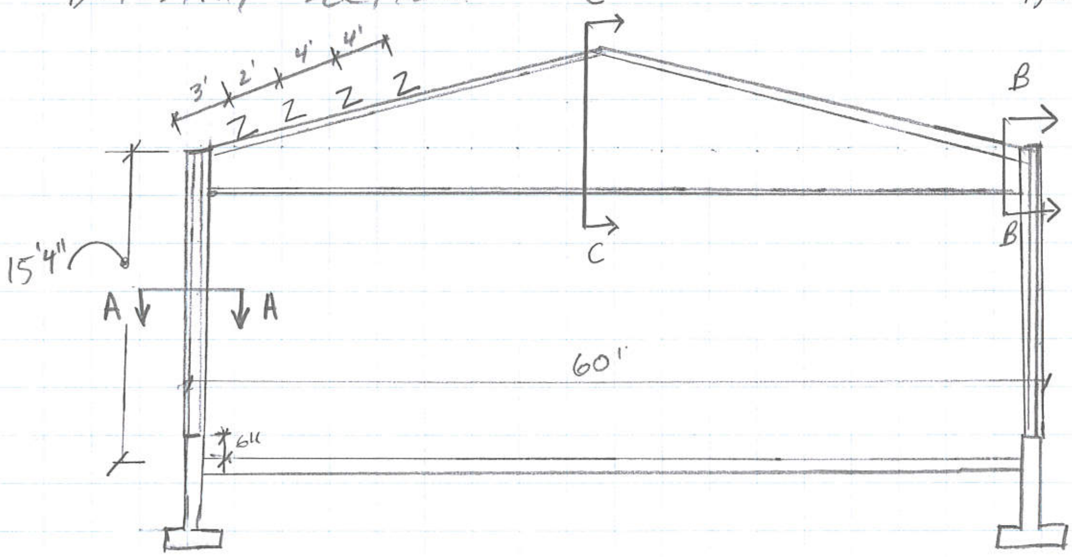
Checked PMB

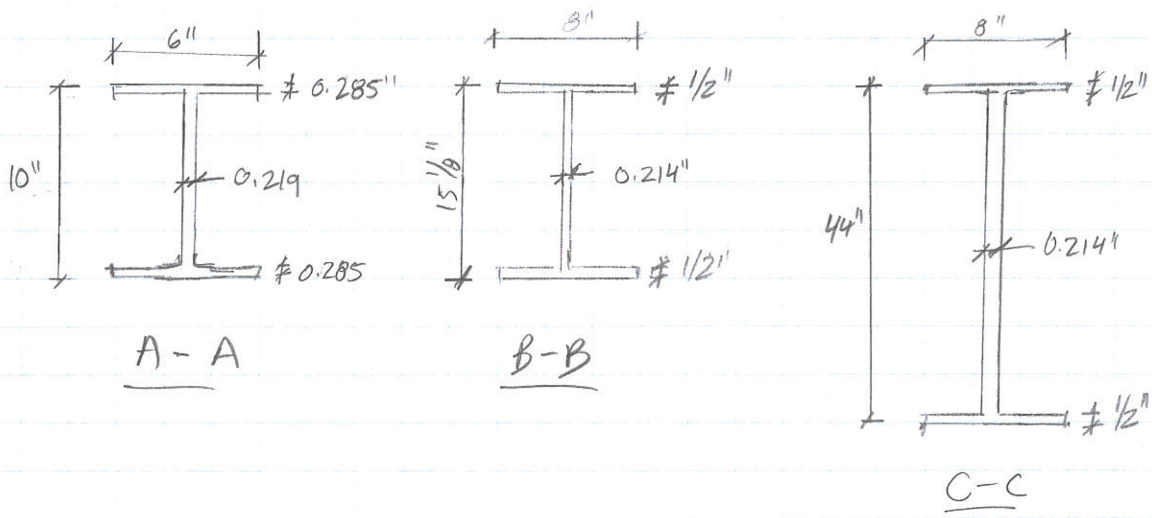
DEPARTMENT OF PUBLIC WORKS GARAGE

BAY 2: ADDITION TO ORIGINAL BLDG
CONSTRUCTED ≈ 1977 +/-

BUILDING SECTION:

BAY TO BAY = 20'





PURLIN SYSTEM IS SAME CONDITION AS BAY 1

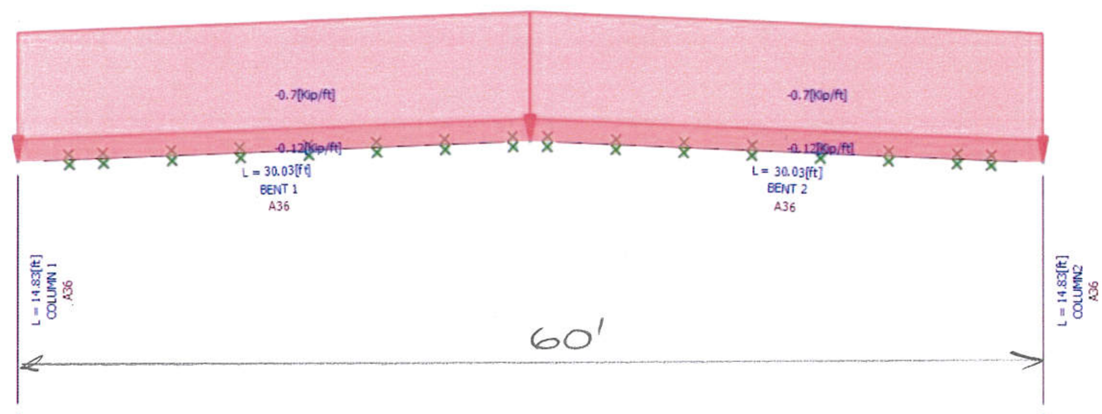
* COLUMNS DO NOT MEET CURRENT CODE
STRUCTURE RATED FOR 25 psf OF SNOW

DEPARTMENT OF PUBLIC WORKS GARAGE

BAY 2: STRUCTURAL ANALYSIS RESULTS

(CONSTRUCTED LATE MID TO LATE 1970'S)

LOADING CONDITION:

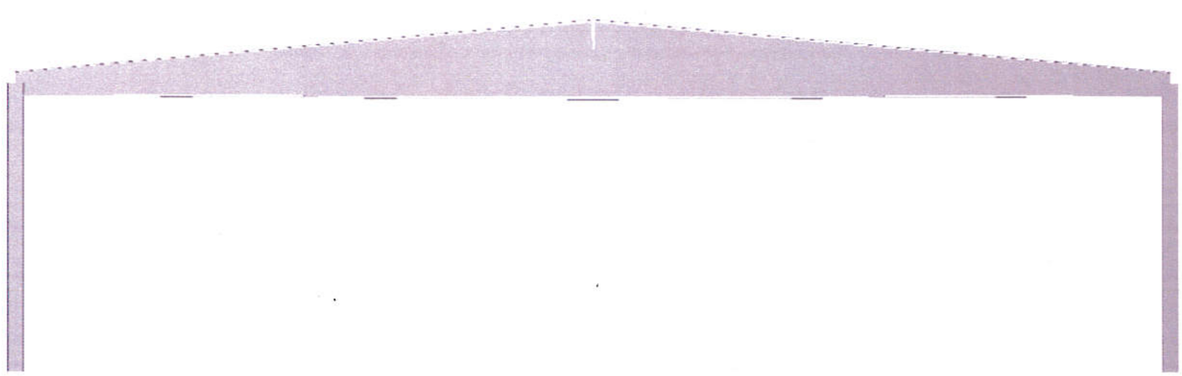


Fy = 36 ksi (TYP. FOR ERA)

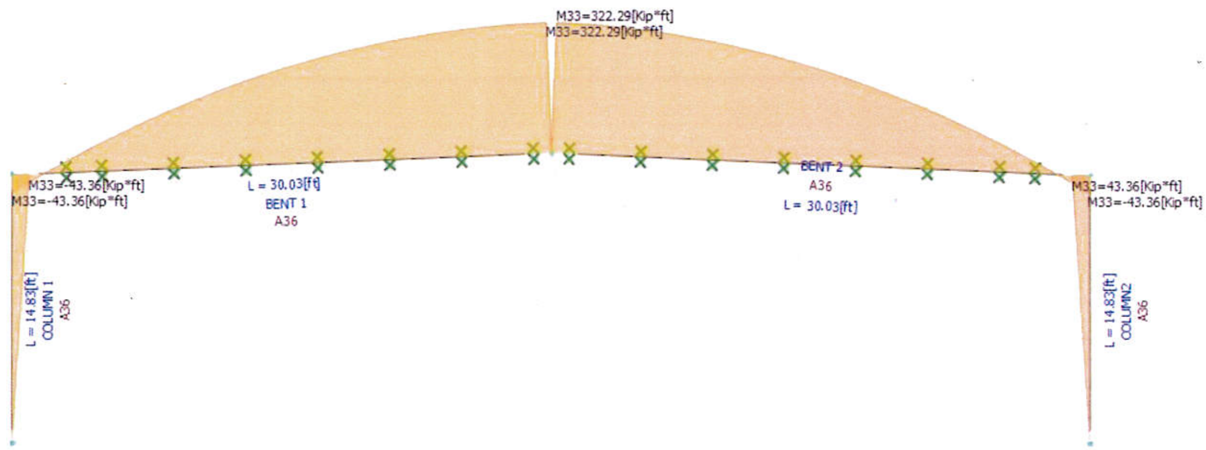
W_D = 120 lb/ft = .12 k/ft

W_{SNOW}= 700 lb/ft = .70 k/ft (based on current NH snow load [35PSF])

RENDERING OF BENT:



MOMENT DIAGRAM:



STEEL CODE CHECK:

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :
a=DL+SL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
BENT 1	*Bay 2 Bent*	2	a at 62.50%	0.85	OK	Eq. H1-1b
BENT 2		3	a at 37.50%	0.85	OK	Eq. H1-1b
COLUMN 1	*Bay 2 Col*	1	a at 100.00%	1.31	N.G.	Eq. H1-1a
COLUMN2		4	a at 100.00%	1.31	N.G.	Eq. H1-1a

NO GOOD
FOR 35psf
SNOW LOAD

Bentley Microsoft

Current Date: 1/11/2016 11:06 AM
Units system: English
File name: P:\4358 Town of Exeter - DPW Roof Analysis\Reports\Structural Analysis\BAY 2\BAY 2 exeter.etz\

Analysis result

Maximum forces at members

Condition : **a=DL+SL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-24.62	-2.92	0.00	0.00	0.00	0.00
Min	-24.62	-2.92	0.00	0.00	0.00	-43.36
MEMBER **2**						
Max	-2.92	24.48	0.00	0.00	0.00	322.29
Min	-3.95	-0.12	0.00	0.00	0.00	-43.36
MEMBER **3**						
Max	-2.92	0.12	0.00	0.00	0.00	322.29
Min	-3.95	-24.48	0.00	0.00	0.00	-43.36
MEMBER **4**						
Max	-24.62	2.92	0.00	0.00	0.00	43.36
Min	-24.62	2.92	0.00	0.00	0.00	0.00

MAX. MOMENT

Local deflections in members

Condition : **a=DL+SL**

Station	Axis 1 [in]	Axis 2 [in]	Axis 3 [in]	Rotation11 [Rad]	Defl. (2) [in]	Defl. (3) [in]
MEMBER **1** Cantilever - type (a)						
0%	0.000	0.000	0.000	0.00000	-	-
25%	-0.007	0.264	0.000	0.00000	0.22784 (L/781)	-
50%	-0.014	0.437	0.000	0.00000	0.36455 (L/488)	-
75%	-0.020	0.428	0.000	0.00000	0.31898 (L/558)	-
100%	-0.027	0.145	0.000	0.00000	-	-
MEMBER **2** Cantilever - type (a)						
0%	-0.146	-0.021	0.000	0.00000	-	-
25%	-0.147	-1.002	0.000	0.00000	-0.47206 (L/763)	-
50%	-0.148	-1.751	0.000	0.00000	-0.71174 (L/506)	-
75%	-0.149	-2.047	0.000	0.00000	-0.49820 (L/723)	-
100%	-0.149	-2.058	0.000	0.00000	-	-
MEMBER **3** Cantilever - type (a)						
0%	0.024	-2.063	0.000	0.00000	-	-
25%	0.023	-1.959	0.000	0.00000	-0.40464 (L/890)	-

max Δ

50%	0.022	-1.572	0.000	0.00000	-0.52694 (L/684)	-
75%	0.022	-0.916	0.000	0.00000	-0.38024 (L/948)	-
100%	0.021	-0.026	0.000	0.00000	-	-

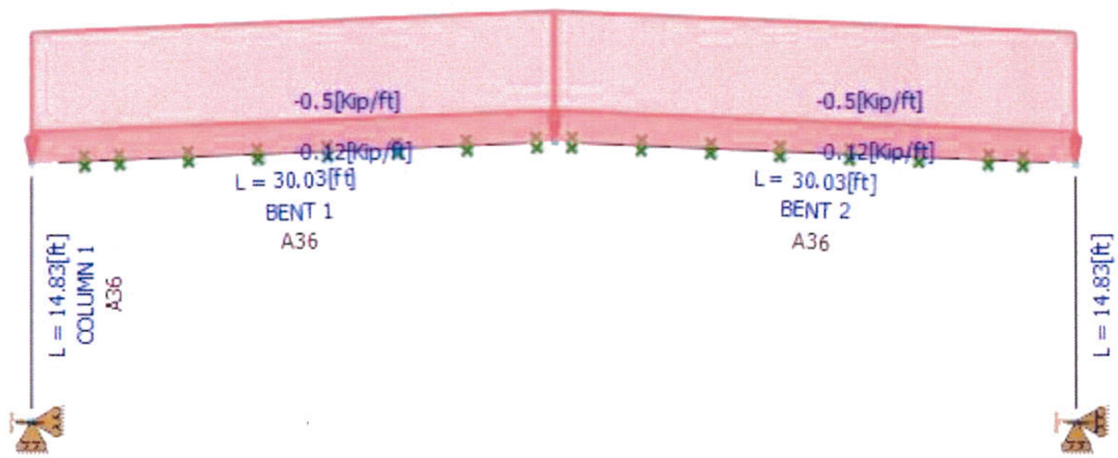
MEMBER **4** Cantilever - type (a)

0%	0.000	0.000	0.000	0.00000	-	-
25%	-0.007	-0.233	0.000	0.00000	-0.22784 (L/781)	-
50%	-0.014	-0.374	0.000	0.00000	-0.36455 (L/488)	-
75%	-0.020	-0.334	0.000	0.00000	-0.31898 (L/558)	-
100%	-0.027	-0.020	0.000	0.00000	-	-

BAY 2: STRUCTURAL RATING ANALYSIS

(CONSTRUCTED MID TO LATE 1970'S)

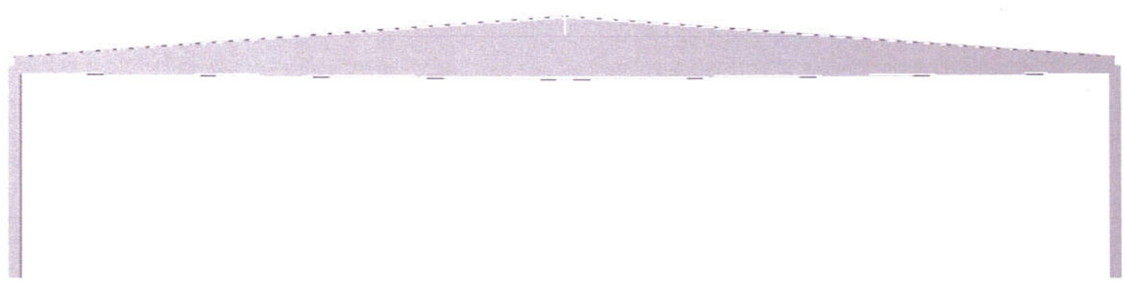
LOADING CONDITION:



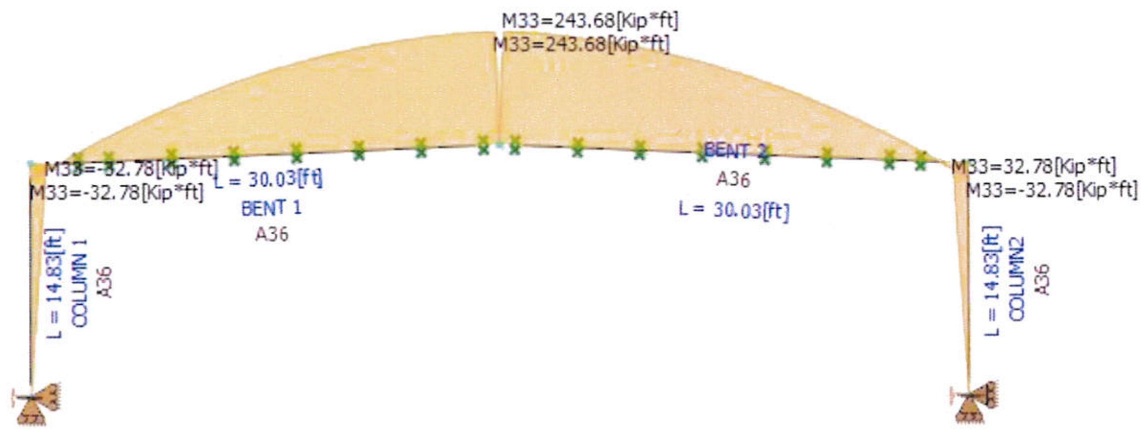
Fy = 36 ksi
W_D = 120 lb/ft = .12 k/ft
W_{SNOW}= 25 psf * (20') = 500 lb/ft

25psf snow load

RENDERING OF BENT:



MOMENT DIAGRAM:



STEEL CODE CHECK:

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

a=DL+SL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
BENT 1	*Bay 2 Bent*	2	a at 62.50%	0.64	OK	Eq. H1-1b
BENT 2		3	a at 37.50%	0.64	OK	Eq. H1-1b
COLUMN 1	*Bay 2 Col*	1	a at 100.00%	0.95	OK	Eq. H1-2
COLUMN2		4	a at 100.00%	0.95	OK	Eq. H1-2

OK for 25psf snow

Job # 4358

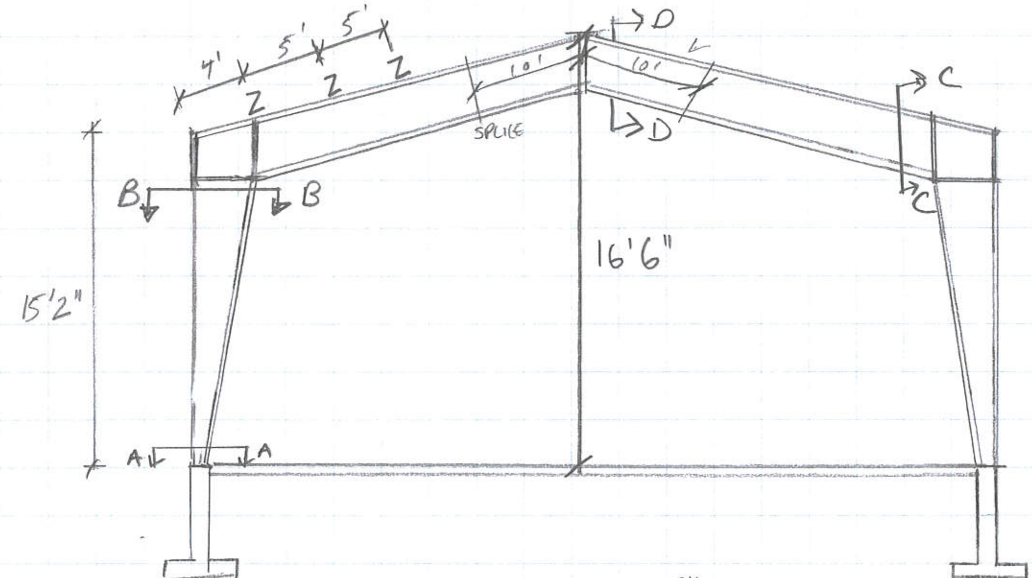
Subject TOWN OF EXETER - ROOF ANALYSIS

Computed JEG

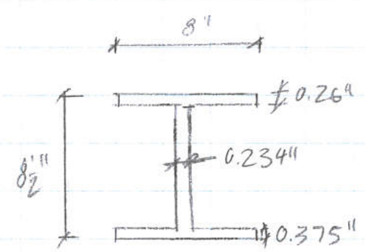
Checked DMB

WATER DEPARTMENT GARAGE

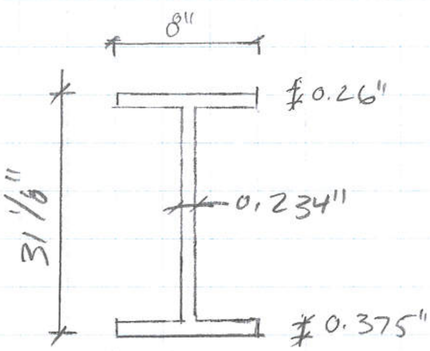
BAY 3: NEWEST METAL BLDG ON SITE
CONSTRUCTED ≈ LATE 1980'S



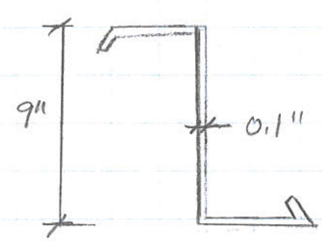
BAY TO BAY = 20'



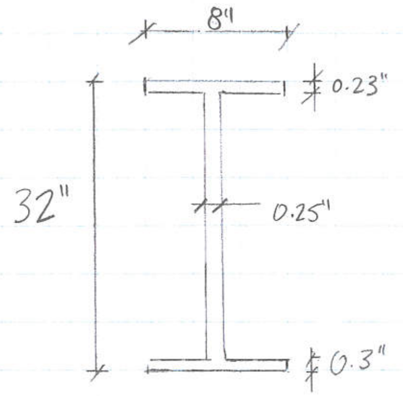
A - A



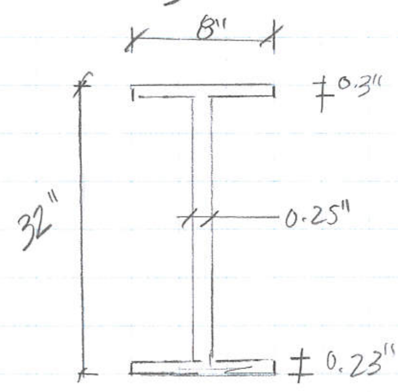
B-B



PURLIN (5' O.C)



C - C



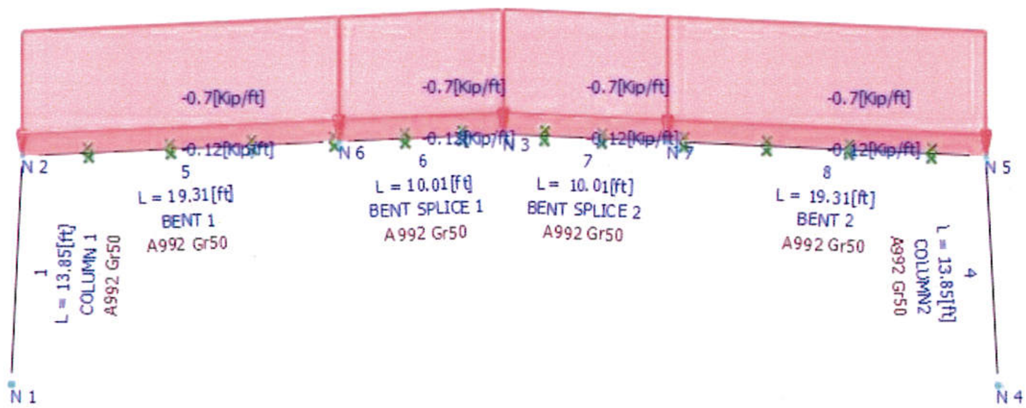
D-D

* STEEL BENT MEETS CURRENT CODE.

BAY 3: STRUCTURAL ANALYSIS RESULTS

(CONSTRUCTED LATE 1980'S)

LOADING CONDITION:



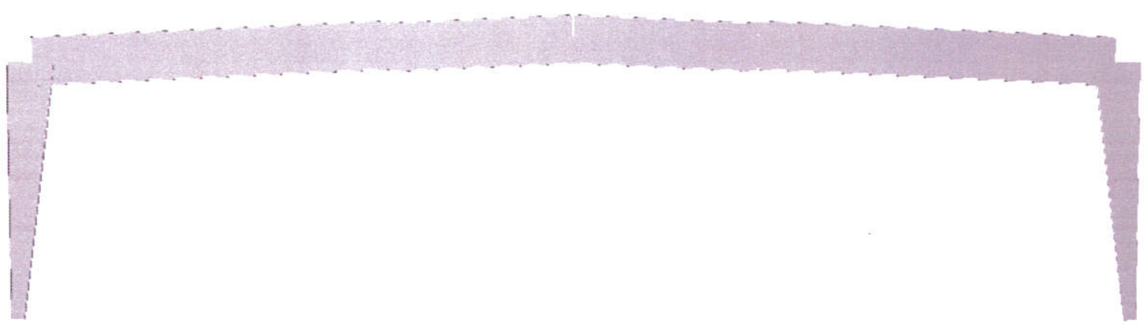
Fy = 50 ksi (TYP. FOR ERA)

W_D = 120 lb/ft = .12 k/ft

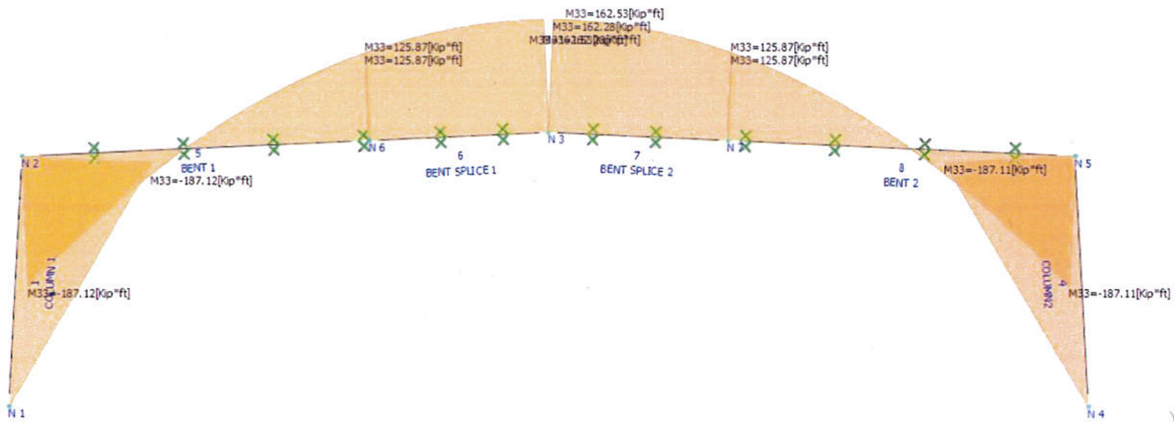
W_{SNOW}= 700 lb/ft = .70 k/ft (based on current NH snow load [35PSF])

35psf SNOW LOAD

RENDERING OF BENT:



MOMENT DIAGRAM:



STEEL CODE CHECK:

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

a=DL+SL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
BENT 1	*BAY 3 BENT (A)*	5	a at 0.00%	0.99	OK	Eq. H1-1a
BENT 2		8	a at 100.00%	0.99	OK	Eq. H1-1a
BENT SPLICE 1	*BAY 3 BENT (B)*	6	a at 93.75%	0.81	OK	Eq. H1-1b
BENT SPLICE 2		7	a at 6.25%	0.81	OK	Eq. H1-1b
COLUMN 1	*BAY 3 COL*	1	a at 100.00%	0.73	OK	Eq. H1-1b
COLUMN2		4	a at 100.00%	0.73	OK	Eq. H1-1b

OK for 35psf Snow LOAD

Bentley Microsoft

Current Date: 1/11/2016 2:34 PM
Units system: English
File name: P:\4358 Town of Exeter - DPW Roof Analysis\Reports\Structural Analysis\BAY 3\BAY 3 exeter.etz\

Analysis result

Maximum forces at members

Condition : **a=DL+SL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	-25.94	-13.50	0.00	0.00	0.00	0.00
Min	-26.38	-13.52	0.00	0.00	0.00	-187.12
MEMBER **4**						
Max	-25.94	-13.50	0.00	0.00	0.00	0.00
Min	-26.38	-13.52	0.00	0.00	0.00	-187.11
MEMBER **5**						
Max	-15.21	24.50	0.00	0.00	0.00	125.87
Min	-15.97	7.91	0.00	0.00	0.00	-187.12 max Moment
MEMBER **6**						
Max	-14.82	7.93	0.00	0.00	0.00	162.53
Min	-15.20	-0.66	0.00	0.00	0.00	125.87
MEMBER **7**						
Max	-14.82	0.66	0.00	0.00	0.00	162.53
Min	-15.20	-7.93	0.00	0.00	0.00	125.87
MEMBER **8**						
Max	-15.21	-7.91	0.00	0.00	0.00	125.87
Min	-15.97	-24.50	0.00	0.00	0.00	-187.11

Local deflections in members

Condition : **a=DL+SL**

Station	Axis 1 [in]	Axis 2 [in]	Axis 3 [in]	Rotation11 [Rad]	Defl. (2) [in]	Defl. (3) [in]
MEMBER **1** Cantilever - type (a)						
0%	0.000	0.000	0.000	0.00000	-	-
25%	-0.004	0.157	0.000	0.00000	0.14194 (L/1171)	-
50%	-0.008	0.223	0.000	0.00000	0.19167 (L/867)	-
75%	-0.012	0.192	0.000	0.00000	0.14563 (L/1141)	-
100%	-0.016	0.062	0.000	0.00000	-	-
MEMBER **4** Cantilever - type (a)						
0%	0.000	0.000	0.000	0.00000	-	-
25%	-0.004	0.157	0.000	0.00000	0.14194 (L/1171)	-
50%	-0.008	0.223	0.000	0.00000	0.19167 (L/867)	-

75%	-0.012	0.192	0.000	0.00000	0.14563 (L/1141)	-
100%	-0.016	0.062	0.000	0.00000	-	-
MEMBER **5** Cantilever - type (a)						
0%	-0.063	-0.010	0.000	0.00000	-	-
25%	-0.066	-0.337	0.000	0.00000	0.02808 (L/8254)	-
50%	-0.068	-0.729	0.000	0.00000	-	-
75%	-0.071	-1.113	0.000	0.00000	-0.03734 (L/6207)	-
100%	-0.073	-1.431	0.000	0.00000	-	-
MEMBER **6** Cantilever - type (a)						
0%	-0.071	-1.431	0.000	0.00000	-	-
25%	-0.073	-1.556	0.000	0.00000	-0.05202 (L/2309)	-
50%	-0.074	-1.647	0.000	0.00000	-0.07095 (L/1693)	-
75%	-0.075	-1.703	0.000	0.00000	-0.05410 (L/2220)	-
100%	-0.077	-1.722	0.000	0.00000	-	-
MEMBER **7** Cantilever - type (a)						
0%	0.077	-1.722	0.000	0.00000	-	-
25%	0.075	-1.703	0.000	0.00000	-0.05410 (L/2220)	-
50%	0.074	-1.647	0.000	0.00000	-0.07095 (L/1693)	-
75%	0.073	-1.556	0.000	0.00000	-0.05202 (L/2309)	-
100%	0.071	-1.431	0.000	0.00000	-	-
MEMBER **8** Cantilever - type (a)						
0%	0.073	-1.431	0.000	0.00000	-	-
25%	0.071	-1.113	0.000	0.00000	-0.03734 (L/6207)	-
50%	0.068	-0.729	0.000	0.00000	-	-
75%	0.066	-0.337	0.000	0.00000	0.02808 (L/8254)	-
100%	0.063	-0.010	0.000	0.00000	-	-

8" PURLINS: (ORIGINAL BLDG.)

FLANGE WIDTH = 3" &
FLANGE CLIP = 0.95"

ℓ = 20' ; SPACING (MAX.) = 4'-0" O.C
F_y = 36 KSI

LOADING:

W_D = 6 psf (4) = 24 lb/ft = 0.024 K/ft
W_{SNOW} = 35 psf (4') = 140 lb/ft = 0.140 K/ft

* INPUT DATA INTO STRUCTURAL ANALYSIS

PURLIN DOES NOT MEET CURRENT CODE

STEEL CODE CHECK RATIO = 1.50 (N.G.)

* PURLIN IS RATED FOR 20 psf SNOW
STEEL CODE CHECK RATIO = 0.96 (O.K.)

9" PURLINS:

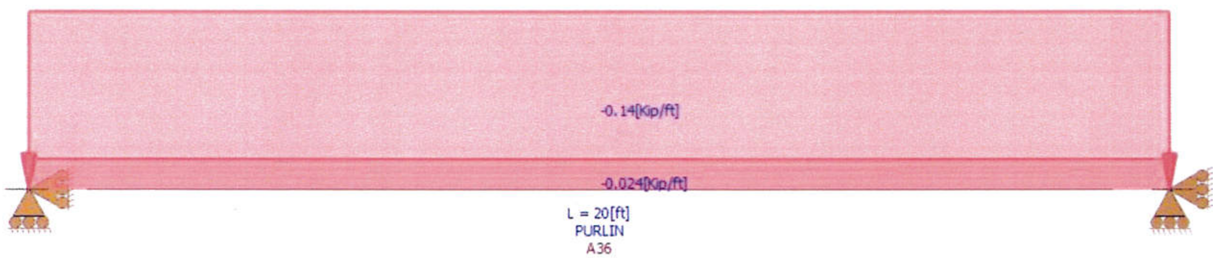
W_D = 6 psf (5') = 30 plf = 0.03 klf
W_S = 35 psf (5') = 175 plf = 0.175 klf

DOES NOT MEET CURRENT CODE

* PURLIN IS RATED FOR 30 psf SNOW

PURLIN BAY 1&2: STRUCTURAL ANALYSIS RESULTS

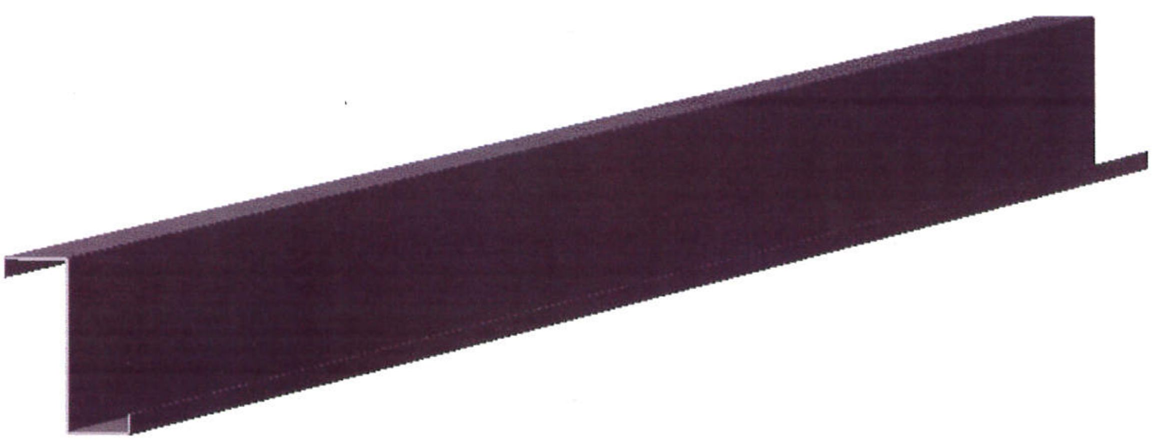
LOADING CONDITION:



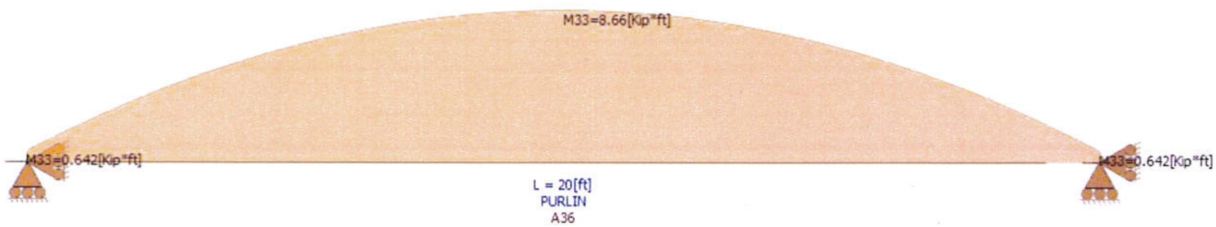
TRIBUTARY WIDTH = 4' (MAX.)
ASSUMING CONTINUOUS COMPRESSION FLANGE BRACING

Fy = 36 ksi (TYP. FOR ERA)
W_D = 24 lb/ft = 0.024 k/ft
W_{SNOW}= 140 lb/ft = 0.14 k/ft (based on current NH snow load [35 PSF])

RENDERING OF PURLIN:



MOMENT DIAGRAM:



STEEL CODE CHECK:

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

a=DL+SL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
PURLIN	*Bay 1 - P*	1	a at 50.00%	1.50	N.G.	Eq. H1-1b

NO GOOD
FOR 35psf
SNOW LOAD

Bentley Microsoft

Current Date: 1/11/2016 10:53 AM
Units system: English
File name: P:\4358 Town of Exeter - DPW Roof Analysis\Reports\Structural Analysis\PURLIN - BAY 1 & 2\BAY 1 PURLIN exeter.etz\

Analysis result

Maximum forces at members

Condition : **a=DL+SL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	0.00	1.60	0.58	0.00	1.12	8.66
Min	0.00	-1.60	-0.58	0.00	-1.78	0.64

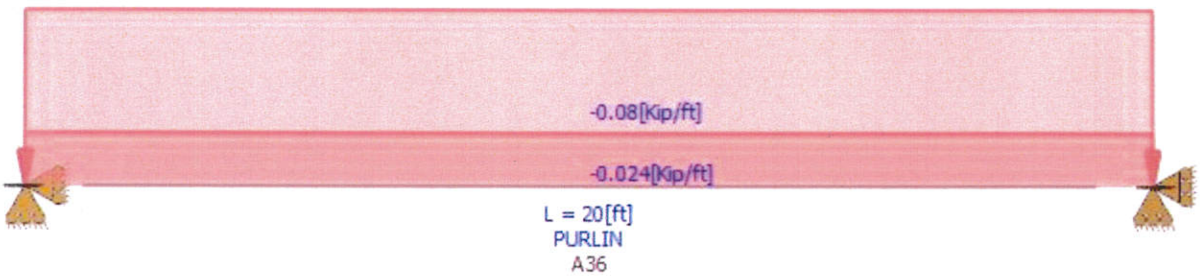
Local deflections in members

Condition : **a=DL+SL**

Station	Axis 1 [in]	Axis 2 [in]	Axis 3 [in]	Rotation11 [Rad]	Defl. (2) [in]	Defl. (3) [in]
MEMBER **1** Cantilever - type (a)						
0%	0.000	0.000	0.000	0.00000	-	-
25%	0.000	-0.953	0.459	0.00000	-0.95254 (L/252)	0.45902 (L/523)
50%	0.000	-1.394	0.816	0.00000	-1.39368 (L/172)	0.81603 (L/294)
75%	0.000	-0.953	0.459	0.00000	-0.95254 (L/252)	0.45902 (L/523)
100%	0.000	0.000	0.000	0.00000	-	-

PURLIN BAY 1&2: STRUCTURAL RATING ANALYSIS

LOADING CONDITION:

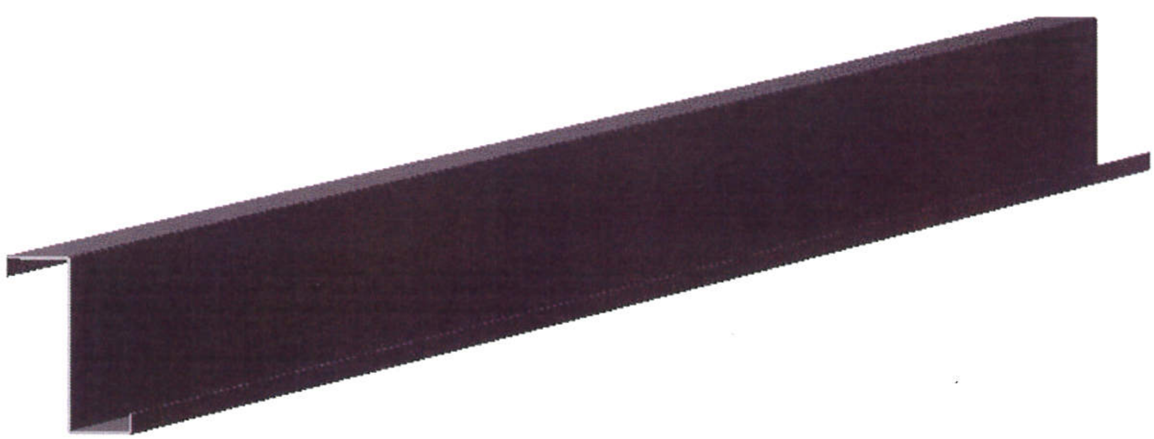


TRIBUTARY WIDTH = 4' (MAX.)
ASSUMING CONTINUOUS COMPRESSION FLANGE BRACING

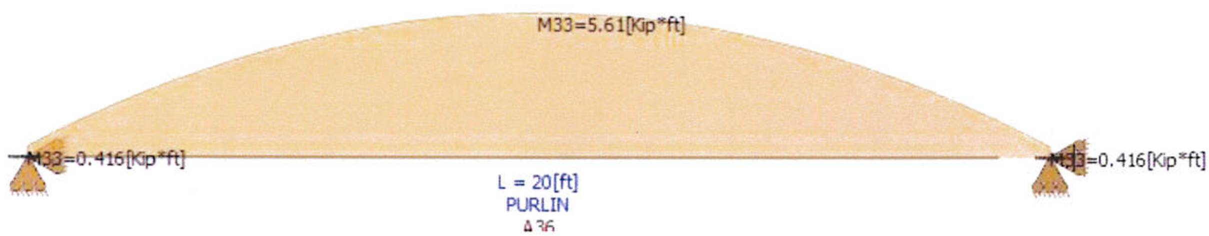
Fy = 36 ksi
W_D = 24 lb/ft = 0.024 k/ft
W_{SNOW}= 20 psf * (4') = 80 lb/ft

20 psf Snow LOAD

RENDERING OF PURLIN:



MOMENT DIAGRAM:



STEEL CODE CHECK:

Steel Code Check

Report: Summary - For all selected load conditions

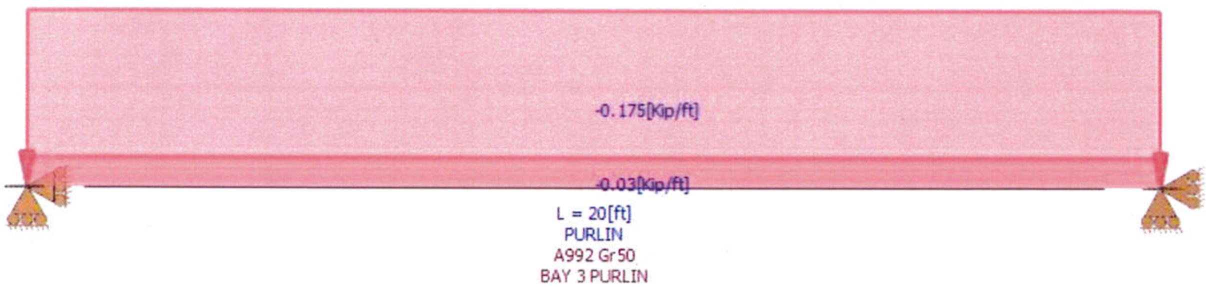
Load conditions to be included in design :
a=DL+SL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
PURLIN	*Bay 1 - P*	1	a at 50.00%	0.97	OK	Eq. H1-1b

OK for 20 psf Snow LOAD

PURLIN BAY 3: STRUCTURAL ANALYSIS RESULTS

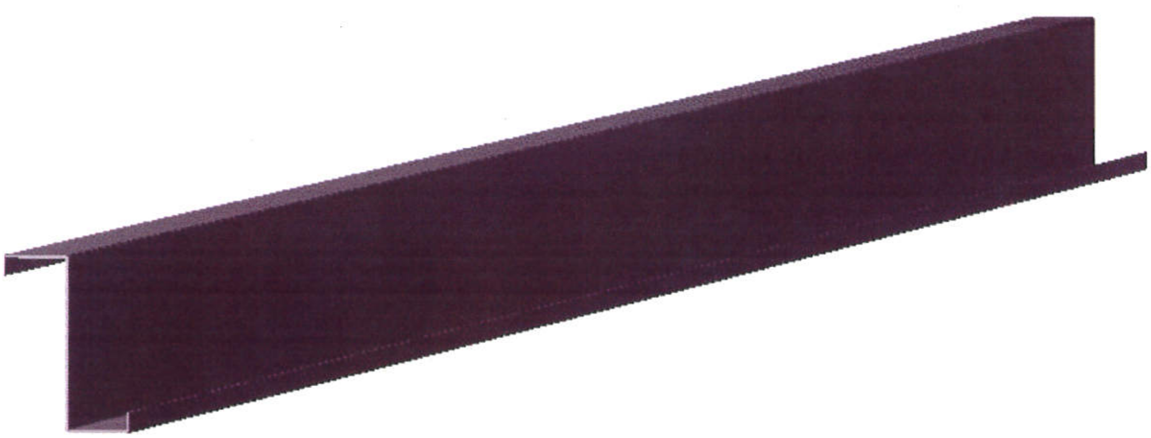
LOADING CONDITION:



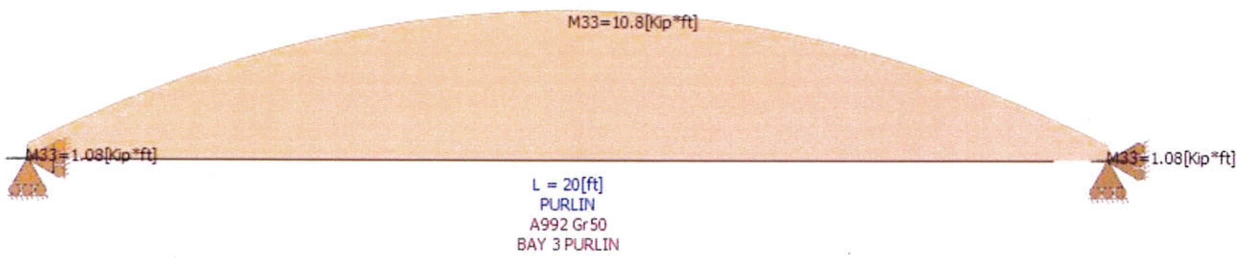
TRIBUTARY WIDTH = 5' (MAX.)
ASSUMING CONTINUOUS COMPRESSION FLANGE BRACING

Fy = 50 ksi (TYP. FOR ERA)
W_D = 30 lb/ft = 0.03 k/ft
W_{SNOW}= 175 lb/ft = 0.175 k/ft (based on current NH snow load [35 PSF])

RENDERING OF PURLIN:



MOMENT DIAGRAM:



STEEL CODE CHECK:

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :

a=DL+SL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
PURLIN	*BAY 3 PURLIN*	1	a at 50.00%	1.16	N.G.	Eq. H1-1b

Bentley Microsoft

Current Date: 1/11/2016 3:21 PM
Units system: English
File name: P:\4358 Town of Exeter - DPW Roof Analysis\Reports\Structural Analysis\PURLIN BAY 3\BAY 3 PURLIN exeter.etz\

Analysis result

Maximum forces at members

Condition : **a=DL+SL**

	Axial [Kip]	Shear V2 [Kip]	Shear V3 [Kip]	Torsion [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
MEMBER **1**						
Max	0.00	1.94	0.85	0.00	1.75	10.80
Min	0.00	-1.94	-0.85	0.00	-2.49	1.08

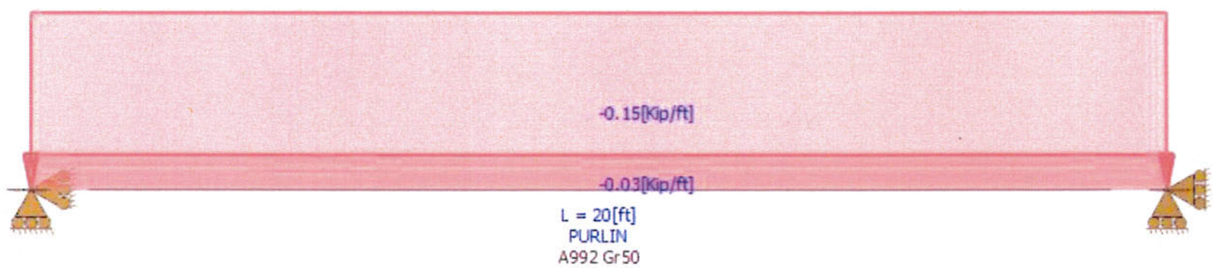
Local deflections in members

Condition : **a=DL+SL**

Station	Axis 1 [in]	Axis 2 [in]	Axis 3 [in]	Rotation11 [Rad]	Defl. (2) [in]	Defl. (3) [in]
MEMBER **1** Cantilever - type (a)						
0%	0.000	0.000	0.000	0.00000	-	-
25%	0.000	-0.956	0.355	0.00000	-0.95611 (L/251)	0.35524 (L/676)
50%	0.000	-1.393	0.632	0.00000	-1.39259 (L/172)	0.63155 (L/380)
75%	0.000	-0.956	0.355	0.00000	-0.95611 (L/251)	0.35524 (L/676)
100%	0.000	0.000	0.000	0.00000	-	-

PURLIN BAY 3: STRUCTURAL RATING ANALYSIS

LOADING CONDITION:

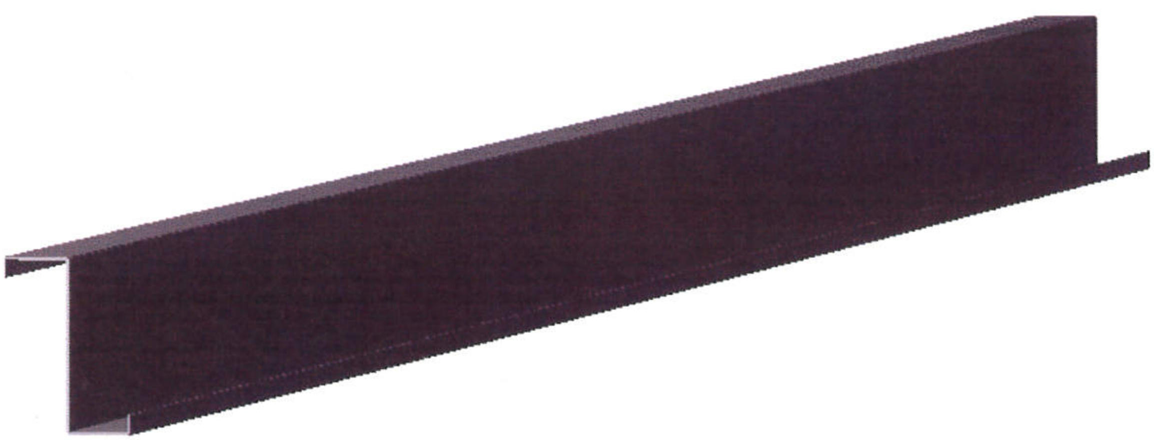


TRIBUTARY WIDTH = 5' (MAX.)
ASSUMING CONTINUOUS COMPRESSION FLANGE BRACING

Fy = 50 ksi
W_D = 30 lb/ft = 0.03 k/ft
W_{SNOW}= 30 psf * (5') = 150 lb/ft

30psf Snow LOAD

RENDERING OF PURLIN:



MOMENT DIAGRAM:

STEEL CODE CHECK:

Steel Code Check

Report: Summary - For all selected load conditions

Load conditions to be included in design :
a=DL+SL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
PURLIN	*BAY 3 PURLIN*	1	a at 50.00%	1.02	N.G.	Eq. H1-1b

SAY OK

CORPORATE OFFICE:

27 Locke Road
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