STRUCTURAL BARN EVALUATION

243 POST ROAD
BOWDOINHAM, MAINE 04008

Prepared for:

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Prepared by:

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Inspection Date: September 25, 2020
Submitted: October 5, 2020

Project No. 20-0377-ME

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Introduction

Criterium Engineers is pleased to provide a structural evaluation of the building located at 243 Post Road in Bowdoinham, Maine. This building was built as a 3-story chicken barn around 90 years ago, was extended to the back shortly thereafter with a similar construction, and has been in used for storage and as the Town of Bowdoinham's Recycling Barn for 30 years. The building also includes one rental apartment on the third floor. The floor loading capacity, and the structural strength of the roof, has been evaluated by various structural engineers. This evaluation is to provide a second opinion.

Criterium Engineer, Helen C. Watts, P.E. (ME), visited the site on September 25, 2020 to inspect the structural condition of the barn. We met onsite with David Berry, who provided additional engineering reports done in 2008 through 2011. The weather was warm and dry.

Standards and Limitations

Our inspection report is limited to observations made from visual evidence and a review of the available engineering reports.

Our inspection and report has been conducted consistent with that level of care and skill that is ordinarily exercised by members of the profession providing the same services under similar conditions at the time the services are performed.

Our report is an opinion about the condition of this portion of the building. It is based on evidence available during a diligent inspection of all reasonably accessible areas. No surface materials were removed, no destructive testing undertaken, and no furnishings moved.

Description

The barn is 36'x288', and approximately 22' from the concrete slab to the eaves. The building is wood-framed, with corrugated metal roofing and siding, and a concrete slab-on-grade with concrete frost walls. There are two lines of columns and beams at 11'-8" from the south and north sides, and the posts are spaced 12' on center, generally with 2x8 construction for the beams and 6x6 or built-up 6x6s for the posts. The posts have some angle bracing. There is no siding on the south face of the building. The building has been modified to remove flooring in some areas, and the siding on the south side was removed and replaced with clear plastic. There is a ground floor addition made of concrete block on a concrete slab and foundation housing the wood boiler heating the building and the apartment.

The barn roof had a partial collapse the winter of 2011 near the back at the south side, at which point the south roof framing was reinforced.

There have been various structural evaluations to determine the floor loading capacity of the building, which contains the municipal recycling program, and which uses parts of all three floors.

For the purposes of this report, the orientation of the building will be discussed as north, south, east, west, , with the 36′ end facing Post Road being the west end, and the 288′ right side being the south side of the building.

See attached photos for more detail.

Observations and Discussion

During the inspection, we went around the outside of the building, north, west and east, then through the three levels of the building inside. The back of the building is grown up with vegetation.

We noted the following items during our inspection:

1. The life safety issues have been evaluated by the State Fire Marshal's office and are not included in this report.
2. The original barn is at the west end of the barn, then the barn was extended to the east. The barn was built as a chicken barn, and includes a large central ventilator on the roof, and a boiler room added near the middle of the barn at the first floor.
3. The framing is in three bays of 2x8 joists at 24″ on center, with 12′ spans. The joists land on two interior wood beams running front-to-back, originally built with 3 – 2x8s, spanning 12′ from post-to-post. The posts are 6x6 solid or built-up posts. There are 45 degree angle braces from the post to the beams, and in some locations there are angle braces north-to-south as well. The material appears to be hemlock or pine. The floor sheathing is wood boards. The joists are generally in good condition, though in some areas they are stained and have debris left over from the chicken barn use. The beams supporting the joists are generally deflected in the center.
4. The barn is somewhat taller than most chicken barns I've inspected, though the floor-to framing heights are still limited.
5. The roof framing is at 36″ on center, and consists of 2x6 or 2x8 rafter framing with eaves ties for each pair of rafters. This supports nailers supporting the corrugated metal roof. The framing is insulated at the front half of the building with a mix of fiberglass batts and blown cellulose insulation; some areas towards the east end are uninsulated.
6. The original interior wood beams have some deflection at the center of the spans for most of the beam spans. Some beams have been reinforced by sistering a 2x10 on each side of the beam.
7. Some floor areas have been removed, including the second floor framing at the front of the building in the center aisle, the second floor framing near the center of the building in the north aisle, and the second floor framing at the back end of the building. The removed post-and-beam framing has been replaced with wood trusses to replace the removed supports for the third floor.

8. There is an apartment on the third floor covering the center and south bays of the building, with an exterior stair and an interior stair. The third floor framing in this area is covered with drywall.
9. The roof to the barn was rebuilt in 2011 in the area where the collapse occurred, comprising 30' at the south end of the barn.
10. The south side of the building has had the corrugated metal siding removed and a layer of clear plastic sheeting installed. This was done to allow some daylighting in the building, with some solar gain. The shear capacity of the siding was supplanted by letting in some diagonal bracing from the eaves to the sill plates at the top of the foundation walls.

Review of the Engineering Reports

It is our opinion that the Associated Design Partners (ADP) report dated May 27, 2011 is an inaccurate report; the dimensions of the building and the framing listed were incorrect.

The ADP report dated September 30, 2011 lists the building as having 2 stories and being 35' wide; we measured 36' out-to-out. The snow loading calculation assumes very good insulation in the roof and doesn't include the slippery roofing surface, resulting in a higher snow load than should be used in calculations. Note that the ASCE 7 snow loading requirements haven't changed since ASCE 7-05, when the unbalanced snow loading requirements were added, so the current requirements are the same.

The unbalanced loading requirements place the full, unfactored ground snow load on one side of a gable roof; these are the typical snow conditions that have been observed with this building, with no snow on the north side, and blown snow collecting on the south side of the building, and also the conditions under which the partial roof collapse occurred.

The Calderwood Engineering (CE) reports are dated July 3, 2008 and February 20, 2009, and cover roof loading and floor loading. The report uses slightly different factors than those that I feel are correct, and I have different dead load (building material weight numbers in some areas) so I gain some additional strength for the calculated floor and roof systems. However, the values I calculated are for a typical floor and roof system, and there have been modifications to the framing in various locations throughout the building, some of which add an adequate amount of strength, and some of which remain undersized. The need for framing modifications, and the size of the modifications, are similar.

None of the ADP or CE calculations included the use of the 1/8" steel plates, which add over 5 pounds per square foot (psf) of dead load to the floor system on the third floor, and which help spread the weight of the loaded pallet jack to multiple rafters. The plates provide a durable and smooth surface for the pallet jack.

CE inspected the barn and created a repair plan in 2013. They re-inspected the building in August of 2020, and found that the needed modifications hadn't been done. They also found other conditions of concern. These two reports include drawings for the repairs.

Discussion

Every wood-framed chicken barn in Maine has an ongoing list of maintenance needed. Most of these barns have a relatively low floor-to-framing height, and therefore don't support other uses well. This barn has been used for the last 30 years as a recycling barn. The areas of the barn that were especially lightly framed are typical for chicken barns but inadequate for the current building code requirements. The applicable building codes from the Maine Uniform Building and Energy Code are the 2015 IBC, the 2015 IEBC (the Existing Building Code), and the ASCE 7-10, which provides the loading requirements. The wood framing design is based on the 2012 NDS (National Design Specification), which uses wood graded to modern specifications by organizations such as NELMA (New England Lumber Manufacturer's Association).

For new buildings, the building framing design is based on the code-specified loading and deflection requirements, with modern building materials.

In this case, we have an existing building that has been built without engineering for an agricultural purpose, matching many other chicken barns built in the 1930s. The building was built without concern for deflection of individual members, and framed using light and repetitive framing. The building would have been warmed by the chickens as well as the boiler, minimizing snow loads on the roof.

When the building was repurposed into a new use as a recycling center, the barn was modified in some locations to make openings in the floors. Some of these modifications weren't engineered, and resulted in an inadequate structure.

Roof System

The roof system needs to be able to handle the expected snow loads adequately. These are best characterized by using the latest ASCE 7 requirements, which include the balanced snow loads, where a uniform load is on both sides of the rafter-framed gable roof. However, the barn roof failure in 2011 demonstrated the unbalanced loading condition, which was a new design requirement placed in the ASCE 7-05 and later versions, including the current ASCE 7-10. The unbalanced snow load needs to be applied to both the north and south sides of the roof, even though the prevailing wind usually has the north side clear and the south side snow-covered. A storm can come from any direction.

The rafters on the south side have been strengthened, but all the rafters should be upgraded to be made adequate for the unbalanced condition. This is reasonably correctly calculated by CE. Every engineer uses slightly different methods, but the framing needed for the repair will be similar when the drawings are stamped. CE has provided two repair methods, to allow the owner to select the least expensive option. Note that most contractors are now using engineered wood screws rather than bolts or lag screws; they are fast to install and make a robust connection, with less section loss in the wood part of the connection. There may be some economies available in revising the connection details.

The rafter beams are undersized, on both sides of the building. This is a typical problem with chicken barns. These can be sistered with LVLs (engineered lumber); and the beams should be made adequate for the unbalanced snow load. My rough calculations showed that two LVLs were needed, rather than one, for each beam.

The rafter tails should be fastened to the top plate of the wall with an uplift fastener such as the Simpson H2.5.

The CE design shows adding one 1.75"x11.25" Versalam to the beam supporting the rafters. I calculated the need for two Versalam beams; I assumed that the stronger and deeper material would take all the load from the 2x8s because the lower edge of the engineered lumber is taking the tension below the bottom of the 2x8s. This calculation should be checked.

The work should be inspected at the start of the work to assure that the design has been properly interpreted by the contractor, then at the end of the project.

Floor System

The analysis of existing building floor systems for a new use is per the 2015 IEBC as well as the 2015 IBC. The allowable loading can be determined by inspecting the building, evaluating the materials and their condition, then performing a structural analysis, or it can be determined by load testing.

While a new building will be analyzed based on a live load from Table 4.1 in the ASCE 7-10, and a new building built for storage of materials like these would have a live load rating of 125 psf for "Light Storage". In this case, an existing building is generally inspected and given a load rating based on the available structure, or, if additional loading is expected, that load is determined and the framing is upgraded to the required loading. All of the different areas to be used should be placarded for their available live load strength.

We also recommend that the areas used for loads over 30 psf be marked out and a design be made to take those loads down to the ground. The design should include the actual expected loads of the loaded pallet jack.

In any case, storage of loads over 30 psf (or 28 psf if using the CE calculations factors instead of mine) should be prioritized on the ground floor slab, and the upper floors should be used for lighter material storage. Pallets sent to the upper floors should be weighed before leaving the first floor, and can be marked or tagged. Heavier loads on the upper floor will be restricted to specifically enhanced floor framing areas. Loads placed on the upper floors should be weighed so the loads don't exceed the allowable storage load.

The Pallet Jack and Bale Transport in the West End

The calculated load on the third floor at the west end should include 5 psf of added dead load for the 1/8" steel plate. The live load for this area should be planned for the expected weight of the bales handled, and the weight of the pallet jack. Assuming a pallet jack weighing 200#, and a bale of materials handled at not-to-exceed 1000#, and multiplying by a factor of safety of 1.2 (20%), with a pallet being 4'x4', gives a floor live load rating requirement of 90 psf for the west end in the center and north bays. The floor joists, floor sheathing, beams and posts should be upgraded to handle this amount, down to the floor slab, but only in the area where this load occurs.

If we assume that the area of the third floor used for the pallet jack is just used for the one pallet jack, and that the rest of the span in that area is unloaded except for dead load, then only that load is on the framing at one time – so the 4x4 area carries 1440 sf, and the rest of the structure supporting the rest of the floor in that bay is unloaded except for the dead load. This would require restricting the load to the one pallet jack. Note that this is the current loading condition in the third floor near the truck opening. This means that no material would be stored in the same bay as the pallet jack travel. The floor that has been enhanced should be painted, or curbed, so the pallet jack can't travel beyond the enhanced floor area, and placarded for only that use with no storage.

Assuming that the 120 psf load is over 5' (for an added 25% factor of safety, and an amount of leeway in the location of the travelling load), the joists in the travel area should still be sistered, or a joist added in each bay, of 2x8 SPF #2 or HF #2. The beam supporting the joists would need to be 4 – 1.75"x11.25" 2.0E Microllams (or Versalams), 2 on each side of the existing 2x8s beam. These can be sistered to the existing beam. The posts can be checked by calculation later.

Other Floor Loading Issues

The CE design for the trusses uses some 1" diameter A325 bolts. These connections work well for steel-to-steel applications, but a steel-to-wood application should use more, smaller, connectors rather than one, larger, connector, to prevent wood failure at the joint.

The extensive repairs recommended for the large trusses at the east end of the building may be more simply addressed by installing a steel beam in these two locations, supported by the 12x12 columns as needed.

This is a large building. The enhancement of the floor load rating should be targeted at the areas with payback. All other areas should be restricted to light loads, and the loads should be put on a scale to prevent overloading before being sent to the framing above. Additional floor areas can be enhanced per the drawings as funded uses occur.

Other Structural Issues

The area around the composter should have the floor sheathing removed, so nothing can be stored there, or the floor joists can be removed and re-installed with full joists and new floor sheathing.

There are some areas with the original board floor sheathing. These areas should be inspected periodically, if kept in use, as the use for chickens may have deteriorated the strength properties over time. Most of the floors have been surfaced over with plywood, which is in good condition and adequately attached, where observed.

Further non-engineered changes to the framing should be avoided, as this is no longer an agricultural building use.

South Wall

The south wall of the building is sheathed with plastic sheeting stapled to the studs. Some limited diagonal braces have been let into the wall for the full height of the wall. I recommend that the building be fully evaluated and that parts of the wall be sheathed and fastened as needed per the evaluation. The sheathing would need to be fully attached to bring the shear loads from the roof framing down to the foundation. We discussed installing the sheathing on the inside; this would involve some complications to the wind design analysis. A better solution still allowing the solar gain may be to install the sheathing to the outside of the studs and then install a Trombe-wall type collector to the outside of the sheathing.

CE's inspection noted some damage to studs and lack of connection between the building and the foundation in one area. Repairs should be made per the Calderwood report.

Conclusion

The Recycling Barn was framed for use as a chicken barn, with light, wood framing. The current use exceeds the design capacity of the framing, as does the expected snow loading; the wind loading is expected to exceed the capacity of the south wall framing. Some modifications are needed to continue the current use, and to accommodate the expected snow loading without distress. Some repairs are needed as well.

The CE design generally meets the needs of the building, but our recommendation for the building is to only improve the second and third floor load rating for the areas that will be specifically used for high loads. The roof framing should be modified to one of the CE repair methods, or another engineered repair design, for the full area of the roof framing.

We hope that you will call if you have further questions concerning this report.

Respectfully submitted,

Criterium Engineers

STATE OF MAINE
HELEN C. WATTS
No.5261
LICENSED
PROFESSIONAL ENGINEER

Helen C. Watts, P.E. (ME)
Senior Structural Engineer

Attachments: Photographs
Schematic Floor Plans
Older Engineering Reports
2020 Calderwood Report and Plans
Watts Calculations
Professional Resume

ATTACHMENT A
PHOTOGRAPHS

Location:
Recycling Barn
243 Post Road
Bowdoinham, Maine

Photo Taken by:
Helen C. Watts, P.E. (ME)

Date:
September 25, 2020

CRITERIUM ENGINEERS

STRUCTURAL EVALUATION

Description:
North and West Exterior

Photo Number
1

Description:
West End of South Wall
Note Let-in Bracing, Plastic Sheathing

Photo Number
2

STRUCTURAL EVALUATION

Description:
South Exterior, Apartment is at the 3rd Floor

Photo Number
3

Description:
East end of South Wall, East Exterior

Photo Number
4

Location:
Recycling Barn
243 Post Road
Bowdoinham, Maine

Photo Taken by:
Helen C. Watts, P.E. (ME)

Date:
September 25, 2020

STRUCTURAL EVALUATION

Description:
West end bays with third floor roof hatch for truck loading

Photo Number

5

Description:
chicken pee debris on location of previous power lines

Photo Number

6

STRUCTURAL EVALUATION

Description:
southwest end 3rd with pallet jack, insulation at roof framing

Photo Number
7

Description:
typical roof framing no insulation

Photo Number
8

Description:
Inadequately supported floor area

Photo Number
9

Description:
truss at north wall opening

Photo Number
10

STRUCTURAL EVALUATION

Description:
roof framing at north side, east end, typical

Photo Number
11

Description:
roof framing with beefed up roof purlin

Photo Number
12

STRUCTURAL EVALUATION

Description:
typical joist and floor sheathing condition

Photo Number
13

Description:
south bay with second floor removed and added trusses taking column loads

Photo Number
14

Location:	*Photo Taken by:*	*Date:*
Recycling Barn	Helen C. Watts, P.E. (ME)	September 25, 2020
243 Post Road		
Bowdoinham, Maine		

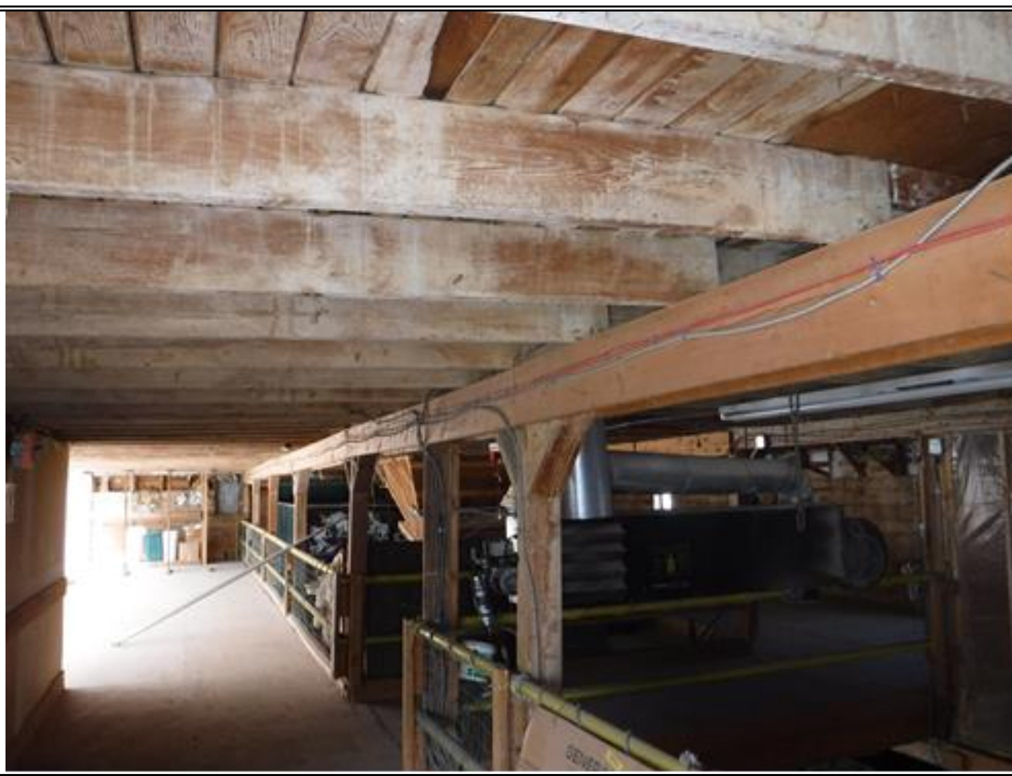
CRITERIUM ENGINEERS

STRUCTURAL EVALUATION

Description:
added beam to support third floor north bay

Photo Number

15



Description:
typical floor framing, with beam supporting joists strengthened

Photo Number

16

Location:
Recycling Barn
243 Post Road
Bowdoinham, Maine

Photo Taken by:
Helen C. Watts, P.E. (ME)

Date:
September 25, 2020

CRITERIUM ENGINEERS

STRUCTURAL EVALUATION

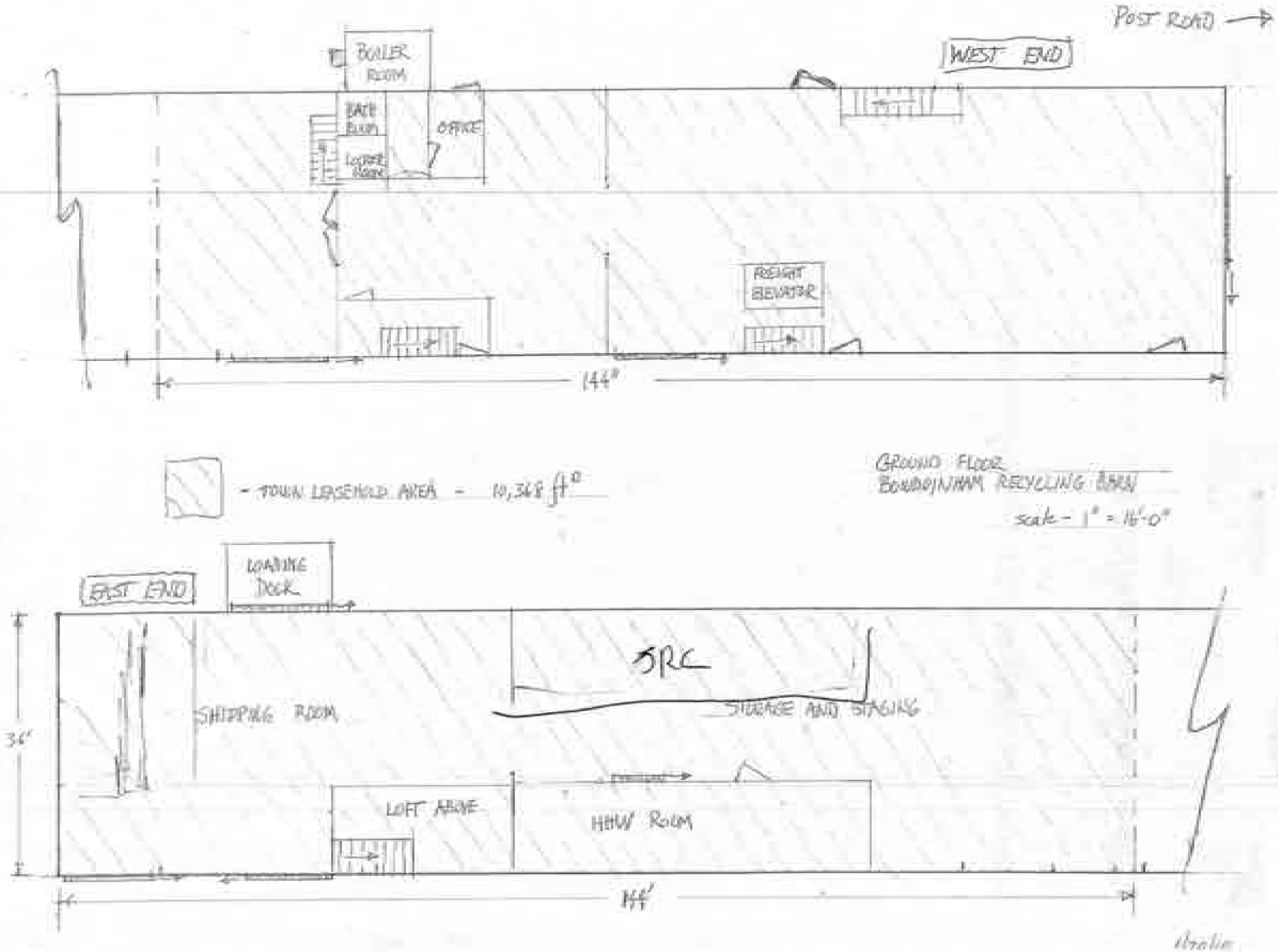
Description:
area of second floor should be checked

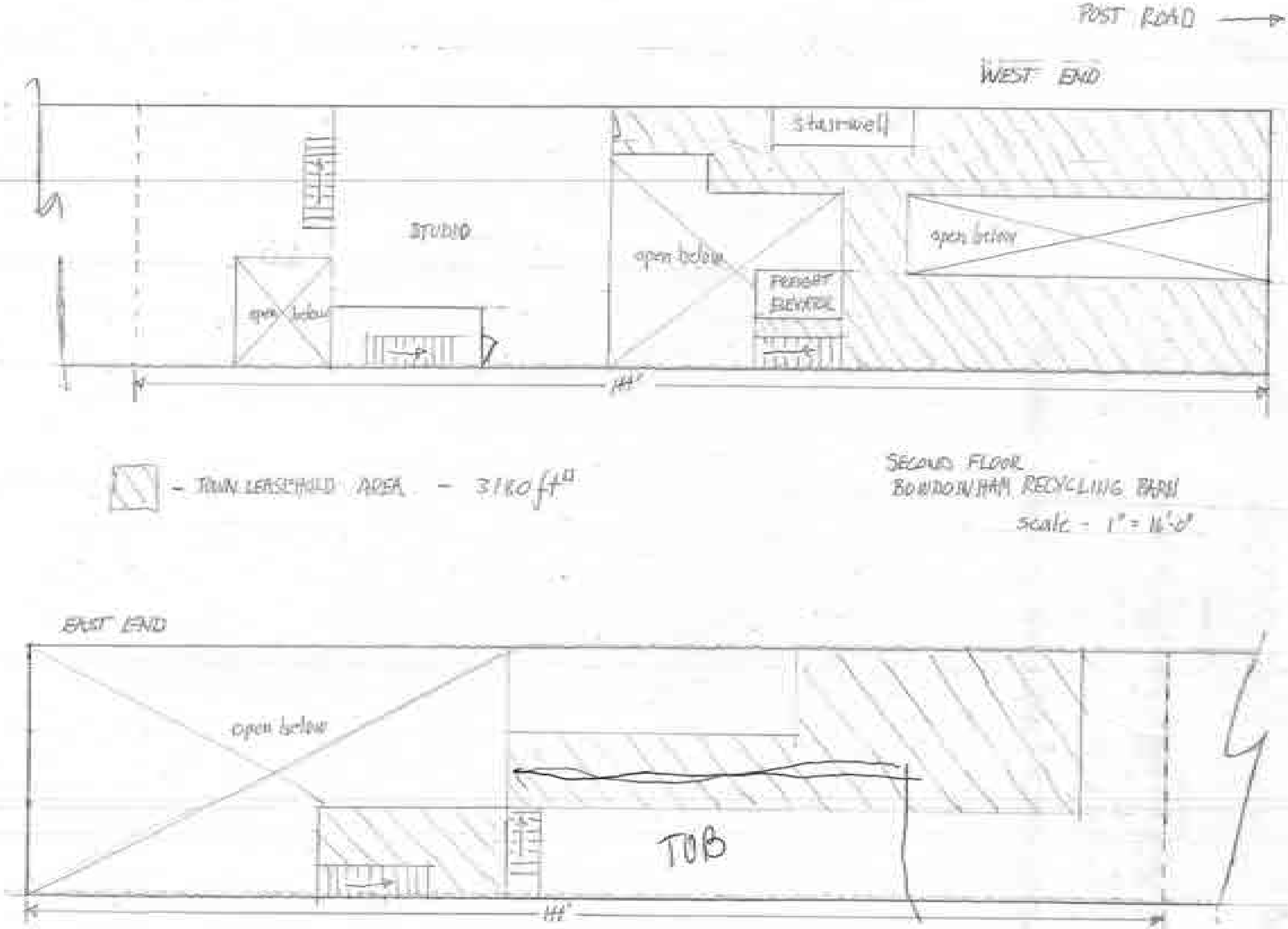
Photo Number
17

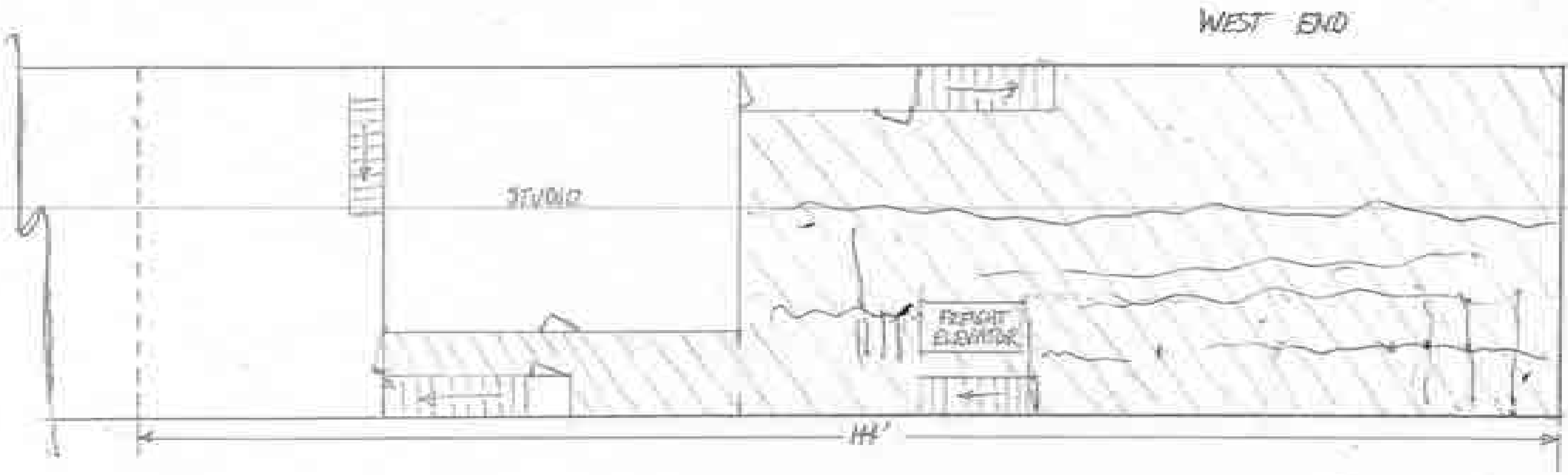
Description:
original floor sheathing

Photo Number
18

ATTACHMENT B
SCHEMATIC FLOOR PLANS

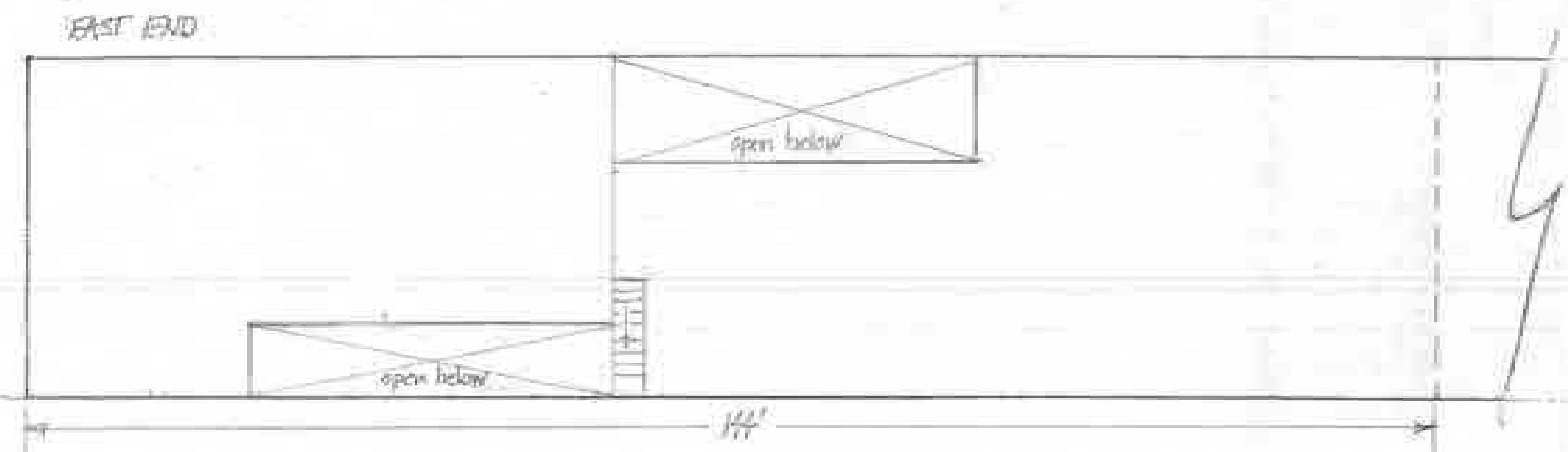

POST ROAD →
WEST END
BOILER ROOM
BATH ROOM
OFFICE
LOCKER ROOM
FREIGHT ELEVATOR
144'
- TOWN LEASEHOLD AREA - 10,368 ft²
GROUND FLOOR
BOWDOINHAM RECYCLING BARN
scale - 1" = 16'-0"
LOADING DOCK
EAST END
SRC
SHIPPING ROOM
STORAGE AND STAGING
LOFT ABOVE
HHW ROOM
36'
144'


POST ROAD
WEST END
stairwell
STUDIO
open below
open below
open below
FREIGHT ELEVATOR
144'
- TOWN LEASEHOLD AREA - 3160 ft²
SECOND FLOOR
BOWDOINHAM RECYCLING BARN
scale - 1" = 16'-0"
EAST END
open below
TOB
144'



- TOWN LEASEHOLD AREA - 3240 ft²

THIRD FLOOR
BOWDOINHAM RECYCLING BARN



ATTACHMENT C
OLDER ENGINEERING REPORTS

CRITERIUM ENGINEERS

Kathy Durgin-Leighton

From: Eric Calderwood [eric@calderwoodengineering.com]

Sent: Friday, February 20, 2009 1:07 PM

To: aberry@gwi.net

Cc: Kevin Prout; Kathy Durgin-Leighton

Details don't show the lally on the second floor, but we could either go with a 6x6 or a lally with a steel base plate lagged to the flooring, solid blocking will need to be provided between the floor above and the triple header below in those cases.

Also although we did not show it we could use a second header beam beneath the 1st and that will double our existing floor load rating. We would need to install a 2X6 at each post from the floor up to the bottom of the lower header to provide sufficient bearing for the second beam.

Adding either the post or the 2X10's bring us to the 30.1 psf allowable live load (my report recommends 24.1, but the code allows a 20% reduction in live load for members supporting more than one floor as is the case with the solid sawn 6x6 posts accounting for that gives us the 30.1)

Adding the second triple 2X8 beam beneath gives us a new capacity of 23 psf, or about double what we have now in those decked over bays.

Adding the bolts in the truss diagonals (tension only) gives us 32.1 psf in that area over the trusses.

Eric T. Calderwood, PE
Calderwood Engineering ETC
222 River Road, Richmond, Me
Ph (207) 737-2007
fax (207) 737-2008
eric@calderwoodengineering.com
www.calderwoodengineering.com

Floor Load Options

Bowdoinham Recycling Center

In the Town of

Bowdoinham, Maine

Prepared for:

The Town of Bowdoinham

By:

Calderwood Engineering etc

February 20th, 2009

STATE OF MAINE
ERIC T. CALDERWOOD
No. 9099
LICENSED
PROFESSIONAL ENGINEER

Cover photo: Turner & Leeds; courtesy MDOT & Reed&Reed

Calderwood Engineering Etc.
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222 River Road
Richmond, ME 04357
(207) 737-4501

SHEAR $F_v' = F_v = 135^{psi}$ OF 3 (2x8)

$f_v = \frac{3V}{2bd}$ $135^{psi} = \frac{3V}{2(7.25)(4.5)}$

$\boxed{V = 2936.25^{lb}/post}$

w/ $\omega = 413.456^{lb}/ft$

$413.456^{lb/ft} \cdot 12' = 4961.47^{lb}/2 \text{ post} = \underline{2480.74^{lb}/post}$

BENDING CONTROLS

Calderwood Engineering Etc.
www.calderwoodengineering.com
222 River Road
Richmond, ME 04357
(207) 737-4501

JOB: Bowman
SHEET NO.: 2 OF: 6
CALCULATED BY: O.K. DATE: 2/14/19
CHECKED BY: DATE:
SCALE:

SPF #2 South

$F_b = 775$ psi $\quad C_r = 1.15$

$F_t = 1000$

$F_v = 135$

$F_c = 1300$

$F_{c\perp} = 785$

C_F Size Factor

$\cancel{C_F = \left[\frac{12}{d}\right]^{1/9}}$ $\quad F_b = (775)(1.15)(1.2) = 1070$ psi

$C_F = \left[\frac{12}{5.5}\right]^{1/9} = 1.14$

$F_b = (775)(1.15)(1.14) = 1016$ psi

$M_{max} = S_x F_b = (1016\,\text{psi})(87.9\,\text{in}^3) = 89306.4$

$= 7442.2$ lb-ft

$wL = 413.456$ lb/ft

$\frac{wL}{14} = 29.53$ $= \boxed{29.53 \text{ psf}}$ $> (4 \times 6)$ 24" columns

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Richmond, ME 04357
(207) 737-4501

NAIL SPACING

$F_b' = (775)(1.14)(1.15) = 1016 \text{ psi}$

$S_x\ (2\times10) = 21.39 \text{ in}^3$

$M_x = (1016 \text{ psi})(21.39 \text{ in}^3) = 21732.2 \text{ lb-in}$

$= 1811.02 \text{ lb-ft}$

$M_x = \frac{w(l)^2}{8} \qquad w = 100.612 \text{ lb/ft}$

FOR 10d COMMON WIRE NAIL ...... 87 lb → nd

FOR 16d COMMON WIRE NAIL ...... 104 lb

d = 1.0 ...

SPACING $\frac{104 \text{ lb}}{100.612 \text{ lb/ft}} = 1.03367'$

OR NAIL EVERY 1'

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ADDING A 6x6 BUILT UP POST FOR (3) 2x8, MAKING $l_b = 6'$

$$W_a = \frac{(3515)8}{6^2} = 781 \text{ lb/ft}$$

$781 \text{ lb/ft} / 6 \text{ sf/lf}$

$= 130.2 \text{ lb/sf}$

-4.9

125.3 psf

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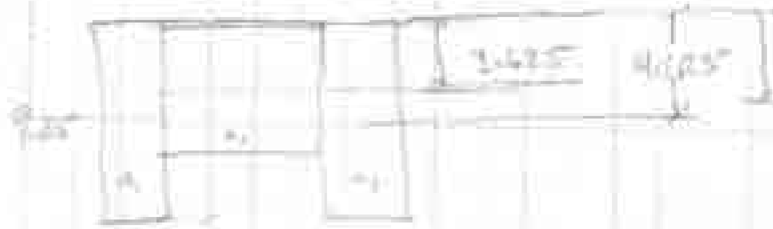
www.calderwoodengineering.com
222 River Road
Richmond, ME 04357
(207) 737-4501

JOB: [illegible]
SHEET NO. 1 OF 6
CALCULATED BY: O.K. DATE: 2/18/09
CHECKED BY: DATE:
SCALE:

$A_1, A_3 = 13.875$
$A_2 = 56.625$

(2) 2x10 and (3) 2x8



	A	y	Ay
A_1	13.875	4.625	64.1719
A_2	56.625	5.625	132.7666
A_3	13.875	4.625	64.1719

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{3601 in^3}{84.375} = 4.556$$

	I	A	d	$I+Ad^2$
A_1	98.93	13.875	.5109	103.422
A_2	1427	56.625	.451	1439.153
A_3	98.93	13.875	.529	103.422
				356.55 in^4

$$S = \frac{I}{c} = \frac{356.55 in^4}{4.056 in} = 87.9 in^3$$

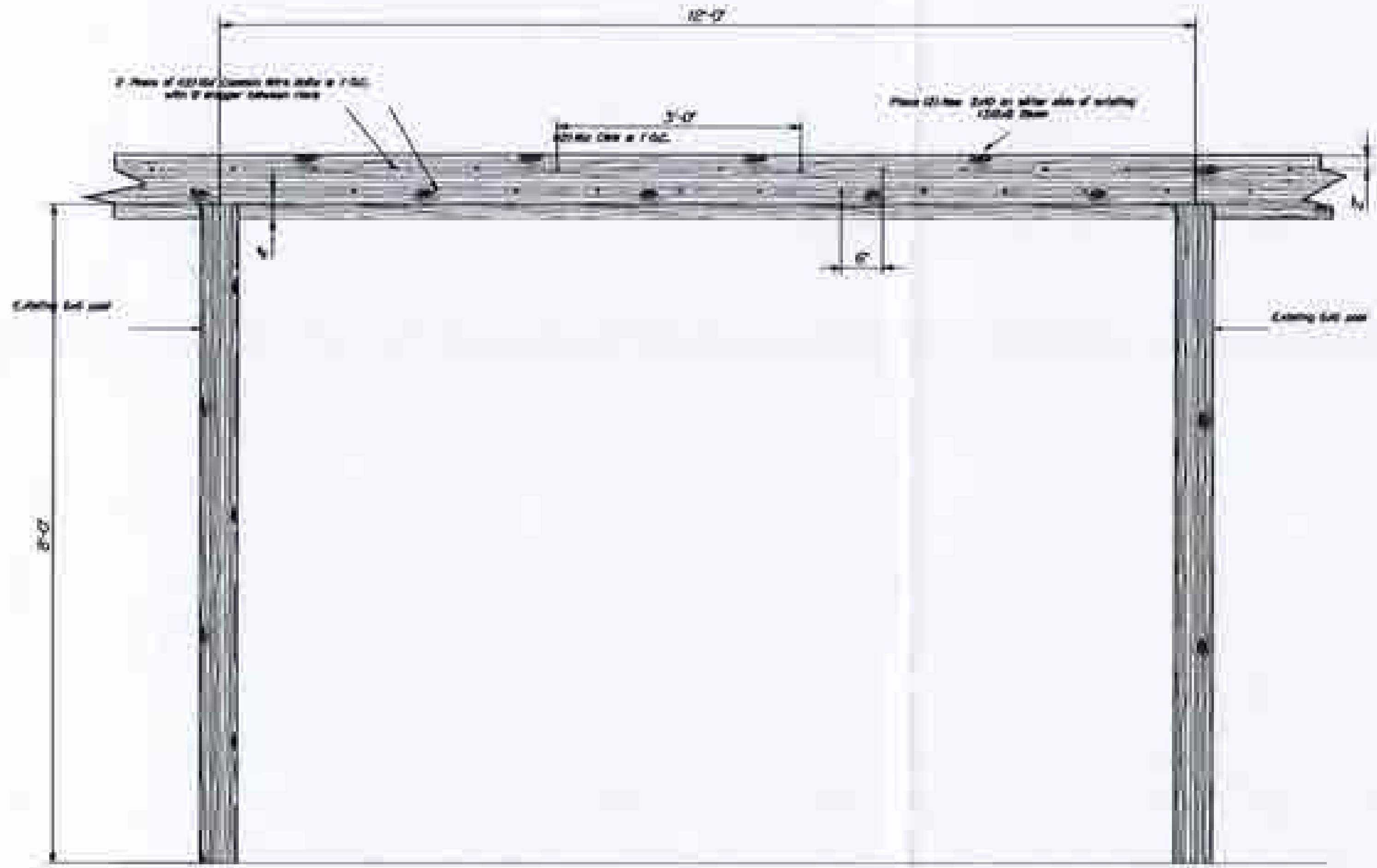
$M_{all} = (1070 psi)(87.9 in^3) = 94053$ lb-in

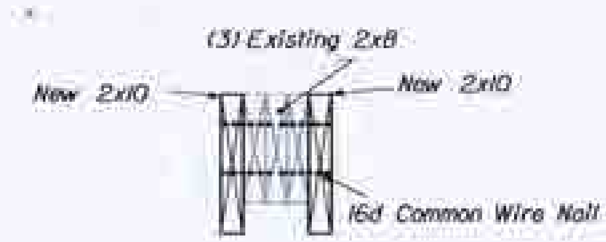
$= 7837.75$ ft-lb

$M = M_{all} = \frac{\omega L^2}{8}$ $\quad \omega = 435.4$ lb/ft

$\omega / 4 = 4?? = \boxed{36.36 \text{ psf}}$

NO





GENERAL NOTES

PROCEDURE

STATE OF MAINE
ERIC W. CALDERWOOD
No 9099
LICENSED PROFESSIONAL ENGINEER

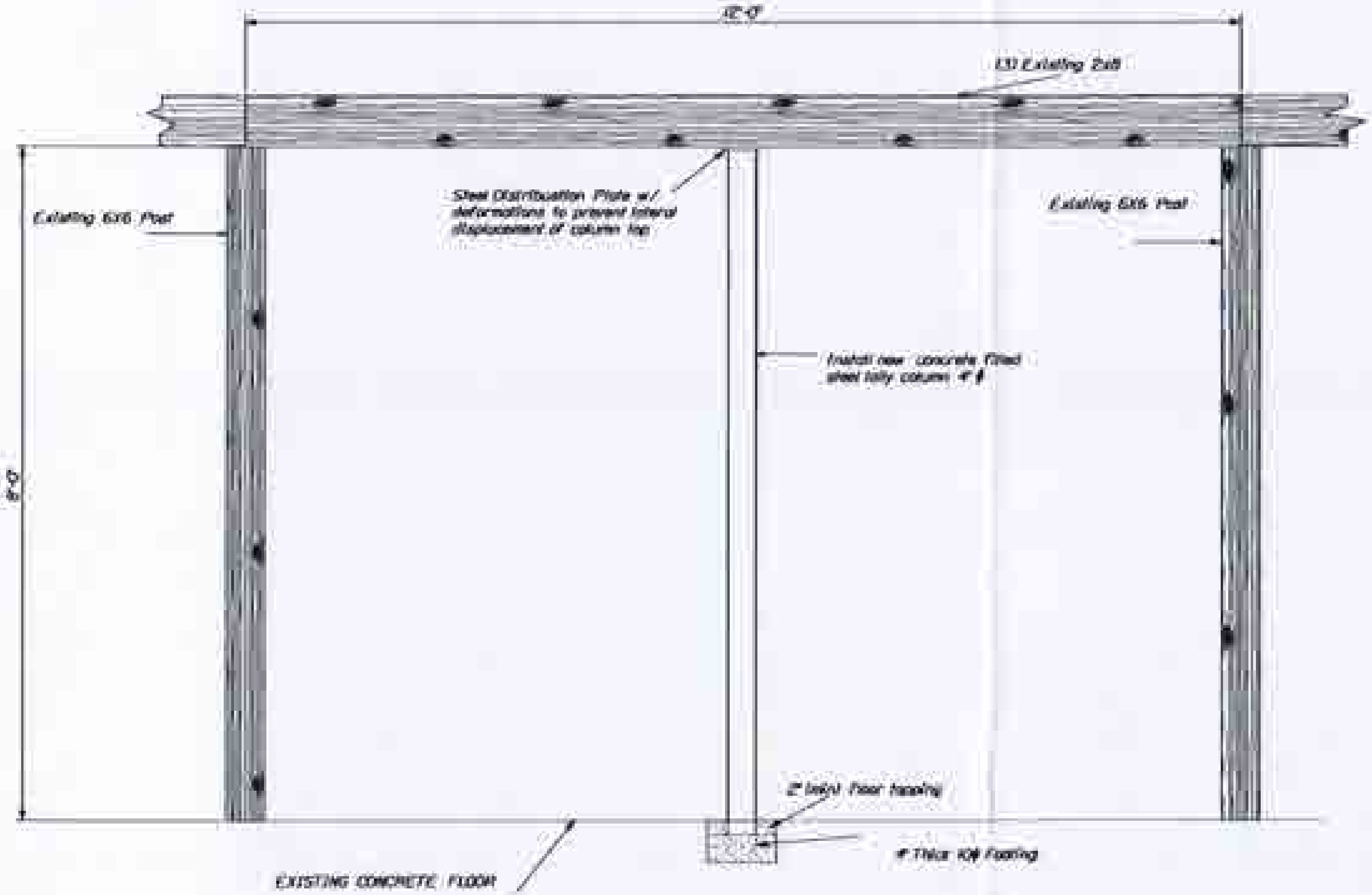
CALDERWOOD ENGINEERING, ETC.
STRUCTURAL ENGINEERING • DETAILING SERVICES

PREPARED FOR: TOWN OF BOWDOINHAM

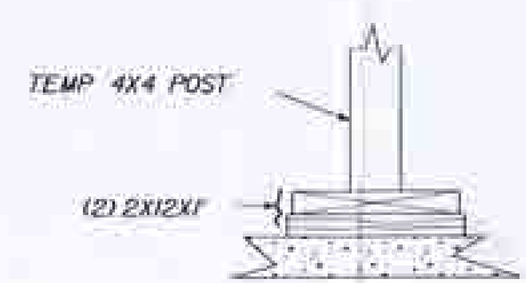
TOWN OF BOWDOINHAM
STRUCTURAL GIRDER MODIFICATIONS

BOWDOINHAM RECYCLING CENTER
REINFORCED GIRDER

SHEET NUMBER
F1



TEMP BRACING POST DETAIL



STATE OF MAINE
ERIC H. CALDERWOOD
No 9099
LICENSED PROFESSIONAL ENGINEER

CALDERWOOD ENGINEERING, ETC.

BOWDOINHAM RECYCLING CENTER

LALLY COLUMN

F2

ASSOCIATED DESIGN
PARTNERS INC.

Office: 207.878.1751
Fax: 207.878.1788
e-mail: adp@adpengineering.com
web: www.adpengineering.com

80 Leighton Road • Falmouth, Maine 04105

May 27, 2011

11145

Janet L. Smith
Interim Town Manager
Town of Bowdoinham
13 School St
Bowdoinham, ME 04008

RE: Structural Evaluation of roof framing at Bowdoinham Recycling Facility

Dear Kevin,

At your request, I have performed a limited evaluation of the existing roof framing of the Recycling Building located at 243 Post Rd in Bowdoinham, Maine. The purpose of my evaluation is to determine the approximate allowable uniform snow load capacity of the existing roof framing. Warren Gerow, PLS, engineering technician, visited the property on April 21st, 2011 and field measured a limited sample of the roof framing elements. ?

Building Description

The building at 243 Post Road is a wood framed, ~~2~~ 3-story agricultural building adapted for use as the Town of Bowdoinham Recycling facility and storage of miscellaneous items. The building footprint measures approximately 35ft wide by 290ft long (10,150 sf +/-).

all of the second line

It is our understanding that a portion of this building experienced a partial roof collapse during this past winter. Some of the roof has since been repaired and reinforced. An analysis of the reinforced area is included in this evaluation. ?

All 2x6 ?

The exterior load bearing walls consist of ~~2x4~~ 2x6 studs, with periodic lateral bracing and ploy sheet or metal panel cladding. The roof system consists of nominal 2x6 rafters spaced at approximately 38" o.c. with 1x4 top chord strapping spaced at 16" o.c. and a screw down corrugated metal roof. At the areas measured, the rafters are supported at mid-span with a built-up girder comprised of a double 2x8. The 2x8 girder span varies between 6'-4 and 12'-9 between (2) 2x8 diagonal post supports. The (2) 2x8 diagonal post supports are connected to a built-up (3 or 5, varies) built up 2x6 beam located below the ceiling joist that runs the length of the facility. Some of the (3) 2x6 beams have been sistered with (2) additional 2x6's, resulting in a (5) 2x6 beams. These beams are continuous and are supported by solid sawn timber posts spaced at approximately 11'-8" o.c. along the length of the building. ? 2x6 ? 2x5?

Analysis All on the S. side

This analysis is based on a limited sample of field measured structural elements. I performed an analysis of the measured rafters and beams utilizing StruCalc 8.0 structural analysis software. The dead load of the roof is estimated to be equal to or less than 7 psf for the rafters, 10 psf for the girts and beams. I applied the estimated Dead Load (weight of building elements), and a uniform snow load by trial and error until I determined the allowable snow load capacity. The following Table 3, lists the results of my analysis:

Table 3 – Theoretical Joist Capacities (summarized)

Member	Span (clr) ft	Spacing (o.c)	Est. Dead Load (psf)	Allowable Uniform Snow Load (psf)
(1) 2x6 Rafter	9ft +/-	38"	7 +/-	17 +/- Not Ok
(2) 2x6 Rafter	9ft +/-	38"	7 +/-	43 +/- Nearly Meets Code
(2) 2x8 Girt	6'-4" +/-	Na	10 +/-	46 +/- Nearly Meets Code
(2) 2x8 Girt	9'-6" +/-	Na	10 +/-	15 +/- Not Ok
(2) 2x8 Girt	12'-8" +/-	Na	10 +/-	0 +/- Not Ok
(3) 2x6 Beam	11'-8" +/-	Na	10 +/-	7 +/- Not Ok
(5) 2x6 Beam	11'-8" +/-	Na	10 +/-	19 +/- Not Ok

Today, the 2009 International Building Code (IBC) specifies that the minimum design uniform snow load for new buildings similar to the Recycling Barn in Bowdoinham, ME to be 46 psf (Pg=60 psf, Ct=1.2, Ps=46psf). However, the IBC 2009 also specifies that gable roofs must be designed for an unbalanced snow load. Unbalanced snow accumulation is described as snow blowing from the windward side of a pitched roof onto and accumulating on the leeward side. For the Recycling Barn, the code specified unbalanced snow accumulation is 60 psf for a distance of 9ft from the ridge.

Comparing the results of the allowable snow load analysis with the code specified design snow load requirements, it is clear that multiple framing elements do not meet the 2009 IBC code. In fact, much of the framing has an allowable snow load capacity less than half that required by code. ?

In order for this roof to remain in a safe and serviceable condition, I recommend structural reinforcement be designed and implemented. At a minimum, a qualified and insured third party roof maintenance contractor should be retained to provide roof snow load monitoring and removal services during the winter months. Snow monitoring and removal is not a long-term solution.

The findings as reported in this letter are based solely on my analysis of the limited field-measured structural elements. I reserve the right to change the findings and recommendations outlined herein and incorporate any new, previously un-foreseen or unknown information that may be discovered.

Sincerely,

Aaron S. Wilson, P.E.
Vice President
Associated Design Partners, Inc.

ASSOCIATED DESIGN
PARTNERS INC.

September 30, 2011

11145

Bill Post
Town Manager
Town of Bowdoinham
13 School St
Bowdoinham, ME 04008

RE: Supplemental Structural Evaluation of roof framing at Bowdoinham Recycling Facility

Dear Bill,

At your request, I have performed a limited supplemental evaluation of the existing roof framing of the Recycling Building located at 243 Post Rd in Bowdoinham, Maine. The purpose of this evaluation is to revise the approximate allowable uniform snow load capacity of the existing roof framing based on additional field measurements provided by the owner, and obtained during our site visit of 9/28/11. Also, a recommendation for the allowable depth of snow that the roof can support before manual snow removal is provided.

Building Description

The building at 243 Post Road is a wood framed, 2-story agricultural building adapted for use as the Town of Bowdoinham Recycling facility and storage of miscellaneous items. The building footprint measures approximately 35ft wide by 290ft long (10,150 sf +/-).

It is our understanding that a portion of this building experienced a partial roof collapse during this past winter. Some of the roof has since been repaired and reinforced. An analysis of the reinforced area is included in this evaluation.

See our original report dated 5/27/11 for further description of the building framing.

Today, the 2009 International Building Code (IBC) specifies that the minimum design uniform snow load for new buildings similar to the Recycling Barn in Bowdoinham, ME to be 46 psf (Pg=60 psf, Ct=1.2, Ps=46psf). However, the IBC 2009 also specifies that gable roofs must be designed for an unbalanced snow load. Unbalanced snow accumulation is described as snow blowing from the windward side of a pitched roof onto and accumulating on the leeward side. For the Recycling Barn, the code specified unbalanced snow accumulation is 60 psf for a distance of 9ft from the ridge.

The following Table 3 lists the results of my revised and expanded analysis:

Table 3 – Theoretical Member Snow Load Capacities (summarized)

Member	Span (clr) ft	Spacing (o.c)	Est. Dead Load (psf)	Allowable Uniform Snow Load (psf)
Upper (1) 2x6 Rafter	7.33ft +/-	36"	7 +/-	**32 +/-** Does Not Meet Code
Lower (1) 2x6 Rafter	10.66ft +/-	36"	7 +/-	**11+/-** Does Not Meet Code
Lower (1) 2x6 Rafter with diagonals	3.5ft and 6.5ft +/-	36"	7 +/-	**20 +/-** Does Not Meet Code
Lower (2) 2x6 Rafter at re-built area	10.66ft +/-	36"	7 +/-	**30+/-** Does Not Meet Code
Lower (2) 2x6 Rafter at re-built area with diagonals	3.5ft and 6.5ft +/-	36"	7 +/-	44+/- Nearly Meets Code
(2) 2x6 Girt	11'-9" +/-	N/A	10 +/-	**0 +/-** Does Not Meet Code
(2) 2x6 + (2) 2x8 Girt	11'-9" +/-	N/A	10 +/-	**18 +/-** Does Not Meet Code
(2) 2x8 Girt at re-built area	11'-9" +/-	N/A	10 +/-	**6 +/-** Does Not Meet Code
(3) 2x6 Beam	11'-9" +/-	N/A	10 +/-	**19 +/-** Not Ok
(5) 2x6 Beam	11'-9" +/-	N/A	10 +/-	**30 +/-** Not Ok

Comparing the results of the allowable snow load analysis with the code specified design snow load requirements, it is clear that multiple framing elements do not meet the 2009 IBC code. In fact, much of the framing has an allowable snow load capacity less than half that required by code.

<u>Three elements are of particular concern: The lower (1) 2x6 rafter (multiple) without diagonal bracing, the (2) 2x6 girts at the north side, the (2) 2x8 girts (new) at the south side re-built area.</u> These elements are theoretically overstressed when exposed to very minimal, or no snow loads. Visually these elements show signs of excessive permanent deformation, indicative of their relatively low structural capacity. I recommend these elements be sistered or reinforced to increase their snow load capacity prior to the 2011-2012 winter season. All reinforcing design should be completed by a Maine Licensed Engineer.

In order for this structure to remain in a safe and serviceable condition over the long term, I recommend comprehensive structural reinforcement be designed and implemented for the entire facility. At a minimum, a qualified and insured third party roof maintenance contractor should be retained to provide roof snow load monitoring and removal services during the winter months. <u>I recommend that when the depth of heavy, wet snow on the roof exceeds 6-8" that snow removal operations be implemented.</u> Snow monitoring and removal is <u>not</u> a long-term solution.

N

The findings as reported in this letter are based solely on my analysis of the limited field measured structural elements. I reserve the right to change the findings and recommendations outlined herein and incorporate any new, previously un-foreseen or unknown information that may be discovered.

Sincerely,

Aaron S. Wilson, P.E.
Vice President
Associated Design Partners, Inc.

Bowdoinham Recycling Center ~ Rehabilitation Summary

On August 21st 2013, Calderwood Engineering inspected the Bowdoinham Recycling center. The building is a converted (3) story chicken barn that has been modified several times throughout its service life. At first glance the 36' wide by 290' long building appears to be in relatively serviceable condition but upon further inspection there are many structural members that are visibly overstressed.

Recently the building has had a structural failure of the roof which since has been repaired. In 2011 Calderwood Engineering performed a structural analysis of the roof system and concluded that the existing members were undersized and needed to be reinforced. Two different rehabilitation options were designed and detailed for the rehabilitation of the roof. During the 2013 inspection only a fraction of roof members were found to be reinforced, and those that were reinforced were not strengthened to the level that was shown in Calderwood Engineering's details. These modifications, although better than leaving the roof as it was originally, are not enough to bring the roof structure up to code. The existing repairs should be added to in order to bring the roof up to code to prevent future structural failure.

Calderwood Engineering has calculated an unbalanced snow loading of 60psf (pound per square foot) on the roof using the 2012 International Building Code (IBC) and the Minimum Design Loads for Buildings 2010 (ASCE 7). This snow load is much larger than the capacity of the roof members. Calderwood Engineering has designed various options to strengthen the roof.

The following roof members were found to be undersized:

- 2x6 Rafter @ 3'-0" Upper Section Spanning 7'-0" +/-
- 2x6 Rafter @ 3'-0" Lower Section Spanning 11'-0" +/-
- (2) 2x8 Spanning 12'-0"

The recommended rehabilitation for the Upper section of the 2x6 rafter is to sister an additional 8'-0" +/- long 2x6 to the existing rafter.

There are (2) recommended rehabilitation options for the lower section of the 2x6 rafter. The first option is to add (1) 2x6 web member and (1) 2x4 web member to the lower section, converting this portion into a truss structure. These members would be fastened to the existing members with plywood or OSB gusset plates and nails. This option would also require (1) 2x4 to be sistered to the existing rafter. (Option 1)

The second option for the lower section of the rafters is to sister (2) additional 2x10 to the lower half of the existing 2x6 rafter. (Option 2)

There are also (2) rehab options for the double 2x8 "Roof Girders" that support the rafters and span 12'. The first option is to add (2) 2x4 kickers off of the supporting posts @ a 45° angle to reduce the 12' span to (3) 4' spans. These smaller spans reduce the stress in the existing members to an allowable level. (Option A)

The second option is to add (1) 11 7/8" x 1 3/4" Boise Cascade Versa-Lam Beam to the existing (2) 2x8. The existing posts carrying the 2x8's will need to be notched to allow the additional of the Versa-Lam Beam. (Option B)

Calderwood Engineering also analyzed the rest of the existing structure under dead, live and snow loads. The use of the building was determined to be a light storage warehouse on the account that the majority of the structure is housing various materials. According to ASCE 7 and the IBC for a light storage warehouse the design live load is 125 psf. The existing allowable live loads were calculated for the structural members, below are the results:

- Floor Joists (2x8 12' Span @ 2' Spacing) ~ 27.4psf
- Triple (2x8 Beams Supporting (1) Span (Center Bay Undecked) ~ 20.6psf
- Triple (2x8 Beams Supporting (2) Span (Center Bay decked) ~ 10.3 psf
- 6x6 Built-up Column (2nd Floor Center) ~ 61.1 psf
- 6x6 Built-up Column (1st Floor Center) ~ 56.1psf

From the above results it is evident that the existing structure is not adequate to support a design live load of 125psf. In order to provide the best solution for the strengthening of these members a number of rehabilitation options have been designed.

The option designed for the strengthening of the existing floor joists is to sister each floor joist with a 5 1/2" x 3 1/2" Boise Cascade Versa-Lam Beam. A secondary option was explored but when looking into the cost for this option is was removed because the cost was extremely high.

There are (2) rehab options for the triple 2x8 beams. The first option is to add (2) 5x5 kickers @ 45° angle to each existing posts to reduce the existing 12' span to (3) 4' spans. This would reduce the stresses in the members to an allowable level. This option is relatively low in cost but it would reduce the available area to move and store materials, which is not ideal. (Option 1)

The second option is the addition of (2) C6x8.2 channels (1) on either side of the existing members. The channels would be connected by 1/2" diameter A325 bolts spaced at 2' centers. The bolts would be through bolted in holes drilled through the channels and the existing 2x8's. This option is more expensive but does not reduce the usable area in the building.(Option 2) Triple 2x8 shall be jacked up 1/4" at center span or until visibly level prior to the addition of the 5x5 kickers or C6x8.2 channels.

The existing columns were found to be built up columns made up of (3) 2x6 or to be solid sawn 6x6. The column rehab was designed separately for the 1st and 2nd floors. The columns on the 3rd floor supporting the roof were found to be adequate and no further modifications are required.

There are no modifications required for the 2nd floor columns if the rehab option using the 5x5 kickers is used. If the kicker option is not included, an additional 2x6 is required to be added to the existing post. The first floor columns are undersized with or without the addition of the kickers and need (2) 2x6 added to the existing column to carry the design loads. These 2x6's should be added (1) on each side of the existing column such that the final section would be (5) 2x6 lined up face to face.

The exterior walls were found to be made up of 2x4 @ 2' centers. For the design loads, these were found to be adequate to resist the vertical loads as long as they are braced laterally by sheathing. During inspection one entire wall did not have any sheathing connecting the 2x4's together. There were few lateral bracing members installed but these members do not brace the 2x4's enough to develop the capacity needed. In order for the existing walls to be able to carry the design loads 1/2" plywood/OSB should be added to all exterior walls that are not covered by plywood or planking.

During inspection it was found that in the rear of the building there was a portion where the first floor is extended up to the floor joists of the 3rd floor. This sections is approximately 16' high and has (2) large trusses that span 24' each. The trusses have been modified with the addition of 1" diameter bolts to the connection of the tension diagonal web members. Calderwood Engineering performed an analysis of the existing truss and found that not all the existing members were adequate to carry the design loads. The following truss members were found to be undersized:

- Bottom Chord - (3) 2x10
- Top Chord - (3) 2x10
- Tension Diagonal - (2) 2x8
- Compression Diagonal - (2) 2x8

The recommended option for the rehabilitation of the bottom chord is to add (2) 1/4" thick plates to the middle 14' +/- of the bottom chord. It was determined that the tension force in the bottom chord in the middle portion could be carried by adding (3) 2x10 to the existing (3) 2x10, but this did not seem feasible so the addition of a steel plate was inspected. The new steel plate should be connected with the existing 1" diameter through bolts located in the bottom chord. The minimum required width of the plate was found to be 4 5/8", but it is very likely that the location of the existing bolt holes will control the width.

The recommendation for the top chord is to add (2) 2x4 lateral braces at 6' centers. Each brace should be connected to the top chord and extend at a 45° +/- angle to the existing floor joists. There would be (1) brace required on each side of the top chord every 6'. This option is recommended because it requires the least amount of work and material added to the top chord.

The recommendation for the tension diagonals is to replace the (2) 2x8 members with (2) 1/4" thick steel plates. Wood was not an option for the tension diagonal because it would require (5) additional 2x8 which did not seem feasible. The 1/4" would be connected to the bottom chord with the same bolts as the bottom chord steel plates and would be connected to the top chord with the existing through bolts. Since the capacity of the bolts in the wood in the top chord is not adequate to develop the tension in the diagonal members, the installation of (2) C6x8.2 channels as described in the repair of the triple 2x8's above would be required. These channels should extend (1) span on each side of the truss and would be enough to strengthen those spans. With the addition of the channels the bolt capacity is large enough to develop the tension required for the diagonal members.

The recommended solution for the compression diagonals is to add (2) 2x10 or (1) 6x6 solid sawn post to the diagonals. These members would be cut to be wedged tight between the existing vertical and horizontal members. Also (2) 1" diameter A325 bolts should also be installed through the existing 2x8 compression members and bottom/top chord in order to develop the compression members. Both the bolts and the direct compression of the additional members would be required to develop the compression force required.

For the rehabilitation of the truss, the web diagonal members will be required to be removed during construction. In order to perform this work temporary supports must be installed to ensure the truss remains in place. It is our recommendation to use (2) 6x6 solid sawn timber posts at a 45° angle off of the existing built up columns. The top of the temporary supports would be located below the location where the web diagonals meet approximately 6' off of the columns. These temporary supports would be required to remain in place until all the truss modifications are completed.

The existing columns in the rear section of the building that extend 16' +/- to the floor joists of the 3rd floor are built up 12x12 columns. After analysis it was found that these columns are adequate to carry the design loads, therefore no modifications are required for the 12x12 columns.

Once all of the above solutions were designed, Calderwood Engineering had a 3rd party perform a cost estimate for each of the options. The following are the estimates for each option.

Roof Truss Rehab Option 1-A (Sheet 2) $ 28,000.00
(Roof Truss Members & 2x4 Kickers Added)

Roof Truss Rehab Option 1-B (Sheet 2) $ 39,500.00
(Roof Truss Members & 11 7/8" x 1 /34" Versa-Lam Added)

Roof Truss Rehab Option 2-A (Sheet 2) $ 31,600.00
(2) 2x10's, (1) 2x6 & 2x4 Kickers Added)

Roof Truss Rehab Option 2-B (Sheet 2) $ 43,000.00
(2) 2x10's, (1) 2x6 & 11 7/8" x 1 /34" Versa-Lam Added)

F**loor Rehab Option 1 (Sheet 3)** $ 63,000.00
(Addition of Versa-Lam to each floor Joist & 5x5 Kickers to each Column)

F**loor Rehab Option 1 (Sheet 3)** $115,000.00
(Addition of Versa-Lam to each floor Joist & (2) C6x8.2 to Carrying Beams)

Truss rehab (Sheet 4 & 5) $ 7,200.00

Exterior sheathing (no finishes) $ 7,000.00

The least expensive of the options is the roof option 1-A and floor option 1. The estimator also advised that an additional $10,000 be added to the overall estimate for miscellaneous unseen items. This brings the least expensive project total budget to $115,200.

If head room is a concern in certain spans where the addition of the 5x5 kickers would not able to be used to maintain room for current needs, floor rehab option 2 may be implemented. For these locations an additional $650.00 per span should be added to the total.

In conclusion, the existing structure has not been designed or built to carry the anticipated design loads for the use of the structure. Many members are currently overstressed and if nothing is added to strengthen the structure or the building use is changed the structure is in danger of failure. Multiple rehabilitation options were explored and the most cost efficient options have been described above to reinforce the existing structure in order to prevent structural failure. With the cheapest options chosen for the structure the estimated cost of the project is $115,200.

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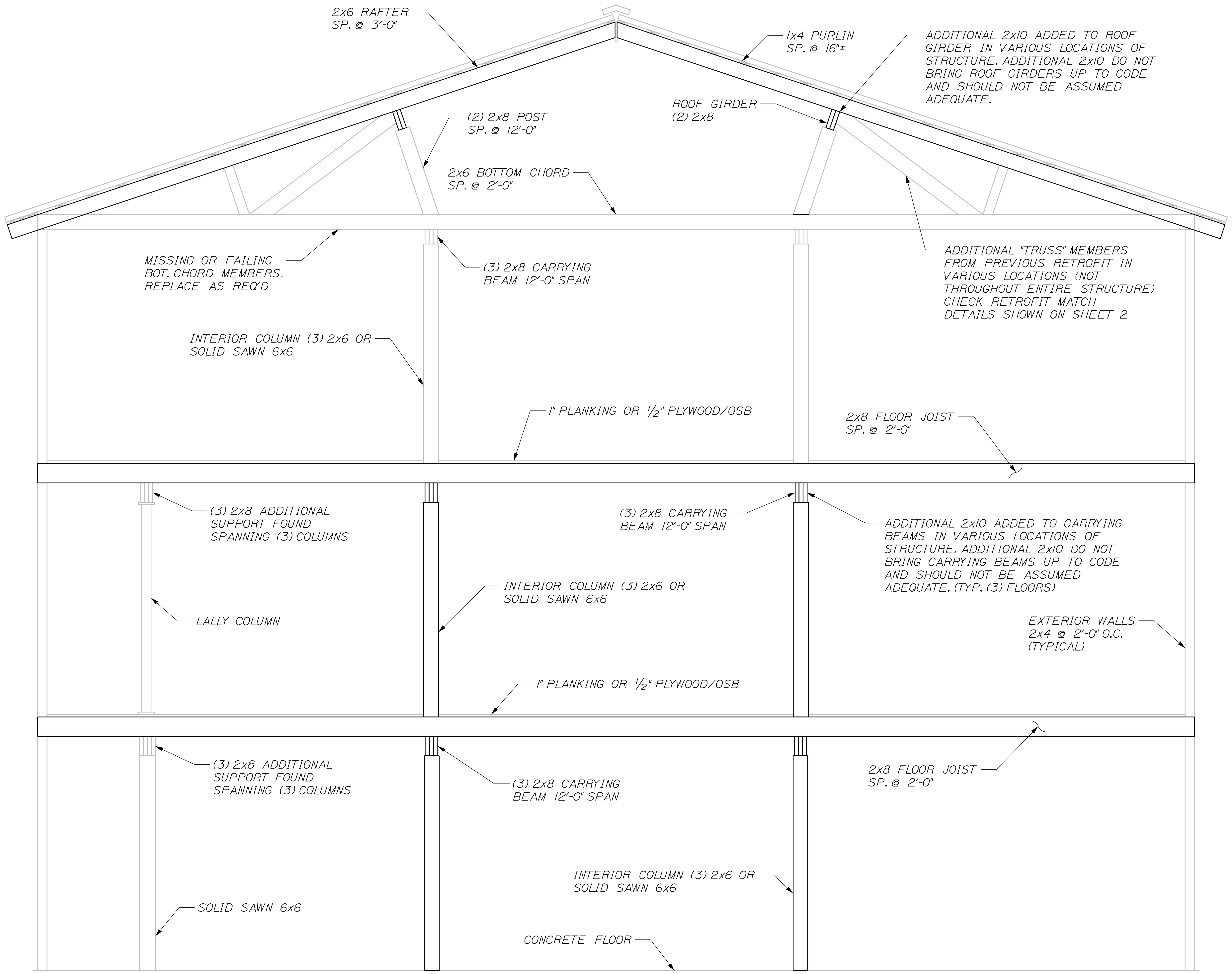
Username: common
Date:1/27/2014



TYPICAL SECTION ~ LOOKING AWAY FROM THE ROAD
1/2" = 1'-0"
NOTE: MEMBERS IN BOLD ARE UNDERSIZED AND NEED RETROFIT TO BE ABLE TO SUPPORT DESIGN LOADS

GENERAL NOTES

1. LAMINATED VENEER LUMBER SHALL HAVE A MINIMUM MODULUS OF ELASTICITY OF E=2,000,000 PSI AND A MINIMUM F =3100 PSI
2. ALL SAWN DIMENSION LUMBER IS NOMINAL SIZE, SPF #2 GRADE
3. ALL STEEL SHALL BE IN NEW OR GOOD USED CONDITION
4. ALL STEEL TO MEET THE REQUIREMENTS OF ASTM A36 OR BETTER
5. ALL BOLTS SHALL BE A325 BOLTS
6. ALL MATERIALS STORED ON THE PROJECT SHALL BE PROTECTED FROM THE ELEMENTS BY BEING STORED INDOORS ABOVE GROUND LEVEL ON SUITABLE DUNNAGE.
7. ALL DIMENSIONS WITHIN THESE DETAILS ARE APPROXIMATE. THE CONTRACTOR IS RESPONSIBLE TO DETERMINE THE EXACT DIMENSIONS IN THE FIELD.
8. ANY CONFLICT BETWEEN THESE PLANS AND EXISTING CONDITIONS SHOULD BE ADDRESSED BY THE ENGINEER OF RECORD. DO NOT PROCEED WITH DEPENDENT WORK UNTIL ANY CONFLICT HAS BEEN ADDRESSED BY THE ENGINEER OF RECORD.
9. ALL MATERIALS REQUIRED TO BE REMOVED AND REUSED, SHALL BE MAINTAINED IN SERVICEABLE CONDITION, STORED IN A MOISTURE FREE ENVIRONMENT INDOORS, AND ABOVE THE GROUND LEVEL.
10. ALL MODIFICATION WORK MUST BE DONE WITH ONLY STRUCTURAL DEAD LOAD. DO NOT PERFORM THIS WORK WHEN THE STRUCTURE IS SUBJECT TO SNOW OR LIVE LOAD CONDITIONS.
11. THESE DESIGNS ARE BASED ON TYPICAL SECTIONS OF THE WHOLE OF THE BUILDING. THERE ARE LOCATIONS WHERE THESE DESIGNS WILL NEED TO BE MODIFIED ACCORDING TO THE EXISTING SECTION. THOSE LOCATIONS WILL BE ADDRESSED ON A CASE BY CASE BASIS.

DESIGN LOADS

DESIGN LOADS PER INTERNATIONAL BUILDING CODE 2012 IN CONJUNCTION WITH MINIMUM DESIGN LOADS FOR BUILDINGS 2010

GROUND SNOW LOAD = 60psf
UNBALANCED ROOF SNOW LOAD = 60psf
LIVE LOAD = 125psf (LIGHT STORAGE)

EXISTING STRUCTURAL MEMBER	EXISTING ALLOWABLE LIVE LOAD
FLOOR JOIST ~ 2x8 @ 2'-0"	27.4psf
CARRYING BEAM ~ (3) 2x8 12' SPAN	10.3psf
2ND FLOOR COLUMNS ~ (3) 2x6 12' SPACING	61.1psf
1ST FLOOR COLUMNS ~ (3) 2x6 12' SPACING	56.1psf

CALDERWOOD ENGINEERING
222 RIVER ROAD
RICHMOND, ME 04357

CALDERWOOD ENGINEERING, ETC.
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222 RIVER RD, RICHMOND, ME 04357 PH/FX (207) 737-2007/(207) 737-2008

PREPARED FOR: **TOWN OF BOWDOINHAM**

CEE 075-ST-13

STATE OF MAINE
ERIC J. CALDERWOOD
No 9099
LICENSED
PROFESSIONAL ENGINEER

P.E. NUMBER
DATE

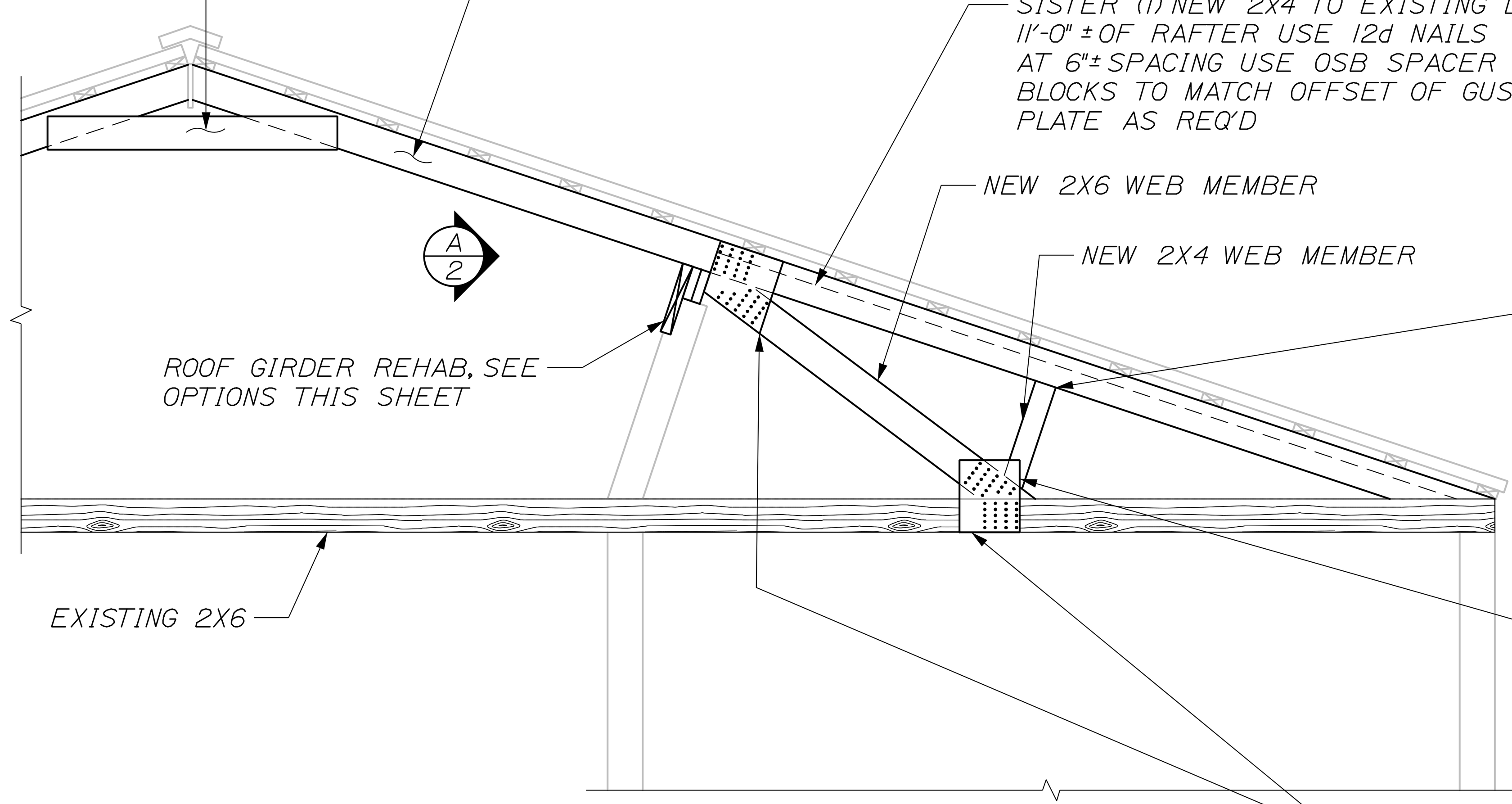
	BY	DATE
DESIGN-DETAILED	GNM	DEC. 2013
CHECKED-REVIEWED	GNM	
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

BOWDOINHAM RECYCLING CENTER

RECYCLING CENTER TYPICAL SECTION

SHEET NUMBER
1

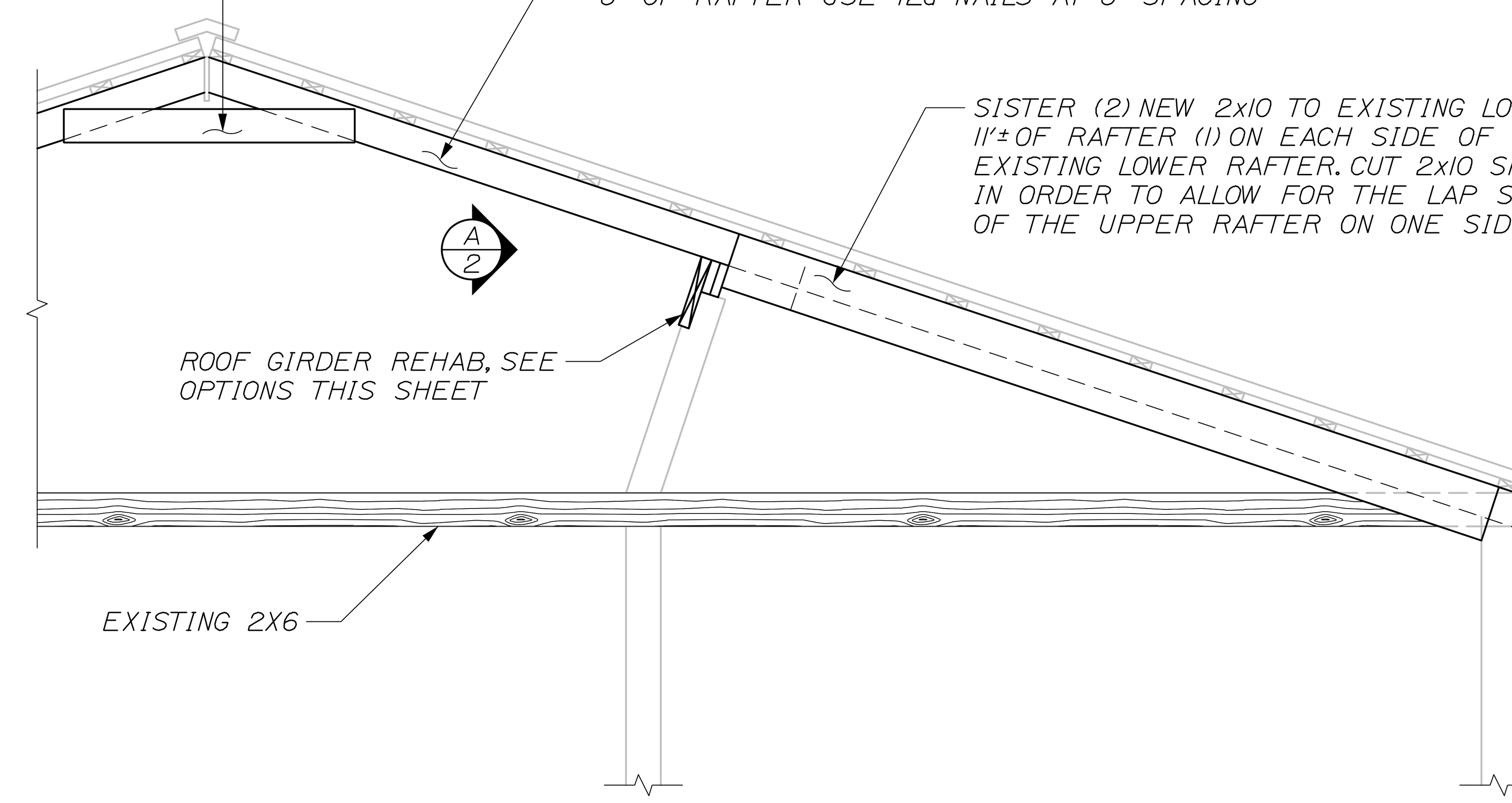
Filename: ...\002_Recycling Roof Rehab Workgroup: Structures Division Username: common Date:1/27/2014



ROOF REHAB OPTION #1

1/2" = 1'-0"

NOTE: OPTION A & B ARE BOTH ACCEPTABLE REPAIRS FOR THE EXISTING FOR GIRDER



ROOF REHAB OPTION #2

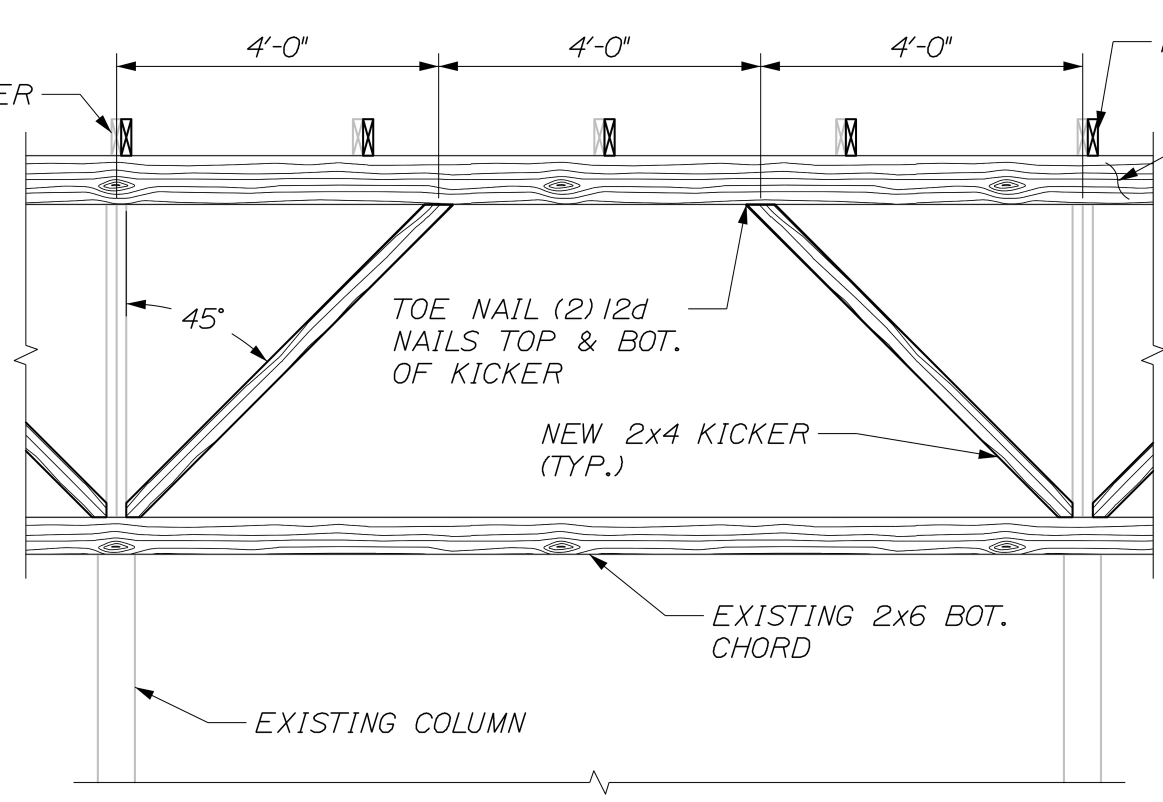
1/2" = 1'-0"

NOTE: OPTION A & B ARE BOTH ACCEPTABLE REPAIRS FOR THE EXISTING FOR GIRDER

GENERAL NOTES

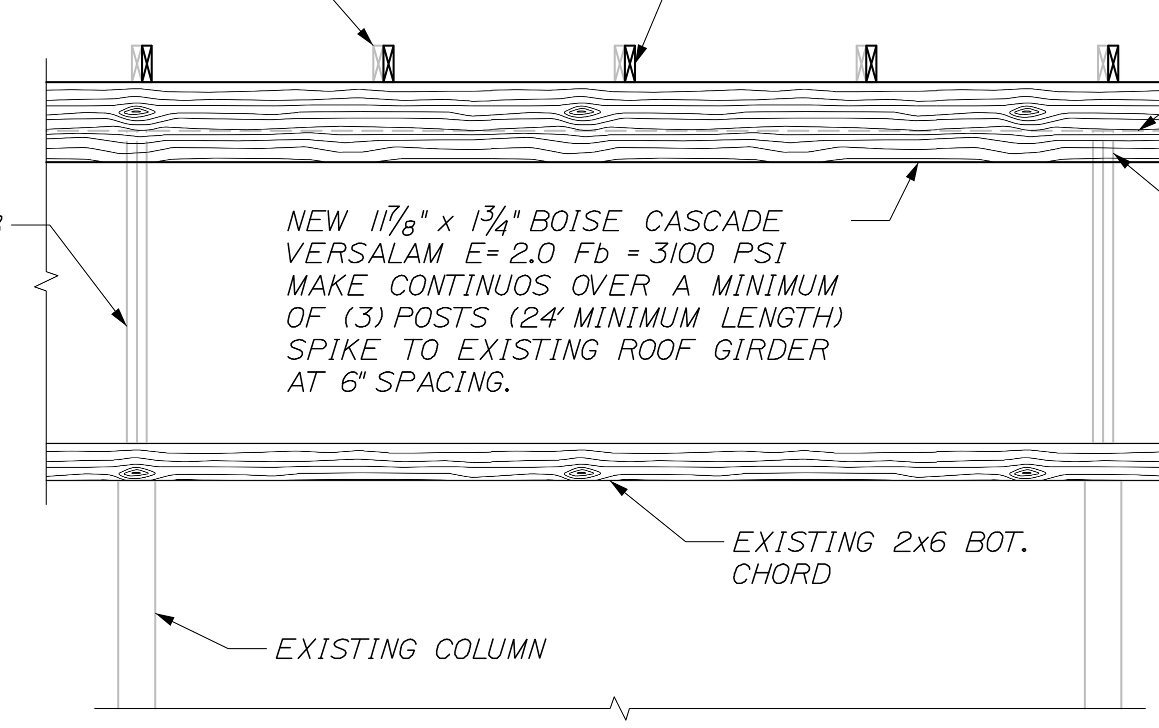
1. ALL NEW LUMBER SHALL BE SPF #2 OR BETTER

2. ALL OSB SHALL BE STRUCTURAL I



A/2 ROOF GIRDER REHAB OPTION A

1/2" = 1'-0"



A/2 ROOF GIRDER REHAB OPTION B

1/2" = 1'-0"

CALDERWOOD ENGINEERING, ETC.
STRUCTURAL ENGINEERING • DETAILING SERVICES
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PREPARED FOR:
TOWN OF BOWDOINHAM

CEE 075-ST-13

STATE OF MAINE ERIC A. CALDERWOOD No 9099 LICENSED PROFESSIONAL ENGINEER

P.E. NUMBER

DATE

	BY	DATE
DESIGN-DETAILED	GNM	DEC. 2013
CHECKED-REVIEWED	GNM	
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

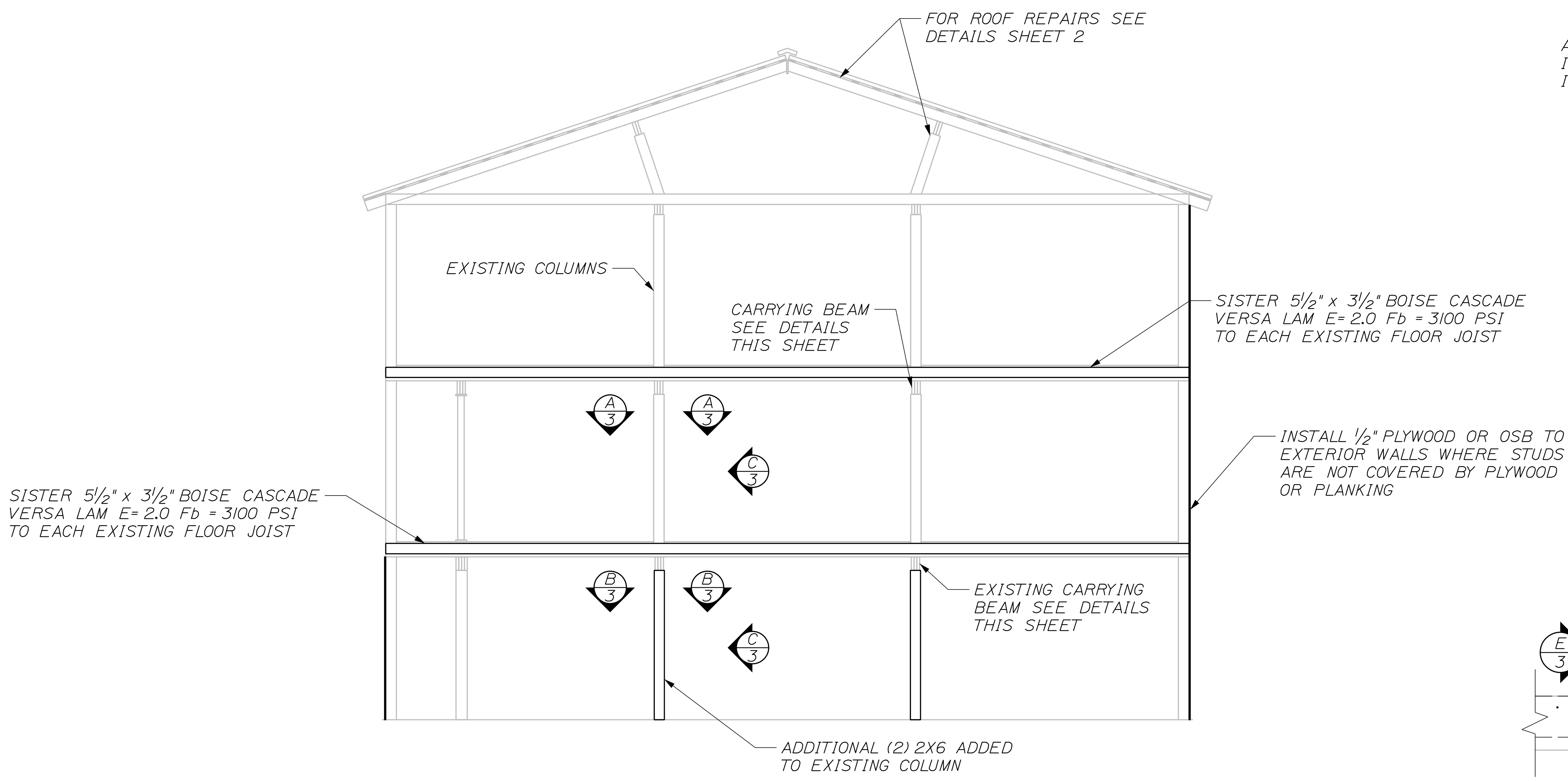
BOWDOINHAM RECYCLING CENTER

ROOF REHAB OPTIONS

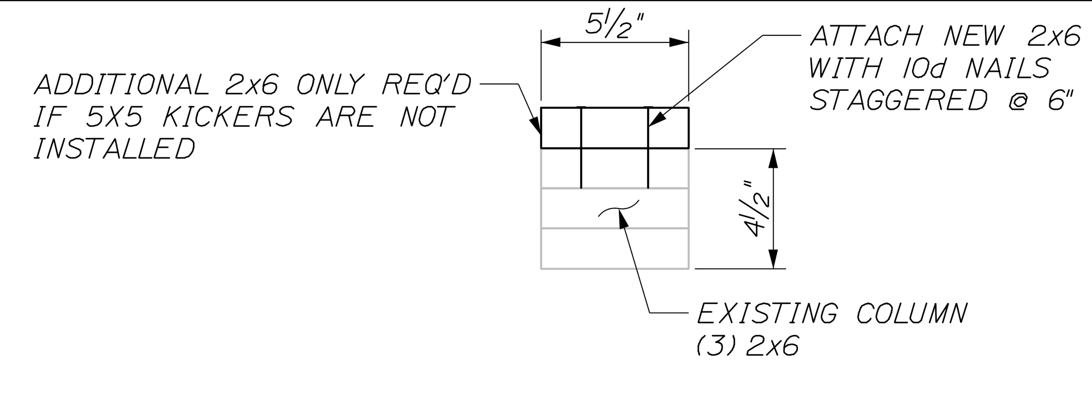
SHEET NUMBER

2

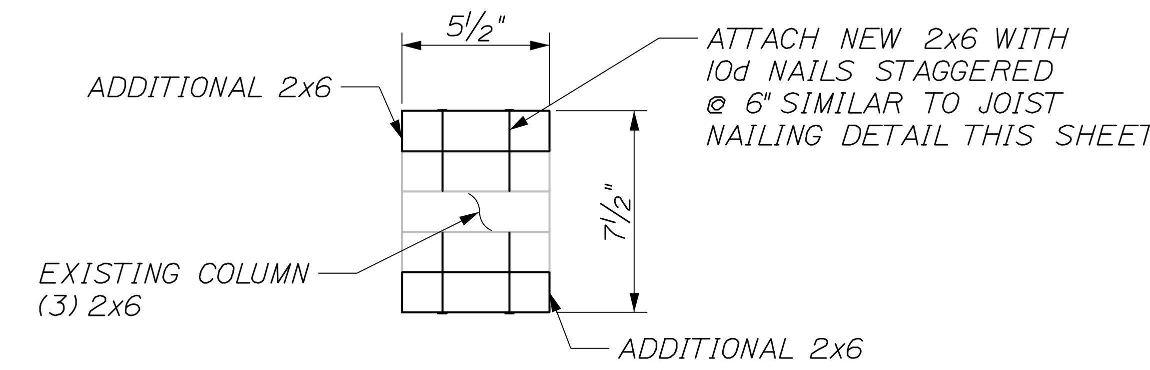
CALDERWOOD ENGINEERING 222 RIVER ROAD RICHMOND, ME 04357



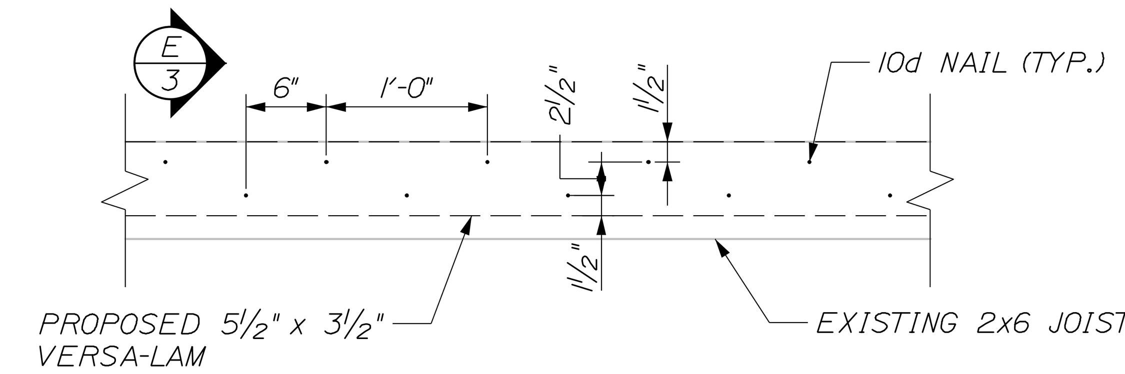
TYPICAL REPAIR SECTION
¼" = 1'-0"



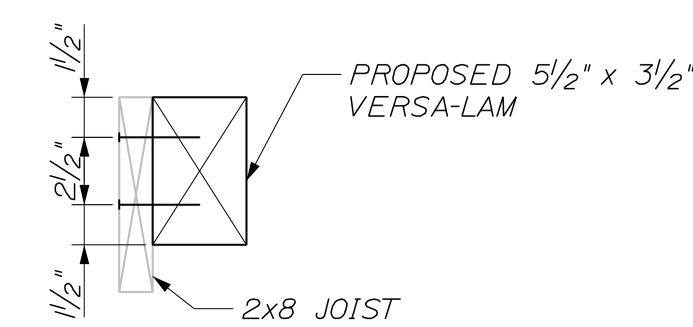
A/3 SECOND FLOOR COLUMN SECTION
2" = 1'-0"



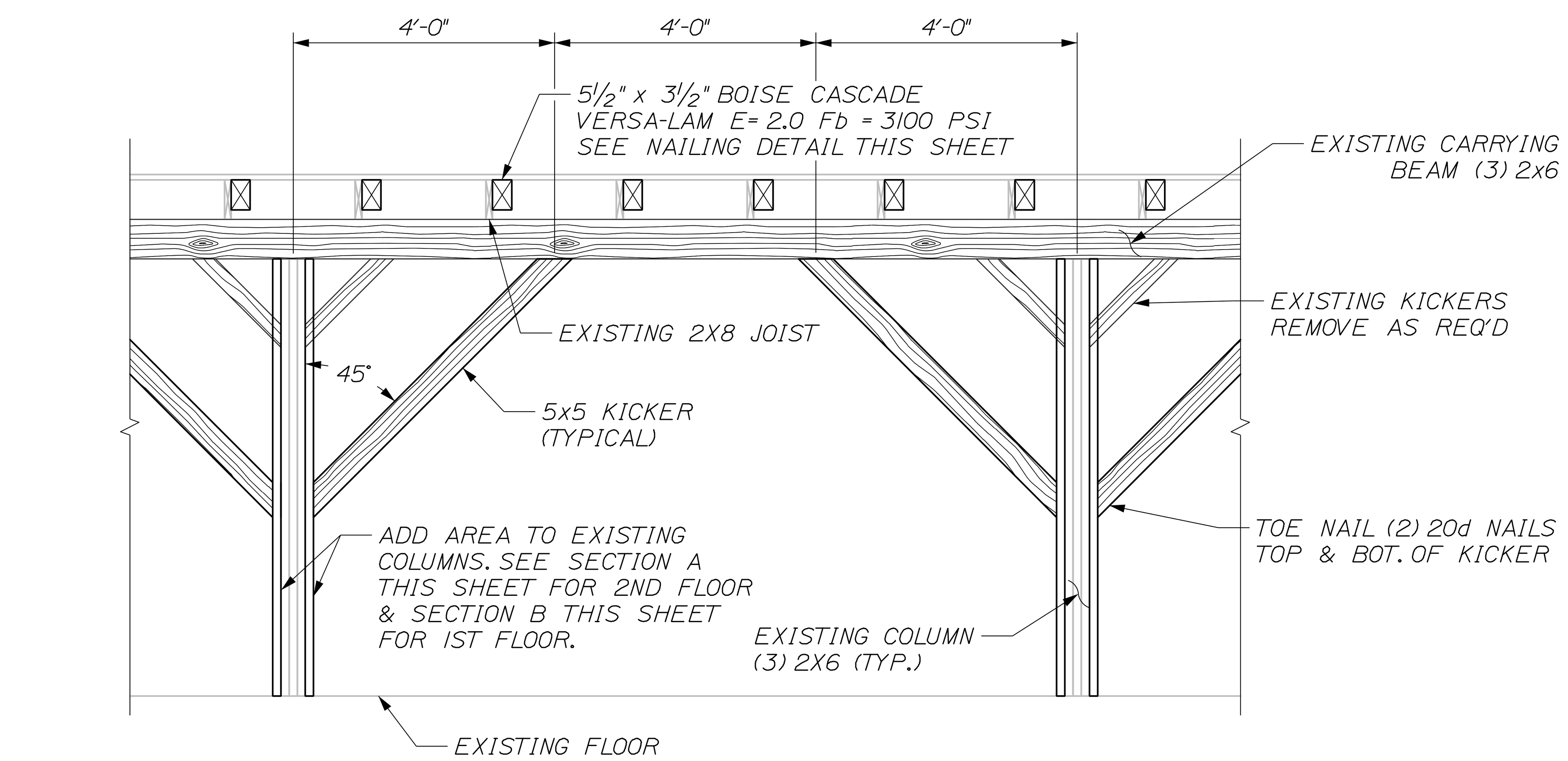
B/3 FIRST FLOOR COLUMN SECTION
2" = 1'-0"



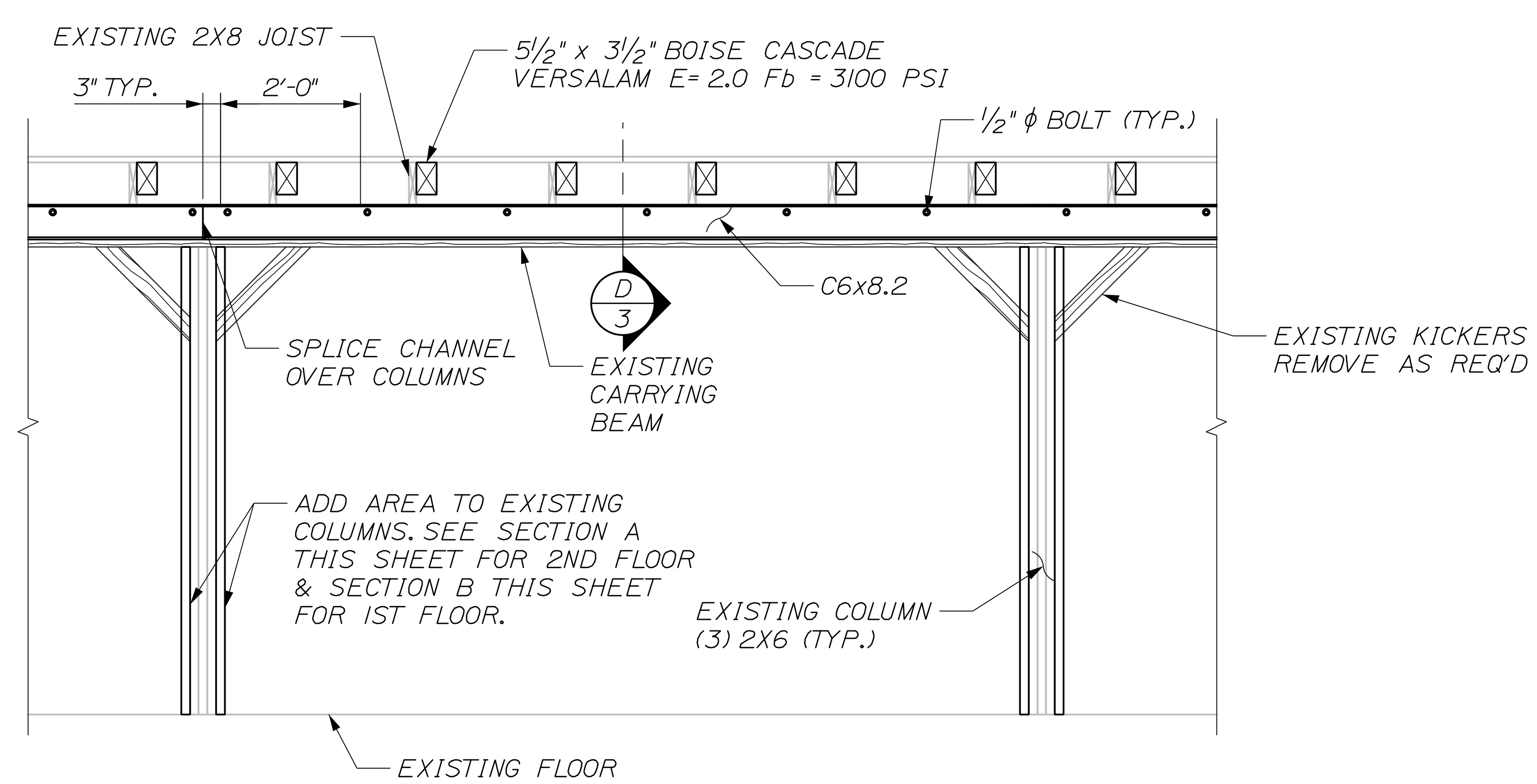
TYPICAL FLOOR JOIST NAIL SPACING
1" = 1'-0"



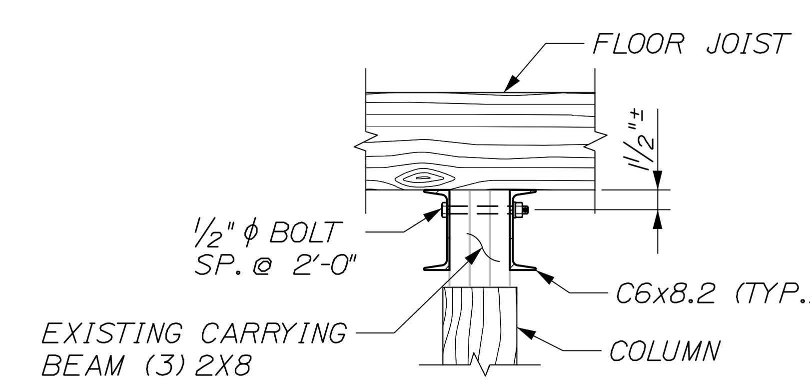
E/3 JOIST SECTION VIEW
2" = 1'-0"



C/3 1ST & 2ND FLOOR CARRYING BEAM REHAB OPTION 1
½" = 1'-0"



C/3 1ST & 2ND FLOOR CARRYING BEAM REHAB OPTION 2
½" = 1'-0"



D/3 CARRYING BEAM SECTION
1" = 1'-0"

REHABILITATION OPTION 1 PROCEDURE

1. RELOCATE ALL EXISTING WIRING PLACED ALONG EXISTING (3) 2x8 CARRYING BEAM

2. JACK UP (3) 2x8 CARRYING BEAM ¼" OR UNTIL VISUALLY LEVEL. USE 4x4 POST SET ON (2) 2x12x1'-0" SET ON (2) LAYERS WITH GRAIN RUNNING PERPENDICULAR BETWEEN THEM.

3. INSTALL ADDITIONAL SECTION TO EXISTING COLUMNS AS SHOWN IN SECTIONS A AND B THIS SHEET.

4. INSTALL 5x5 KICKERS TIGHT BETWEEN CARRYING BEAM AND COLUMNS AS SHOWN

5. REMOVE 4x4 TEMPORARY POST

REHABILITATION OPTION 2 PROCEDURE

1. RELOCATE ALL EXISTING WIRING PLACED ALONG EXISTING (3) 2x8 CARRYING BEAM

2. JACK UP (3) 2x8 CARRYING BEAM ¼" OR UNTIL VISUALLY LEVEL. USE 4x4 POST SET ON (2) 2x12x1'-0" SET ON (2) LAYERS WITH GRAIN RUNNING PERPENDICULAR BETWEEN THEM. JACK UP CONSECUTIVE SPANS AS REQ'D TO MATCH THE LENGTH OF C6x8.2 CHANNELS. SOLIDLY BLOCK TEMP POST.

3. DRILL 9/16" HOLES @ 2'-0" CENTERS ALONG EXSITING CARRYING BEAM TO MATCH HOLES IN NEW C6x8.2 CHANNELS.

4. INSTALL (2) C6x8.2 CHANNELS AS SHOWN WITH ½" ϕ BOLTS 2'-0".

5. REMOVE 4x4 TEMPORARY POST

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CEE 075-ST-13

STATE OF MAINE
ERIC T. CALDERWOOD
No. 9099
LICENSED
PROFESSIONAL ENGINEER

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BY: GNM	GNM					
DATE: DEC. 2013						

BOWDOINHAM RECYCLING CENTER

FLOOR REHAB OPTION #1

SHEET NUMBER
3

CALDERWOOD ENGINEERING
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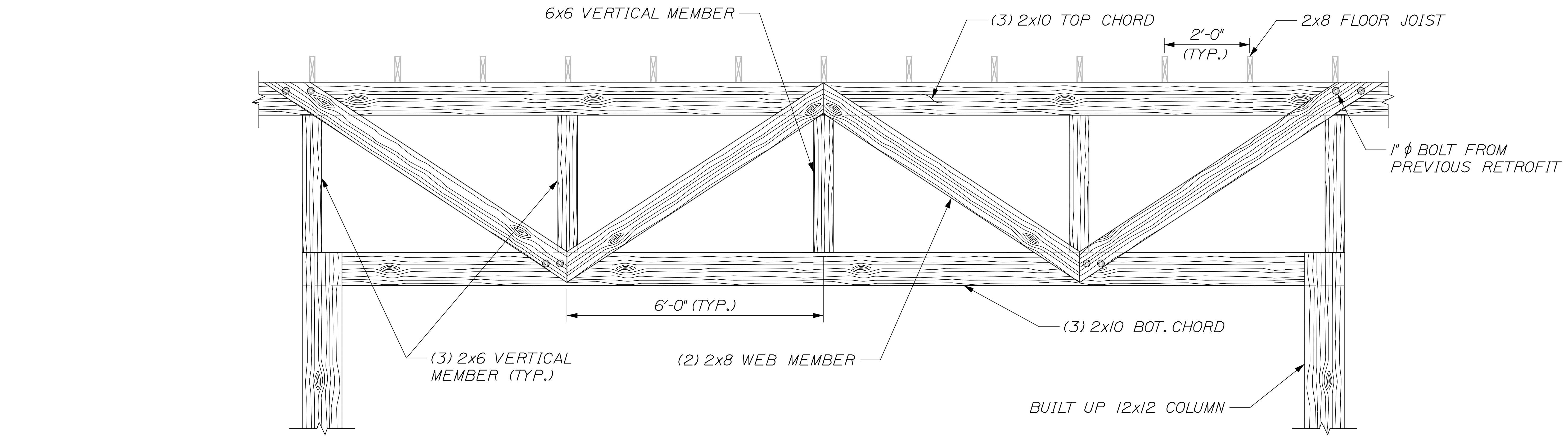
Filename: ...\003_Floor Rehab Option.dwg   Workgroup: Structures Division   Username: common   Date:1/27/2014

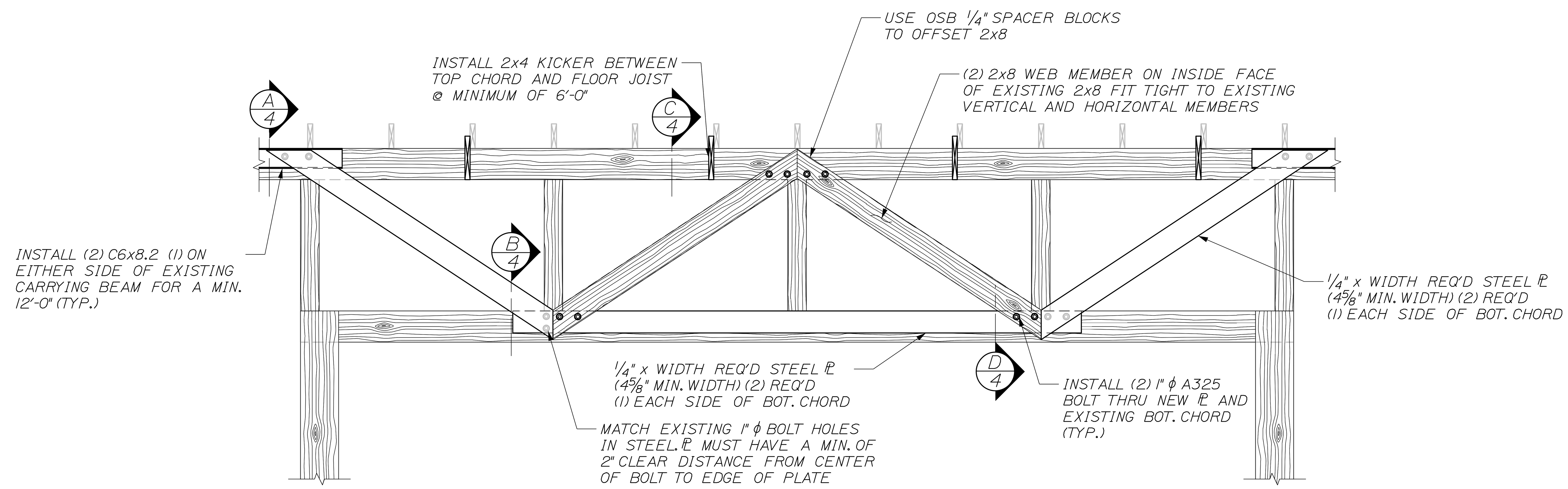
Filename: ...\004_Truss Rehab.dgn Workgroup: Structures Division Username: common Date:1/27/2014



EXISTING CEILING TRUSS

1/2" = 1'-0"

NOTE: EXISTING CEILING TRUSS FOUND IN REAR OF BUILDING

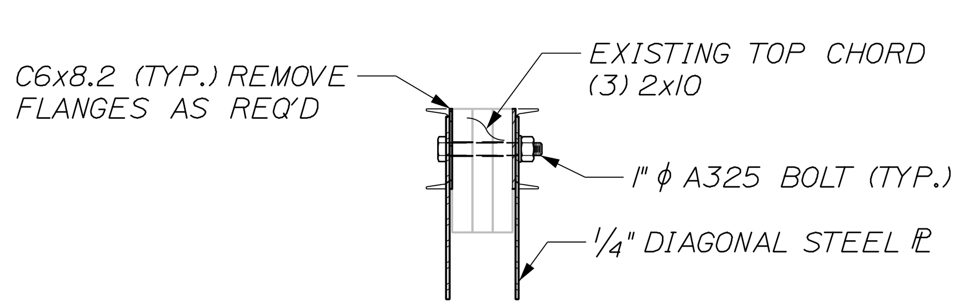


SEE SHEET 7 FOR CONSTRUCTION SEQUENCE

CEILING TRUSS REHAB

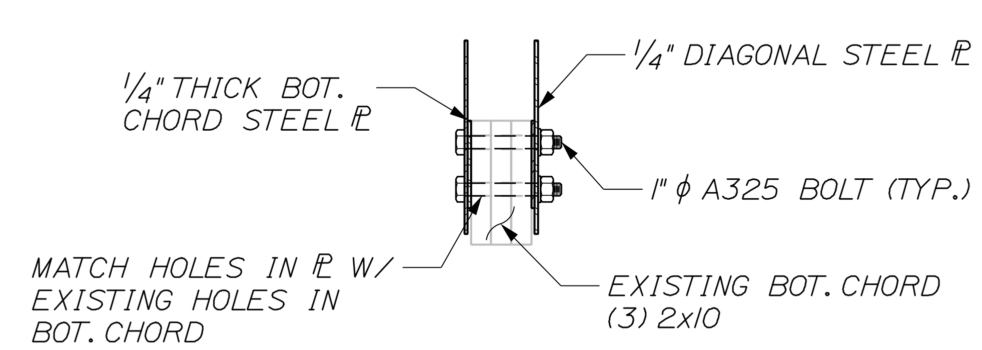
1/2" = 1'-0"

NOTE: (2) 2x8 MAY BE SUBSTITUTED WITH (1) SOLID SAWN 6x6



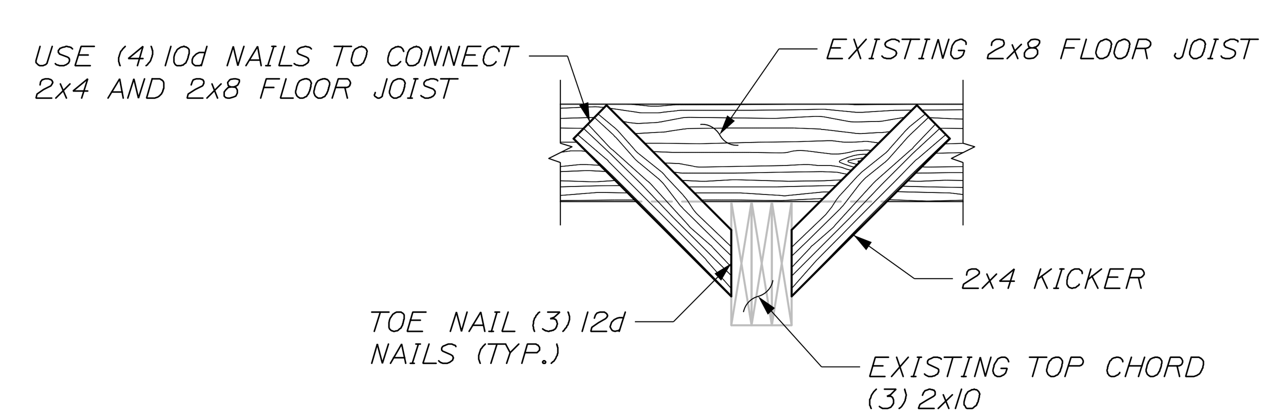
A/4 TOP CHORD SECTION

1" = 1'-0"



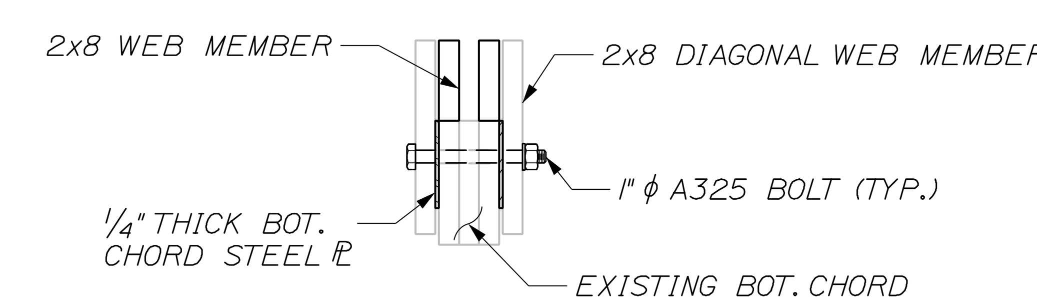
B/4 BOTTOM CHORD SECTION

1" = 1'-0"



C/4 TOP CHORD BRACE SECTION

1" = 1'-0"



D/4 BOTTOM CHORD SECTION

1" = 1'-0"

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PREPARED FOR: TOWN OF BOWDOINHAM

CEE 075-ST-13

STATE OF MAINE
ERIC A. CALDERWOOD
No. 9099
LICENSED
PROFESSIONAL ENGINEER

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FIELD CHANGES		

BOWDOINHAM RECYCLING CENTER

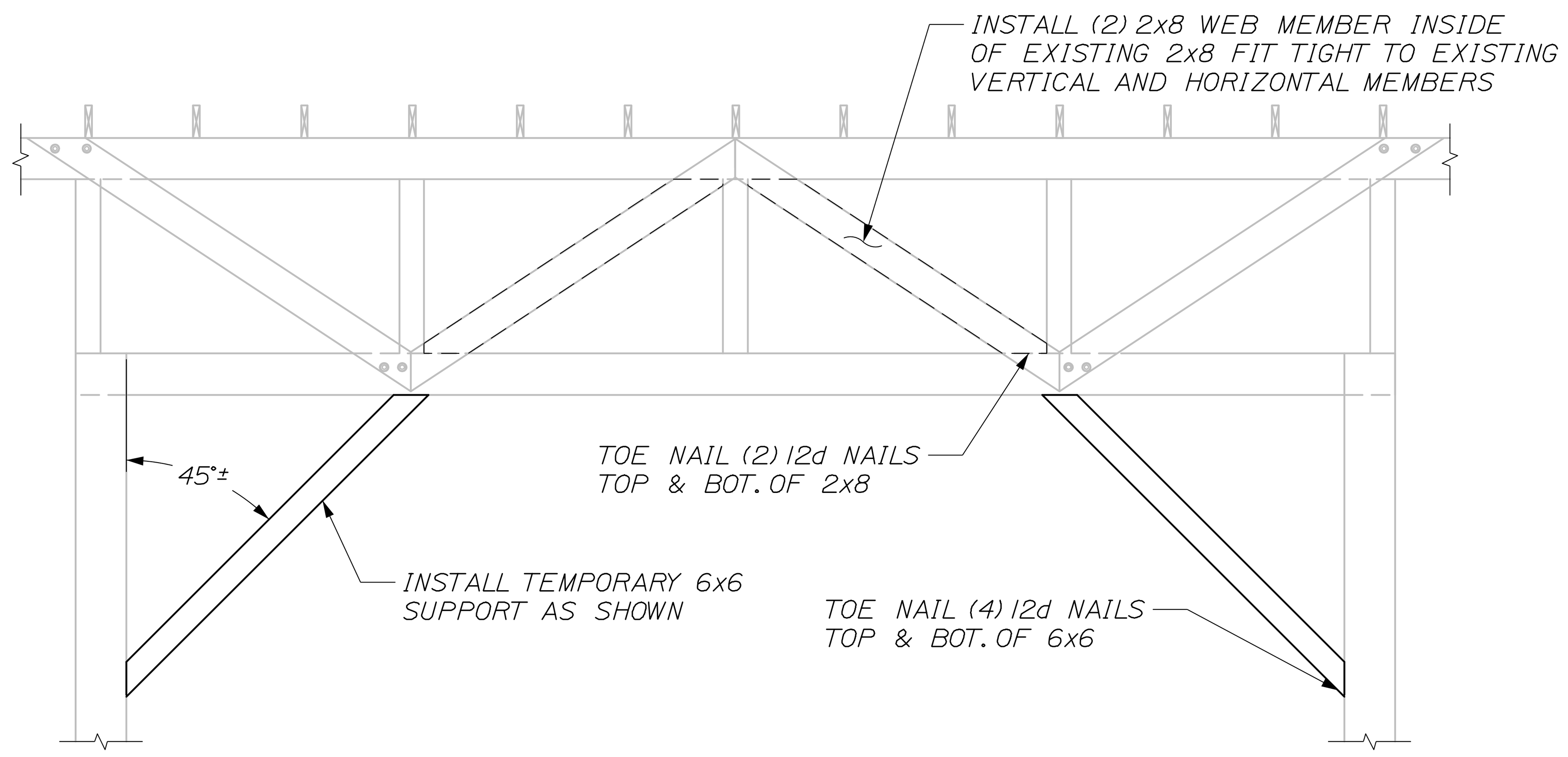
TRUSS REHABILITATION DETAILS

SHEET NUMBER

4

CALDERWOOD ENGINEERING, 222 RIVER ROAD, RICHMOND, ME 04357, PH. (207) 737-2007 FAX (207) 737-2008

Filename: ...\005_Truss Rehab steps.dgn Workgroup: Structures Division Username: common Date:1/27/2014

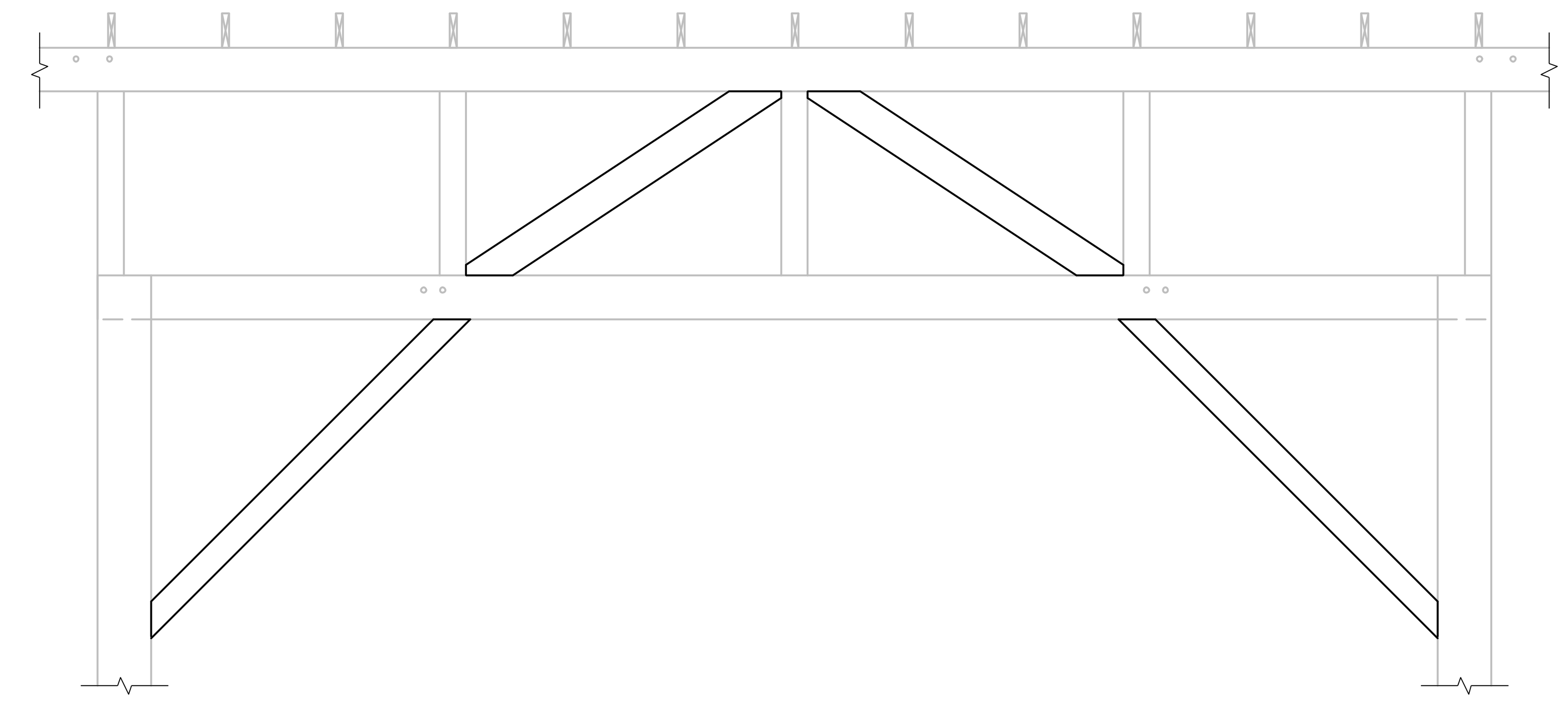


TRUSS REHAB ~ STEP 1

3/8" = 1'-0"

INSTALL TEMPORARY SUPPORTS

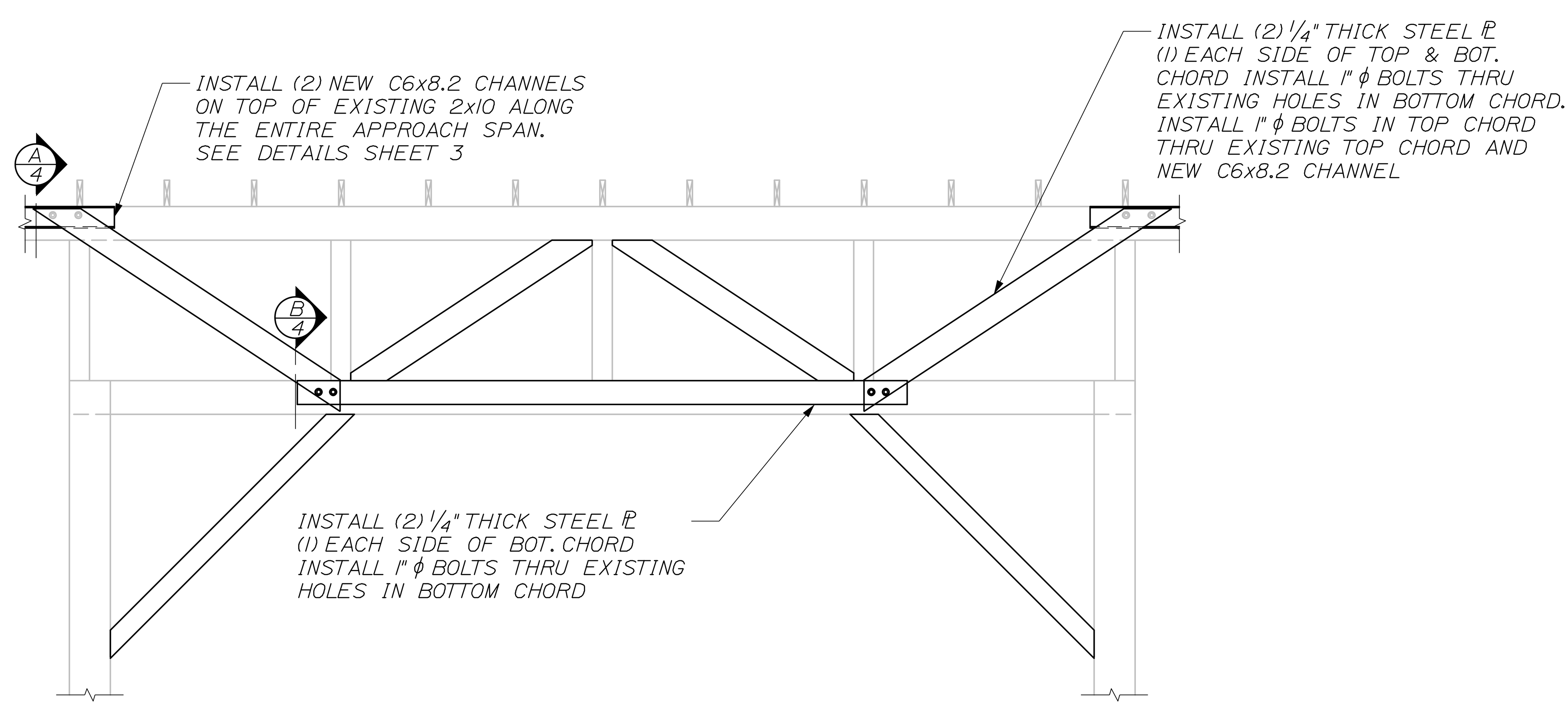
NOTE: (2) 2x8 MAY BE SUBSTITUTED WITH (1) 6x6 POST



TRUSS REHAB ~ STEP 2

3/8" = 1'-0"

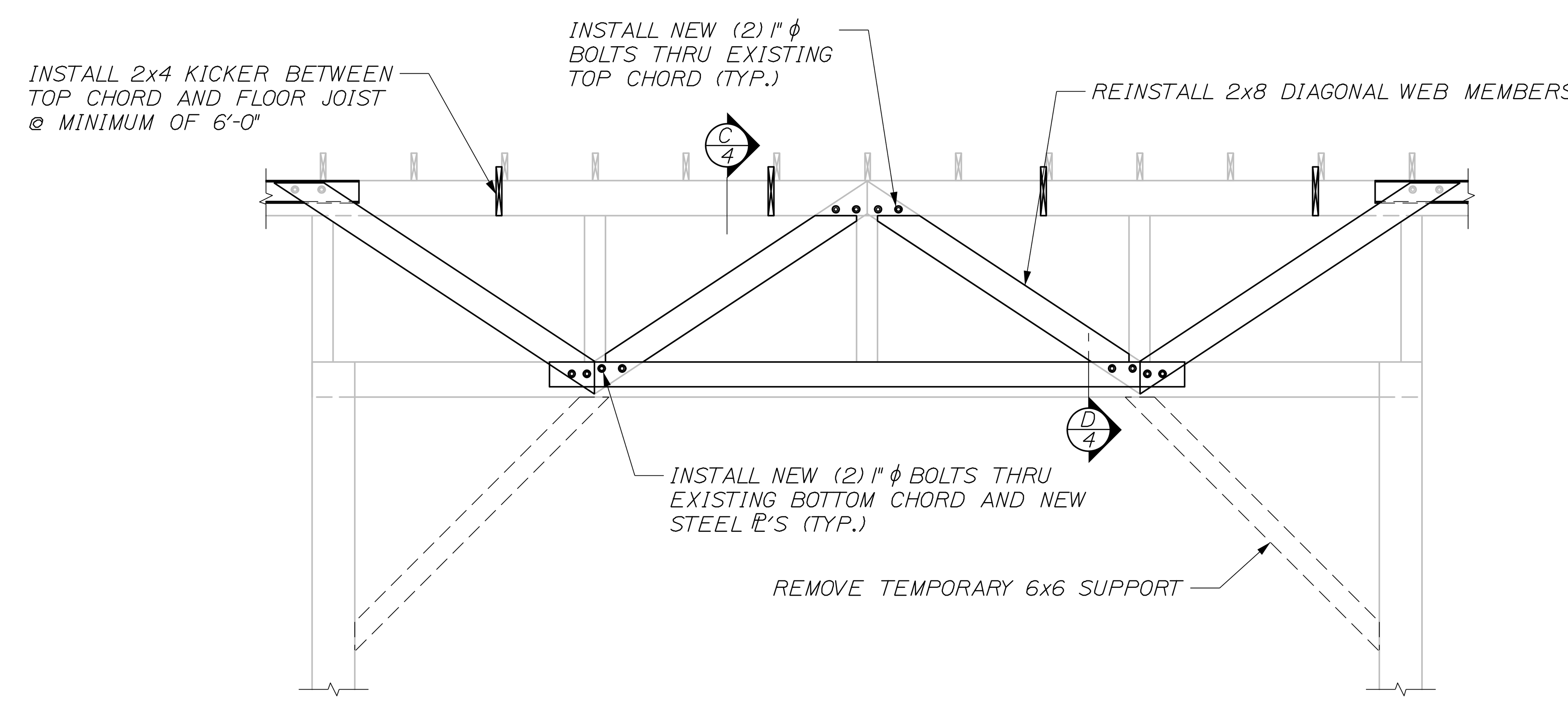
REMOVE DIAGONAL WEB MEMBERS



TRUSS REHAB ~ STEP 3

3/8" = 1'-0"

INSTALL NEW 1/4" ℄ ON BOTTOM CHORD AND DIAGONAL WEB MEMBERS



TRUSS REHAB ~ STEP 4

3/8" = 1'-0"

REINSTALL 2x8 WEB MEMBERS & 2x6 LATERAL SUPPORTS FOR TOP CHORD

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PREPARED FOR:
TOWN OF BOWDOINHAM

CEE 075-ST-13

STATE OF MAINE
ERIC A. CALDERWOOD
No. 9099
LICENSED
PROFESSIONAL ENGINEER

P.E. NUMBER

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CHECKED-REVIEWED	GNM	
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

BOWDOINHAM RECYCLING CENTER

TRUSS REHAB SEQUENCE

SHEET NUMBER

5

CALDERWOOD ENGINEERING
222 RIVER ROAD
RICHMOND, ME 04357
PH (207) 737-2007 FAX (207) 737-2008

CALDERWOOD ENGINEERING

August 10th, 2020

Town Manager
Town of Bowdoinham
13 School Street,
Bowdoinham, ME 04008

RE: Bowdoinham Recycling Building Modifications

To whom it may concern,

This memo is to address the modifications required for the Bowdoinham Recycling center building.

Calderwood Engineering inspected the Bowdoinham Recycling center on June 24th, 2020. Calderwood Engineering had previously inspected the building on August 21st, 2013, and noted several areas of the building that required modification to bring the building up to code. During the inspection on June 24th, 2020, Calderwood Engineering noted that none of the proposed changes had been made. Attached to this Memo are the details and memo provided in 2013. All of the changes outlined in that memo, as well as the modifications below must be performed to bring the building up to code. The only part of the existing memo that no longer applied is the cost estimate, which does not reflect 2020 prices. In addition to the existing modifications, Calderwood Engineering found the following issues.

On the 1st floor, there are (4) columns located under a set of Lally columns placed on the 2nd floor. Currently, (1) column is located off center and leaves the beam on top of the columns with 1" of bearing. This column must be repositioned to have a minimum of 2.5" of bearing length.

On the Northeast corner of the building, the wall next to existing door frame is not connected to the foundation. Calderwood Engineering has designed a connection between the existing timber wall frame to the concrete footing by installing a sill plate and connecting this to the existing door frame. See the attached details.

On the Southeast corner of the building, several of the 2x4's in the exterior wall have deteriorated and must be replaced. Calderwood Engineering noted at least (11) that must be replaced, however the exact number must be determined in the field. These 2x4's have been exposed to the elements due to the lack of sheathing or any type of facing on the exterior of the building. As noted in the attached memo from 2013, ½" plywood/OSB should be added to all exterior wall that are not covered by plywood or by planking.

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RICHMOND, ME 04357
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Attached are the supporting calculations, details, and the memo and details provided in 2013.

Should you have any further questions please feel free to contact us directly.

Respectfully Submitted

Thad D. Chamberlain, EI

Project: 11-Town of Bowdoinham-20; 01-Recycling Center-20
Client: Town of Bowdoinham

Construction Engineering Design: Calderwood Engineering
Design Computations by: Thad Chamberlain, EI
Design Check by: Eric Calderwood, PE

Project Notes:
Check Bowdoinham Recycling Building, determine capacity of additional members not addressed in the calculations and details dated December 2013.

References: NDS 2012, ASCE 7-14, IBC 2009

Check Design of additional beam supporting second floor:

(4) 2x8's at single span between additional support columns:

$b_{beam} := 1.5\ in$ $\quad d_{beam} := 7.25\ in$

$L_{beam} := 12\ ft$ spacing between columns

$w_1 := 3\ ft + 2\ in$ distance from center of column to center of exterior column

$w_2 := 8\ ft + 10.25\ in$ distance from center of column to center of interior column

$\sigma_{LL} := 125\ psf$ light storage warehouse (From Table 4-1, ASCE 7)

$\sigma_{floor} := 5\ psf$ timber framing, assume 5psf (see pg 6 of 93 of original calculations)

Calculate total load applied by beam:

$$w_{beam} := \left(\sigma_{LL} + \sigma_{floor}\right) \cdot \left(\frac{w_1}{2} + \frac{w_2}{2}\right) + 45\ pcf \cdot \left(4 \cdot b_{beam} \cdot d_{beam}\right) = 794.948\ plf$$

$$P_{beam} := \frac{w_{beam} \cdot L_{beam}}{2} = 4.77\ kip$$

This is the reaction load at the end of the beam

Calculate beam in bearing:

$F_{cperp} := 335\ psi$ SPF No.2 South, Ref. NDS 2012

$l_{bear} := 2.5\ in$ length of bearing

$C_m := 1.0$ $\quad C_t := 1.0$ $\quad C_i := 1.0$ NDS 4.3.3/4/8

$$C_b := \frac{l_{bear} + 0.375\ in}{l_{bear}} = 1.15$$ NDS 3.10-2

$F_{cperp}' := F_{cperp} \cdot C_m \cdot C_t \cdot C_i \cdot C_b = 385.25\ psi$ NDS Table 4.3.1

$A_{bear} := l_{bear} \cdot b_{beam} = 3.75\ in^2$ total bearing area of single beam

Check Design of additional beam supporting second floor:

$$f_{cperp} := \frac{P_{beam}}{A_{bear} \cdot 4} = 317.979 \text{ psi}$$

$$\text{Check} := \begin{array}{l} \text{if } f_{cperp} \le F_{cperp}' \\ \quad \| \text{"Ok for bearing"} \\ \text{else} \\ \quad \| \text{"Check"} \end{array} = \text{"Ok for bearing"}$$

2.5 inches of bearing is required for the 2x8's, shift column as required to provide enough bearing for each beam, or install additional 6x6 column.

Check concrete blocks under Truss columns:

Existing 7"x7"x5" concrete blocks in compression:

$P_{vert} := 19440 \text{ lbf} = 19.44 \text{ kip}$ — See page 84 of 94 of previous design calculations
This is the factored load in the columns supporting the truss

$f'_c := 2 \text{ ksi}$ — no information on existing concrete blocks, assume 2ksi

$b_{conc} := 6 \text{ in}$ — concrete block dimension (7" square block, assume some section loss to 6" square block)

$t_{conc} := 5 \text{ in}$ — thickness of concrete block

$A_1 := b_{conc} \cdot b_{conc} = 36 \text{ in}^2$ — bearing area on concrete block

$B_n := 0.85 \cdot f'_c \cdot A_1 = 61.2 \text{ kip}$ — unfactored bearing capacity of block (ACI 318, Table 22.8.3.2)

$\phi_{bearing} := 0.65$ — (ACI 318, Table 21.2.1)

$$\phi B_n := \phi_{bearing} \cdot B_n = 39.78 \text{ kip}$$

$$\text{Check} := \begin{array}{l} \text{if } P_{vert} \le \phi B_n \\ \quad \| \text{"Concrete Ok for Bearing"} \\ \text{else} \\ \quad \| \text{"Check"} \end{array} = \text{"Concrete Ok for Bearing"}$$

Concete blocks are ok for bearing, should be monitored for section loss

Capacities of Tapcon Blue Anchors:

Buildex **Tapcon**®
#1 CHOICE OF PROFESSIONALS

PERFORMANCE TABLES
BLUE, WHITE, AND STAINLESS

ULTIMATE TENSION AND SHEAR VALUES (LBS/KN) IN CONCRETE

ANCHOR DIA In.(mm)	MIN. DEPTH OF EMBEDMENT In.(mm)	f'c = 2000 PSI (13.8 MPa)		f'c = 3000 PSI (20.7 MPa)		f'c = 4000 PSI (27.6 MPa)		f'c = 5000 PSI (34.5 MPa)	
		TENSION Lbs. (kN)	SHEAR Lbs. (kN)	TENSION Lbs. (kN)	SHEAR Lbs. (kN)	TENSION Lbs. (kN)	SHEAR Lbs. (kN)	TENSION Lbs. (kN)	SHEAR Lbs. (kN)
3/16 (4.8)	1 (25.4)	600 (2.7)	720 (3.2)	625 (2.8)	720 (3.2)	650 (2.9)	720 (3.2)	800 (3.6)	860 (3.8)
	1-1/4 (31.8)	845 (3.7)	720 (3.2)	858 (3.8)	720 (3.2)	870 (3.9)	720 (3.2)	1,010 (4.5)	860 (3.8)
	1-1/2 (38.1)	1,090 (4.8)	860 (3.8)	1,090 (4.8)	860 (3.8)	1,090 (4.8)	860 (3.8)	1,220 (4.8)	860 (3.8)
	1-3/4 (44.5)	1,450 (6.5)	870 (3.9)	1455 (6.5)	870 (3.9)	1,460 (6.5)	990 (4.4)	1,730 (7.7)	990 (4.4)
1/4 (6.4)	1 (25.4)	750 (3.3)	900 (4.0)	775 (3.4)	900 (4.0)	800 (3.6)	1,360 (6.1)	950 (4.2)	1,440 (6.4)
	1-1/4 (31.8)	1,050 (4.7)	900 (4.0)	1,160 (5.2)	900 (4.0)	1,270 (5.6)	1,360 (6.1)	1,515 (6.7)	1,440 (6.4)
	1-1/2 (38.1)	1,380 (6.1)	1,200 (5.3)	1,600 (7.2)	1,200 (5.3)	1,820 (8.1)	1,380 (6.1)	2,170 (9.7)	1,670 (7.4)
	1-3/4 (44.5)	2,020 (9.0)	1,670 (7.4)	2,200 (9.8)	1,670 (7.4)	2,380 (10.6)	1,670 (7.4)	2,770 (12.3)	1,670 (7.4)

Safe working loads for single installation under static loading should not exceed 25% of the ultimate load capacity.

ULTIMATE TENSION AND SHEAR VALUES (LBS/KN) IN HOLLOW BLOCK

ANCHOR DIA In.(mm)	ANCHOR EMBEDMENT In.(mm)	LIGHTWEIGHT BLOCK		MEDIUM WEIGHT BLOCK	
		TENSION Lbs. (kN)	SHEAR Lbs. (kN)	TENSION Lbs. (kN)	SHEAR Lbs. (kN)
3/16 (4.8)	1 (25.4)	220 (1.0)	400 (1.8)	340 (1.5)	730 (3.2)
1/4 (6.4)	1 (25.4)	250 (1.1)	620 (1.8)	500 (2.2)	1,000 (4.4)

Safe working loads for single installation under static loading should not exceed 25% of the ultimate load capacity.
NOTE: 3/16" Tapcon requires 5/32" bit, 1/4" Tapcon requires 3/16" bit.

ALLOWABLE EDGE AND SPACING DISTANCES

PARAMETER	ANCHOR DIA. In.(mm)	NORMAL WEIGHT CONCRETE			CONCRETE MASONRY UNITS (CMU)		
		FULL CAPACITY (Critical Distance Inches)	REDUCED CAPACITY (Minimal Distnce Inches)	LOAD REDUCTION FACTOR	FULL CAPACITY (Critical Distance Inches)	REDUCED CAPACITY (Minimal Distance Inches)	LOAD REDUCTION FACTOR
Spacing Between Anchors - Tension	3/16 1/4	3 4	1-1/2 2	0.73 0.66	3 4	1-1/2 2	1.00 0.84
Spacing Between Anchors - Shear	3/16 1/4	3 4	1-1/2 2	0.83 0.82	3 4	1-1/2 2	1.00 0.81
Edge Distance - Tension	3/16 1/4	1-7/8 2-1/2	1 1-1/4	0.83 0.82	3 4	2 2	0.91 0.81
Edge Distance - Shear	3/16 1/4	2-1/4 3	1-1/8 1-1/2	0.70 0.59	3 4	2 2	0.93 0.80

For SI: 1 inch = 25.4 mm

Design connection for existing wall to foundation at swinging door:

Determine applied load:

$w_{gap} := 2.5\ ft$ — width of section not connected to floor

$h_{gap} := 15\ ft + 3\ in$ — height of section not connected to floor

Wind load on this section of wall:

$V_{wind} := 115\ mph$ — wind speed (ASCE 7-16, Figure 26.5-1b) Category 2 building

$k_d := 0.85$ — ASCE 7-16, Table 26.6-1

$k_z := 0.85$ — ASCE 7-16, Table 26.10-1 (less than 15ft above ground level)

$k_e := 1.00$ — ASCE 7-16, Table 26.9-1

$k_{zt} := 1.0$ — ASCE 7-16, 26.8.2

$$q_z := 0.00256 \cdot k_z \cdot k_{zt} \cdot k_d \cdot k_e \cdot \left(\frac{V_{wind}}{mph}\right)^2 \cdot psf = 24.461\ psf$$ ASCE 7-16, 26.10-1

$G := 0.85$ — Gust effect factor, ASCE 7-16, 26.11.1

$Gcp_i := -0.18$ — ASCE 7-16, Table 26.13-1

$C_p := 0.8$ — ASCE 7-16, Figure 27.3-1

$$P_{wind} := \left(q_z \cdot C_p \cdot G - q_z \cdot Gcp_i\right) \cdot \frac{h_{gap}}{2} \cdot w_{gap} = 0.401\ kip$$ ASCE 7-16, 27.3-1

This is the wind load reaction at each end of the wall section, design connection to footing to carry this applied load:

Install nail plate (2x4) into concrete slab with concrete screws, install 4x4 block under existing end wall column and nail column to block, and block to nail plate.

Design Nail Plate:

2x Nail plate, calculate number of concrete anchors required to carry wind loads:

(2) 1/4" Tapcon Blue Concrete Screws or equivalent:

$N_{screw} := 3$ $\qquad \phi_{screw} := 0.25\ in$

$V_{screw} := 900\ lbf$ — assuming 1" of embedment into 2ksi concrete

$f_{red\_spacing} := 0.82$ — Reduction for 2in spacing (minimum allowable)

$f_{red\_edge} := 0.59$ — Reduction for 1-1/2" edge distance (Minimum allowable)

$$V_{allow} := V_{screw} \cdot f_{red\_spacing} \cdot f_{red\_edge} = 0.435\ kip$$

```
Check := if N_screw · V_allow ≥ P_wind        = "Screws Ok for applied loads"
           ‖ "Screws Ok for applied loads"
         else
           ‖ "Check"
```

Ok with (2) Anchor screws, check with Powder Actuated nails:

Design Nail Plate:

Check design with Powder actuated nails:

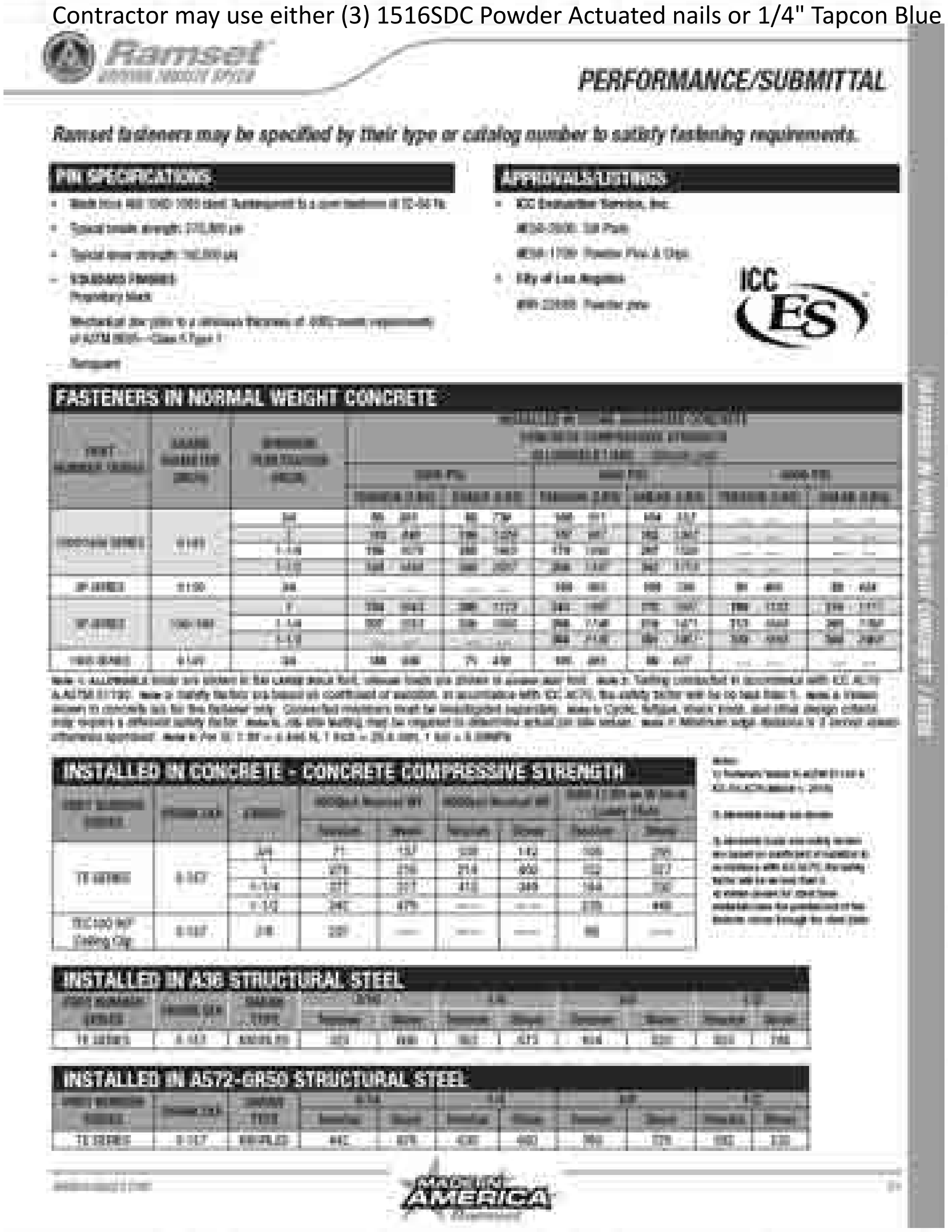
$V_{nail} := 166 \text{ lbf}$ — This is the shear strength of a single Powder Actuated nail, with 1" of embedment (assuming using the 1516SDC with 2-1/2" overall length, in 2ksi concrete)

$N_{nail} := 3$ — Number of nails

$$Check := \text{if } \left(N_{nail} \cdot V_{nail}\right) \geq P_{wind} \;\; \| \text{"Ok for Shear"} \;\; \text{else} \;\; \| \text{"Check"} = \text{"Ok for Shear"}$$

(3) Nails Required

Contractor may use either (3) 1516SDC Powder Actuated nails or 1/4" Tapcon Blue Concrete screws


Ramset

PERFORMANCE/SUBMITTAL

Ramset fasteners may be specified by their type or catalog number to satisfy fastening requirements.

PIN SPECIFICATIONS

APPROVALS/LISTINGS

ICC ES

FASTENERS IN NORMAL WEIGHT CONCRETE

INSTALLED IN CONCRETE - CONCRETE COMPRESSIVE STRENGTH

INSTALLED IN A36 STRUCTURAL STEEL

INSTALLED IN A572-GR50 STRUCTURAL STEEL

MADE IN AMERICA

Design connection from nail plate to block to wall end column:

#8 wood screw:

$N_{screw} := 3$

$W := 82\ \dfrac{lbf}{in}$ Table 11.2b, NDS 2012 withdrawal of #8 wood screws

$Z := 78\ \dfrac{lbf}{in}$ Table 11L, NDS 2012 for #8 wood screws with 1in side member thickness (conservatively)

$C_D := 1.6$ NDS Table 2.3.2, wind load factor

$C_M := 1.0$ $C_t := 1.0$ $C_g := 1.0$ $C_\Delta := 1.0$

$C_{eg} := 1.0$ $C_{di} := 1.0$ $C_{tn} := 1.0$ Toe nail factor for screws is 1.0

$Z' := Z \cdot C_D \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta \cdot C_{eg} \cdot C_{di} \cdot C_{tn} = 124.8\ \dfrac{lbf}{in}$ Table 10.3.1, NDS 2012

$W' := W \cdot C_D \cdot C_t \cdot C_{eg} \cdot C_{tn} = 131.2\ \dfrac{lbf}{in}$ Table 10.3.1, NDS 2012

$W_{applied} := \dfrac{P_{wind}}{\cos(45\ deg)} = 567.109\ lbf$ total withdrawal force

$L_{embed} := 1.5\ in = 1.5\ in$ this is the required embedment depth into the nail plate, at a 45deg angle

$W_{resist} := (W' \cdot L_{embed}) \cdot N_{screw} = 590.4\ lbf$

$$Check := \begin{array}{l} \text{if } W_{applied} \le W_{resist} \\ \quad \Vert \text{“Ok for Withdrawal”} \\ \text{else} \\ \quad \Vert \text{“Check”} \end{array} = \text{“Ok for Withdrawal”}$$

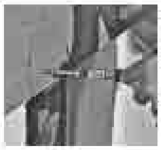
$Z_{resist} := (Z' \cdot L_{embed}) \cdot N_{screw} = 561.6\ lbf$

$$Check := \begin{array}{l} \text{if } P_{wind} \le Z_{resist} \\ \quad \Vert \text{“Ok for Shear”} \\ \text{else} \\ \quad \Vert \text{“Check”} \end{array} = \text{“Ok for Shear”}$$

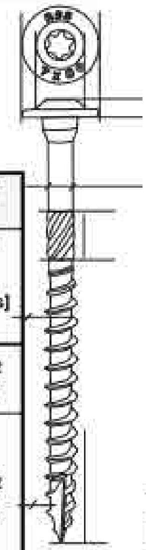
(3) #8 wood screws required to carry applied wind loads, conservatively use GRK-RSS 1/4" Diameter screws, these do not split the wood and are stronger than wood screws.

RSS™ Technical Data

Building Code Approved with a Limited Lifetime Warranty.
Scan to view to view for detailed
IBC/IRC Code Compliant ESR #2442



RSS™ Rugged Structural Screws: Ideal for anywhere you would use a traditional lag screw and more. High tensile torque and shear strength means a 5/16" diameter RSS™ screw has the same strength as a 1/2" lag screw. Available from #10 up to 3/8" diameters in lengths from 1-1/2" to 16". Approved for use in all applications that include treated lumber. Also available in *PHE*INOX™ Stainless Steel, RSS™ JTS used for joists and trusses, RSS™ LPS for structural insulated panel systems and RSS™ LTF designed for log home and timber frames.



	FASTENER DESIGNATION	OVERALL LENGTH[1] (inches)	LENGTH OF THREAD[2] (inches)	MINOR THREAD DIAMETER[3] (inches)	SHANK DIAMETER[3] (inches)	OUTSIDE THREAD DIAMETER[3] (inches)	ALLOWABLE STEEL STRENGTH: Bending Yield Strength[4] F_{yb} (psi)	Tensile (psi) [pounds]	Shear (psi) [pounds]
RSS	1/4 x 2 1/2"	2 3/8	1 1/2	0.150	0.169	0.239	170,427	188,301 [3,336]	127,792 [2,264]
RSS	1/4 x 3 1/8"	3 1/8	2						
RSS	1/4 x 3 1/2"	3 1/2	2 3/8						
RSS	5/16 x 2 1/2"	2 3/8	1 1/2	0.174	0.199	0.280	190,920	178,051 [4,247]	123,592 [2,948]
RSS	5/16 x 2 3/4"	2 3/4	1 3/4						
RSS	5/16 x 3 1/8"	3 1/8	2 1/8						
RSS	5/16 x 3 1/2"	3 1/2	2 1/2						
RSS	5/16 x 4"	3 7/8	2 3/4						
RSS	5/16 x 5 1/8"	5	3 1/2						
RSS	5/16 x 6"	5 7/8	3 7/8						
RSS	3/8 x 3 1/8"	3 1/8	2 1/8	0.191	0.223	0.310	178,080	203,809 [5,824]	129,305 [3,695]
RSS	3/8 x 4"	3 7/8	2 3/4						
RSS	3/8 x 5 1/8"	5 1/8	3 1/2						
RSS	3/8 x 6"	5 7/8	4						
RSS	3/8 x 7 1/4"	7	4 1/2						
RSS	3/8 x 8"	7 7/8	4 3/8						
RSS	3/8 x 10"	9 3/4	5						
RSS	3/8 x 12"	11 7/8	5 7/8						
RSS	3/8 x 14 1/8"	14 1/8	5 7/8						
RSS	3/8 x 16"	15 5/8	5 3/4						
LPS	1/4 x 8"	7 7/8	2 7/8	0.152	0.172	0.238	172,620	172,950 [3,155]	109,635 [2,000]
LTF	3/8 x 8"	7 7/8	3 7/8	0.191	0.220	0.310	167,580	179,390 [5,144]	114,525 [3,284]
LTF	3/8 x 10"	9 7/8	3 7/8						
LTF	3/8 x 12"	11 3/4	3 7/8						
*PHE*INOX	1/4 x 2 1/2"	2 3/8	1 1/2	0.152	0.170	0.237	111,460	103,799 [1,886]	90,260 [1,640]
*PHE*INOX	5/16 x 2 1/2"	2 3/8	1 5/8	0.171	0.195	0.276	118,360	104,767 [2,419]	86,880 [2,006]
*PHE*INOX	5/16 x 3 1/8"	3 1/8	2 1/8						
*PHE*INOX	5/16 x 4"	3 7/8	2 1/2						
*PHE*INOX	5/16 x 5 1/8"	5 1/8	3 3/8						
*PHE*INOX	5/16 x 6"	5 7/8	3 7/8						
JTS	1/4 x 3 3/8"	3 3/8	1 3/8	0.153	0.173	0.240	226,373	180,999 [3,312]	126,131 [2,308]
JTS	1/4 x 5"	5	1 5/8						
JTS	1/4 x 6 3/4"	6 3/4	1 1/2						

For **SI:** 1 inch = 25.4 mm; 1 psi = 6.9 kPa.

ULTIMATE LOAD VALUES TENSILE AND SHEAR

ATTACHMENT D

CALCUALTION SHEETS

ROOF

Bowdoinham P_g = 60 psf (hazards@ATCouncil.org)

Roofing: Corrugated Metal @ 0.4 psf.
Strapping assume 1x4 HF @16' oc = 0.95 plf nailed on rafter
2x8 HF = 2.11 plf rafter
HF, dry, 28 pcf (Engineering toolbox.com); Assume #1
Cellulose insul 3 pcf $\times \frac{8"}{12"}$ = 2 psf

DL roofing = 5.01 psf

wind (hazards @ AT council.org) RC II ASCE 7-10, 115 mph

Snowload calcs Medeek
Slippery roof C_s = 0.79 (CALDERWOOD (CEE) USED .85, MORE CONSERVATIVE THAN CODE)
Thermal C_t = 1.1
Exposure C_e = 1.0
Importance I_s = 1.0

p_f = 46.2 psf
p_s = 36.5 psf

Unbalanced snowload
P windward (North) = 0 psf
P lee (SOUTH) = 60 psf
This condition matches owner's experience, roof collapse was on South side @ East end

C_r (repetitive member) only applies for spacing of 24" or less

Rafter
For Vitruvius Calcs: OK in Bending by 6%. w/ no wind or Roof Live load, not unbalanced, 9' span

Span = 9' + 9'

Rafter Beam Span = 12', A_T = 12' x 14' (10'+4')
Fails in Bending by 64%, shear by 8%, in Deflection by 28%

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C: 207.522.9366
TF: 800.242.1969

Project Name 243 Post Rd, B'ham

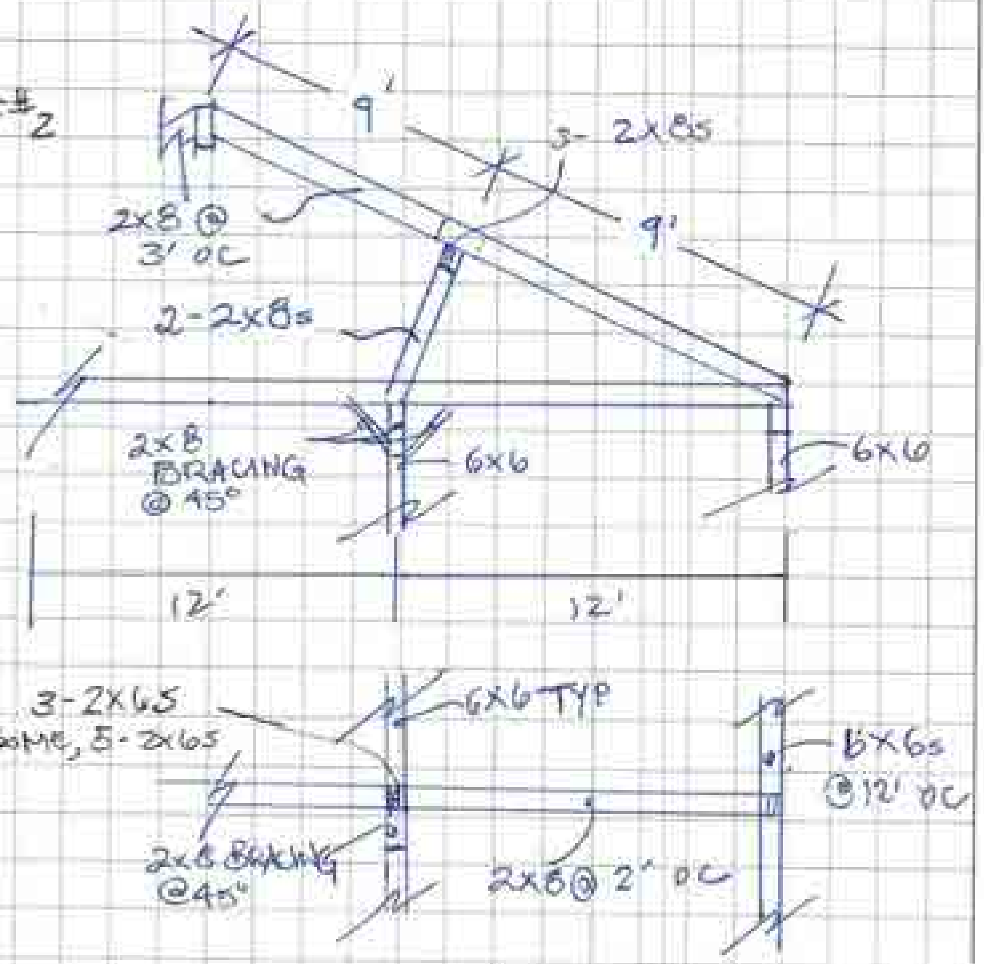
Designed by HCW Date 9/28/20

Checked by ______ Date ______

HWE P/N 20-0388-ME Sheet 1 of 3

FRAMING SCHEMATIC
NTS

MATERIAL ASSUMED
HF #1
EXCPT NEW 2x12s, HF #2



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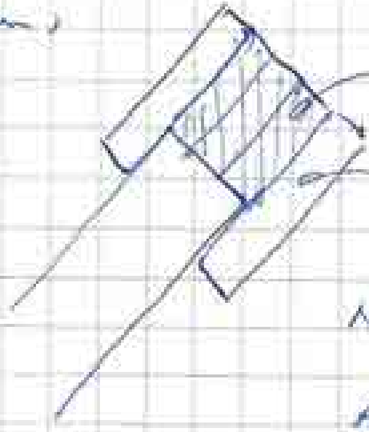
Project Name 243 POST RD B'HAM
Designed by HCW Date 9/28/20
Checked by ______ Date ______
CE P/N 20-0389-ME Sheet 2 of 3

ROOF

Rafter Beam, MODIFIED

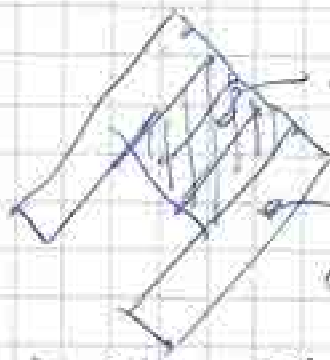


3-2X8s

2-2X12s ADDED
ASSUME HF #2

NEW 2X12s ALONE FAIL BY 58%
Bending, 4% shear
ADDING 2 MORE 2X12s FAILS IN BENDING BY 16%

TRY ADDING LVLs



3-2X8s

2-2.0E 1.75" x 11.25" LVL
OK BY 34% (REGULAR LOAD)

RAFTERS @ 8' WORKS BY 25% (REGULAR LOAD)
RAFTERS @ 9' WORKS BY 6% (" ")
UPPER RAFTERS NEED TO BE SIZED FOR UNBALANCED LOAD.

RAFTER BEAM: ADD 1 LVL EA SIDE OF BEAM - SIZE FOR ADDITIONAL LOAD OF UNBALANCED SNOW.

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Project Name 243 POST RD B'HAM
Designed by HCW Date 9/28/20
Checked by Date
HWE P/N 20-0388-ME Sheet 3 of 3

FLOOR LOADING

FRAMING AT THE
THE OPENING ABOVE THE COMPOSTER ISN'T WELL SUPPORTED.
1) REMOVE THE PLYWOOD SHEATHING, RELOC. THE HANDRAIL
OR 2) REINFORCE W/ AN ENG. DESIGN

FLOOR DL

4/4 DECKING 2.15 PSF x 2 = 4.3 PLF
2X8S @ 24" OC 2.67# PLF
6.97 PLF EXISTING ORIGINAL
ADD 1/2" PLYWOOD 1.42 PSF x 2 = 2.84#
9.81 PLF MOST OF 2nd & 3rd FLOORS

WHERE USED,
ADD 1/8" STEEL PLATE SHEETS
5.1 PSF x 2 = 10.2 PLF
20.01 PLF AREAS USING PALLET JACK

CEE USED 4.9 PSF FOR
DEAD LOAD, = 9.8 PLF ✓

SPANS: 36' − 1' = 35'
35'/3 = 11'−8"
MAX LOAD W/ DL OF 9.8 PLF = 65 PLF = 32.5 PSF
W/ DL OF 20.01 PLF = 44 PLF = 22.0 PSF

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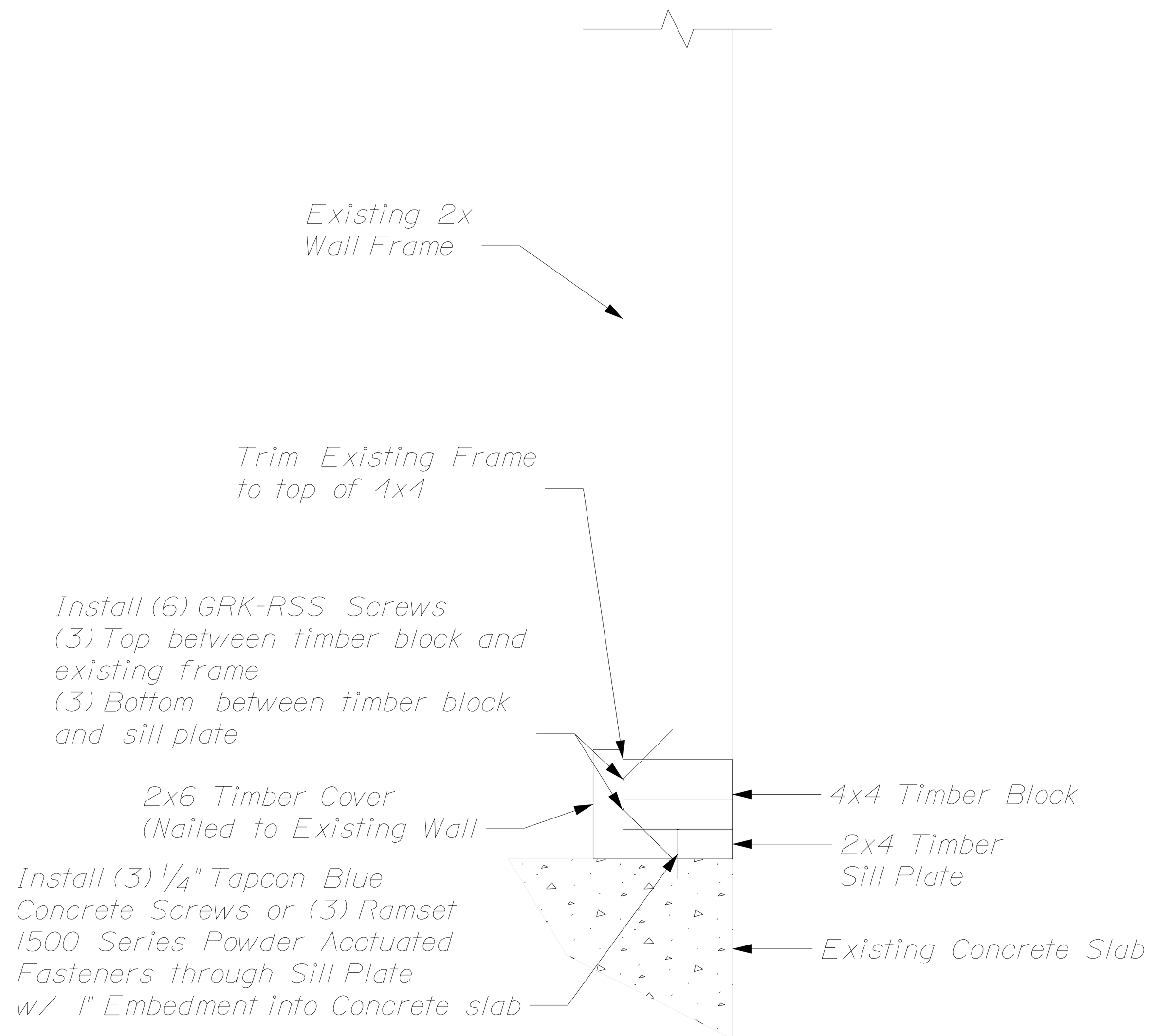
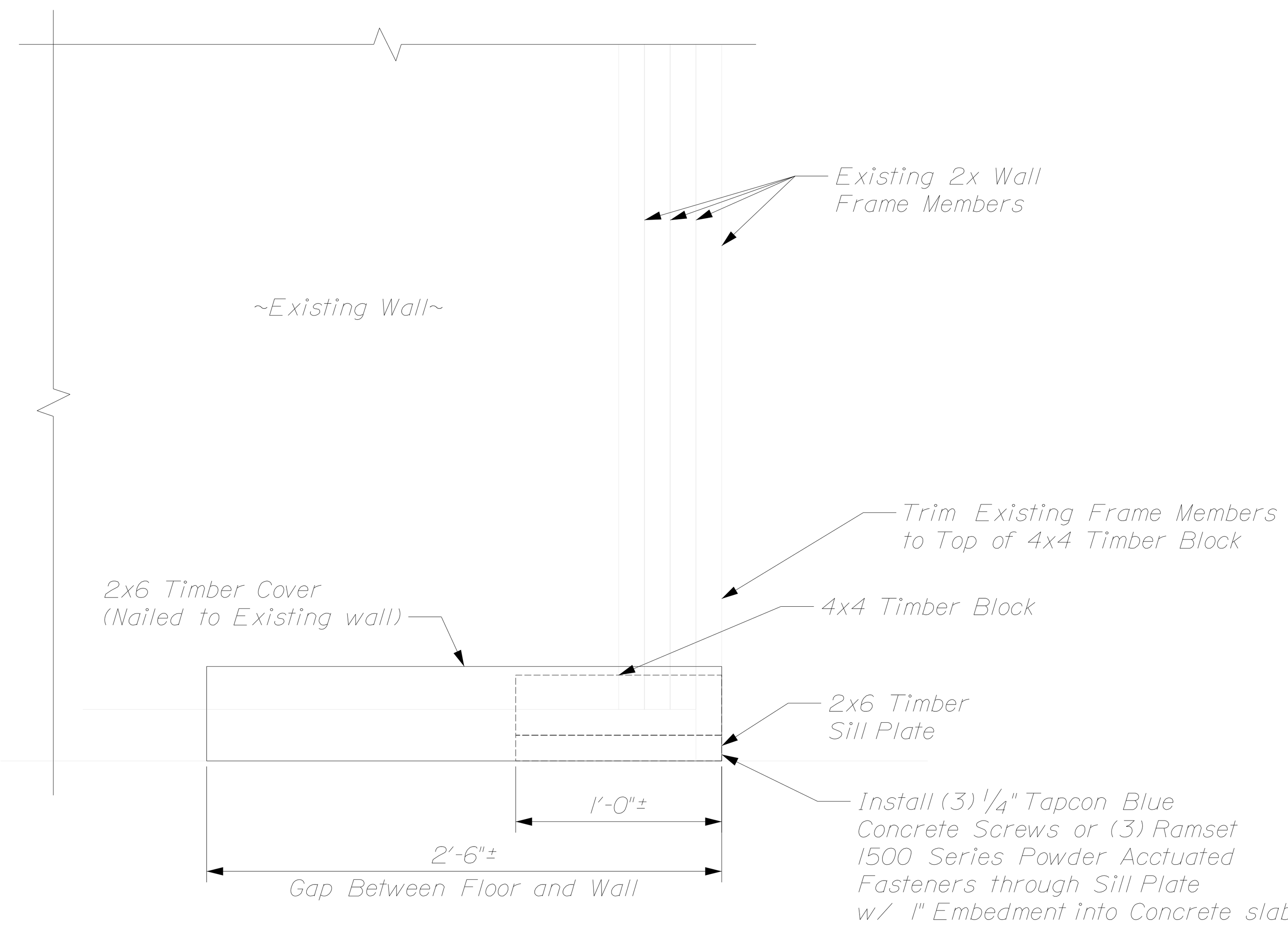
Project Name 243 POST RD B'HAM	
Designed by HCW	Date 9/28/20
Checked by	Date
HWE P/N 20-0388-ME	Sheet 1 of 1



Wall Connection Repair Section

Scale: $1\frac{1}{2}$" = 1'-0"

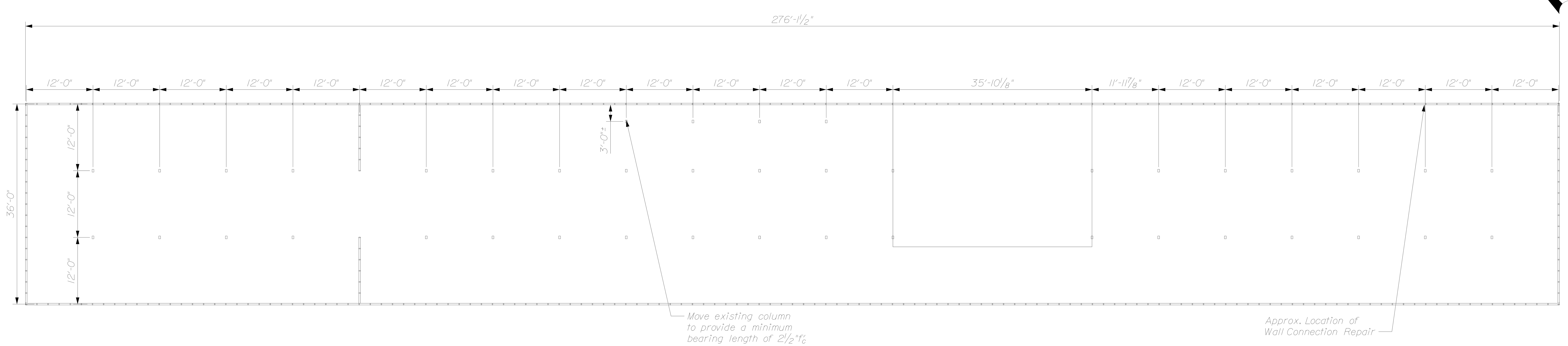


Wall Connection Repair Elevation

Scale: $1\frac{1}{2}$" = 1'-0"

General Notes

1. Concrete Screws to be Tapcon Blue with 1/4"ϕ and 1" Embedment into Concrete, or approved alternative.
2. Powder Acctuated Nails to be Ramset 1500 Series with 1" Embedment into Concrete, or approved alternative.
3. Timber Screws to be GRK-RSS with a diameter of 1/4" and $1\frac{1}{2}$" Embedment into timber, or an approved alternative.
4. Timber cover to be measured in the field to cover the existing gap.



GENERAL BUILDING PLAN

Scale: 1" = 10'

CALDERWOOD ENGINEERING, ETC.
STRUCTURAL ENGINEERING • DETAILING SERVICES
222 RIVER RD, RICHMOND, ME 04357 PH/FX (207)737-2007/(207)737-2008

PREPARED FOR: TOWN OF BOWDOINHAM

PROJECT NUMBER: 011-TOWNOFBOWDOINHAM-20

STATE OF MAINE ERIC CALDERWOOD No 9099 LICENSED PROFESSIONAL ENGINEER

SIGNATURE 9099
P.E. NUMBER AUG 2020
DATE

	NAME	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED		TDC	AUG 2020
CHECKED-REVIEWED		ETC	AUG 2020
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

BOWDOINHAM RECYCLING CENTER

RECYCLING CENTER BUILDING MODIFICATIONS

SHEET NUMBER

1

OF 1

CALDERWOOD ENGINEERING 222 RIVER ROAD RICHMOND, ME 04357 PH. (207) 737-2007 FAX (207) 737-2008

Filename: ...\001_RecyclingCenter.dgn Division: Bridge Username: common Date: 8/24/2020

PROJECT SUMMARY

Project Name: 243 Post Road, Bowdoinham, ME

Governing Codes:
Building Code: 2018 International Building Code
ASCE: ASCE 7-16
Steel: AISC 360-16
Concrete: ACI 318-14
Masonry: TMS 402/602-16

Module Location: 2x8 Roof Rafter
Module Level: Roof
Module Type: Roof Rafter
Material Type: Solid Sawn Hem-Fir No. 1
Member Dimensions: 1.5 in. X 7.25 in. X 10 ft @ 36 in. Spacing
Section Adequacy: -14.2%
Controlling Factor: Bending-Tension

Module Location: Roof Purlins
Module Level: Roof
Module Type: Roof Beam
Material Type: Solid Sawn Hem-Fir No. 2
Member Dimensions: 1.5 in. X 11.25 in. X 12 ft
Section Adequacy: -57.7%
Controlling Factor: Bending Stress Y

Module Location: roof purlins w/ added 2x12s
Module Level: Roof
Module Type: Roof Beam
Material Type: Structural Composite Lumber Weyerhaeuser 2.0E Microlam LVL
Member Dimensions: 1.75 in. X 11.25 in. X 12 ft
Section Adequacy: 33.8%
Controlling Factor: Bending Stress Y

Module Location: 2x8 Roof Rafter @9 ft span
Module Level: Roof
Module Type: Roof Rafter
Material Type: Solid Sawn Hem-Fir No. 1
Member Dimensions: 1.5 in. X 7.25 in. X 9 ft @ 36 in. Spacing
Section Adequacy: 5.6%
Controlling Factor: Bending-Tension

Module Location: 2x8 Floor Joist max w plywood over 44 decking
Module Level: Main Floor
Module Type: Floor Joist
Material Type: Solid Sawn Hem-Fir No. 1
Member Dimensions: 1.5 in. X 7.25 in. X 11.67 ft
Section Adequacy: -0.2%
Controlling Factor: Bending Stress Y

Module Location: 2x8 Floor Joist max w plywood + steel plate over 44 decking
Module Level: Main Floor
Module Type: Floor Joist
Material Type: Solid Sawn Hem-Fir No. 1
Member Dimensions: 1.5 in. X 7.25 in. X 11.67 ft
Section Adequacy: 0.5%
Controlling Factor: Bending Stress Y

Module Location: 2x8 Floor Joist w plywood + steel plate and 5 ft of loading
Module Level: Main Floor
Module Type: Floor Joist
Material Type: Solid Sawn Hem-Fir No. 1
Member Dimensions: 1.5 in. X 7.25 in. X 11.67 ft @ 24 in. Spacing
Section Adequacy: 13.9%
Controlling Factor: Bending Stress Y

Module Location: Beam supporting travel lane of pallet jack
Module Level: Main Floor
Module Type: Roof Beam
Material Type: Structural Composite Lumber Weyerhaeuser 2.0E Microlam LVL
Member Dimensions: 1.5 in. X 11.25 in. X 12 ft @ 12 in. Spacing
Section Adequacy: 15.9%
Controlling Factor: Bending Stress Y

FAIL

DATE:	10/2/2020	COMPANY:	Helen Watts Engineering PLLC
VITRUVIUS BUILD:	Base	DESIGNED BY:	Helen Watts
CUSTOMER:		REVIEWED BY:	Helen Watts
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	2x8 Roof Rafter	CODE:	2018 International Building Code
TYPE:	ROOF RAFTER	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 1	(1) 1.5 X 7.25	36(in) O.C.	DRY	

2x8 Roof Rafter DIAGRAM

BEAM PROPERTIES

Start (ft): 0 End (ft): 10 Member Slope: 5/12 Actual Length (ft): 10.83 Roof Pitch: 5/12 O.C. Spacing(in): 36

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
10.88	47.63	2.04	2.15	1	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	975	625	150	1350	405	1500	550
Adjusted Values	1170	750	150	1417	405	1500	550
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.2	1.2	1	1.05	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

BEAM DATA

		Unbraced Length (ft)		Beam End				
Span	Length (ft)	Top	Bottom	Elev. Diff (ft)	CL(Top)	CL(Bottom)	CL(Left)	CL(Right)
1	10	0	10	4.166667	1.00	0.70	1.00	1.00

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (49.3%)**	87.5	172.5	0	D+S	1.15
Bending Stress Y (psi)	**FAIL (-14.2%)**	1568.2	1345.5	5.42	D+S	1.15
Deflection (in)	**PASS (39.3%)**	0.438 (=L/297)	0.722 (=L/180)	5.42	S	
Compressive Stress (psi)	**PASS (97.9%)**	24.3	1133.3	0	D+S	1.15
Tensile Stress (psi)	**PASS (97.2%)**	24.3	862.5	10.83	D+S	1.15
Bearing Stress (psi)	**PASS (72.5%)**	111.5	405.0	0	D+S	1.15
Bending-Compression (Unit)	**FAIL (-14.2%)**	1.17	1.00	4.9	D+S	1.15
Bending-Tension (Unit)	**FAIL (-14.2%)**	1.17	1.00	5.1	D+S	1.15

REACTIONS

V-(lbf) M-(lbf-ft)

Y axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	94	0	0	593	0	0	0	0	0	0	0
B	94	0	0	593	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Uniform (lbf/ft)	15.17	15.17	0	10	Dead	Y
Uniform (lbf/ft)	109.5	109.5	0	10	Snow	Y
Self Weight (lbf/ft)	2.15	2.15	0	10	Dead	Y

NOTES

FAIL

DATE:	10/2/2020	COMPANY:	Helen Watts Engineering PLLC
VITRUVIUS BUILD:	Base	DESIGNED BY:	Helen Watts
CUSTOMER:		REVIEWED BY:	Helen Watts
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	Roof Purlins	CODE:	2018 International Building Code
TYPE:	ROOF BEAM	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(2) 1.5 X 11.25	DRY		

Roof Purlins DIAGRAM

BEAM PROPERTIES

Start (ft): 0 End (ft): 12 Member Slope: 0/12 Actual Length (ft): 12 Roof Pitch: 5/12

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
33.75	355.96	6.33	6.66	2	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	850	525	150	1300	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1	1	1	1	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

BEAM DATA

		Unbraced Length (ft)		Beam End				
Span	Length (ft)	Top	Bottom	Elev. Diff (ft)	CL(Top)	CL(Bottom)	CL(Left)	CL(Right)
1	12	0	12	0	1.00	0.54	1.00	1.00

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**FAIL (-4.4%)**	156.9	150.0	12	D+L	1
Bending Stress Y (psi)	**FAIL (-57.7%)**	2008.7	850.0	6	D+L	1
Deflection (in)	**PASS (35.6%)**	0.515 (=L/280)	0.800 (=L/180)	6	L	
Bearing Stress (psi)	**PASS (17.0%)**	336.3	405.0	0	D+L	1

REACTIONS

V-(lbf) M-(lbf-ft)

Y axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	465	3066	0	0	0	0	0	0	0	0	0
B	465	3066	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Uniform (lbf/ft)	70.84	70.84	0	12	Dead	Y
Uniform (lbf/ft)	511	511	0	12	Live	Y
Self Weight (lbf/ft)	6.66	6.66	0	12	Dead	Y

PASS

DATE:	10/2/2020	COMPANY:	Helen Watts Engineering PLLC
VITRUVIUS BUILD:	Base	DESIGNED BY:	Helen Watts
CUSTOMER:		REVIEWED BY:	Helen Watts
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	roof purlins w/ added 2x12s	CODE:	2018 International Building Code
TYPE:	ROOF BEAM	NDS:	2018 NDS
MATERIAL:	STRUCTURAL COMPOSITE LUMBER		

Weyerhaeuser	2.0E Microlam LVL	(2) 1.75 X 11.25	DRY		

roof purlins w/ added 2x12s DIAGRAM

BEAM PROPERTIES

Start (ft): 0 End (ft): 12 Member Slope: 0/12 Actual Length (ft): 12 Roof Pitch: 5/12

Area	Ix	Iy	BSW	Lams	Cfn	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
39.38	415.28	10.05	11.48	2	7.35	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	2600	1895	285	2510	750	2000	1016.535
Adjusted Values	2600	1895	285	2510	750	2000	1017
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1

Bending Adjustment Factors C_V = 1.01 C_r = 1 Volume factor Is applied on a load combination basis And Is Not reflected in the adjusted values

BEAM DATA

Span	Length (ft)	Unbraced Length (ft) Top	Bottom	Beam End Elev. Diff (ft)	CL(Top)	CL(Bottom)	CL(Left)	CL(Right)
1	12	0	12	0	1.00	0.52	1.00	1.00

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (52.4%)**	135.6	285.0	12	D+L	1
Bending Stress Y (psi)	**PASS (33.8%)**	1735.9	2622.9	6	D+L	1
Deflection (in)	**PASS (64.1%)**	0.287 (=L/502)	0.800 (=L/180)	6	L	
Bearing Stress (psi)	**PASS (61.3%)**	290.6	750.0	0	D+L	1

REACTIONS

V-(lbf) M-(lbf-ft)

Y axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	494	3066	0	0	0	0	0	0	0	0	0
B	494	3066	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Uniform (lbf/ft)	70.84	70.84	0	12	Dead	Y
Uniform (lbf/ft)	511	511	0	12	Live	Y
Self Weight (lbf/ft)	11.48	11.48	0	12	Dead	Y

NOTES

PASS

DATE:	10/2/2020	COMPANY:	Helen Watts Engineering PLLC
VITRUVIUS BUILD:	Base	DESIGNED BY:	Helen Watts
CUSTOMER:		REVIEWED BY:	Helen Watts
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	2x8 Roof Rafter @9 ft span	CODE:	2018 International Building Code
TYPE:	ROOF RAFTER	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 1	(1) 1.5 X 7.25	36(in) O.C.	DRY	

2x8 Roof Rafter @9 ft span DIAGRAM

BEAM PROPERTIES

Start (ft): 0 End (ft): 9 Member Slope: 5/12 Actual Length (ft): 9.75 Roof Pitch: 5/12 O.C. Spacing(in): 36

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
10.88	47.63	2.04	2.15	1	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	975	625	150	1350	405	1500	550
Adjusted Values	1170	750	150	1417	405	1500	550
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.2	1.2	1	1.05	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

BEAM DATA

		Unbraced Length (ft)		Beam End				
Span	Length (ft)	Top	Bottom	Elev. Diff (ft)	CL(Top)	CL(Bottom)	CL(Left)	CL(Right)
1	9	0	10	3.75	1.00	0.70	1.00	1.00

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (54.4%)**	78.7	172.5	9	D+S	1.15
Bending Stress Y (psi)	**PASS (5.6%)**	1270.3	1345.5	4.88	D+S	1.15
Deflection (in)	**PASS (55.7%)**	0.288 (=L/407)	0.650 (=L/180)	4.88	S	
Compressive Stress (psi)	**PASS (98.2%)**	21.9	1242.1	0	D+S	1.15
Tensile Stress (psi)	**PASS (97.5%)**	21.9	862.5	9.75	D+S	1.15
Bearing Stress (psi)	**PASS (75.2%)**	100.3	405.0	0	D+S	1.15
Bending-Compression (Unit)	**PASS (5.6%)**	0.94	1.00	4.41	D+S	1.15
Bending-Tension (Unit)	**PASS (5.6%)**	0.94	1.00	4.59	D+S	1.15

REACTIONS

V-(lbf) M-(lbf-ft)

Y axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	84	0	0	534	0	0	0	0	0	0	0
B	84	0	0	534	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Uniform (lbf/ft)	15.17	15.17	0	9	Dead	Y
Uniform (lbf/ft)	109.5	109.5	0	9	Snow	Y
Self Weight (lbf/ft)	2.15	2.15	0	9	Dead	Y

NOTES

FAIL

DATE:	10/2/2020	COMPANY:	Helen Watts Engineering PLLC
VITRUVIUS BUILD:	Base	DESIGNED BY:	Helen Watts
CUSTOMER:		REVIEWED BY:	Helen Watts
PROJECT LOCATION:	,		
LEVEL:	Main Floor	LOADING:	ASD
LOCATION:	2x8 Floor Joist max w plywood over 44 decking	CODE:	2018 International Building Code
TYPE:	FLOOR JOIST	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 1	(1) 1.5 X 7.25	0(in) O.C.	DRY	

2x8 Floor Joist max w plywood over 44 decking DIAGRAM

BEAM PROPERTIES

Start (ft): 0 End (ft): 11.67 Member Slope: 0/12 Actual Length (ft): 11.67 O.C. Spacing(in): 24

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
10.88	47.63	2.04	2.15	1	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	975	625	150	1350	405	1500	550
Adjusted Values	1346	750	150	1417	405	1500	550
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.2	1.2	1	1.05	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1.15

BEAM DATA

		Unbraced Length (ft)		Beam End				
Span	Length (ft)	Top	Bottom	Elev. Diff (ft)	CL(Top)	CL(Bottom)	CL(Left)	CL(Right)
1	11.67	0	10	0	1.00	0.70	1.00	1.00

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (53.5%)**	69.8	150.0	0	D+L	1
Bending Stress Y (psi)	**FAIL (-0.2%)**	1348.5	1345.5	5.83	D+L	1
Deflection (in)	**PASS (2.4%)**	0.380 (=L/369)	0.389 (=L/360)	5.83	L	
Bearing Stress (psi)	**PASS (76.2%)**	96.4	405.0	0	D+L	1

REACTIONS

V-(lbf) M-(lbf-ft)

Y axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	127	379	0	0	0	0	0	0	0	0	0
B	127	379	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Uniform (lbf/ft)	19.6	19.6	0	11.67	Dead	Y
Uniform (lbf/ft)	65	65	0	11.67	Live	Y
Self Weight (lbf/ft)	2.15	2.15	0	11.67	Dead	Y

DATE:	10/2/2020	COMPANY:	Helen Watts Engineering PLLC
VITRUVIUS BUILD:	Base	DESIGNED BY:	Helen Watts
CUSTOMER:		REVIEWED BY:	Helen Watts
PROJECT LOCATION:	,		
LEVEL:	Main Floor	LOADING:	ASD
LOCATION:	2x8 Floor Joist max w plywood + steel plate over 44 decking	CODE:	2018 International Building Code
TYPE:	FLOOR JOIST	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 1	(1) 1.5 X 7.25	0(in) O.C.	DRY	

2x8 Floor Joist max w plywood + steel plate over 44 decking DIAGRAM

BEAM PROPERTIES

Start (ft): 0 End (ft): 11.67 Member Slope: 0/12 Actual Length (ft): 11.67 O.C. Spacing(in): 24

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
10.88	47.63	2.04	2.15	1	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	975	625	150	1350	405	1500	550
Adjusted Values	1346	750	150	1417	405	1500	550
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.2	1.2	1	1.05	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1.15

BEAM DATA

		Unbraced Length (ft)		Beam End				
Span	Length (ft)	Top	Bottom	Elev. Diff (ft)	CL(Top)	CL(Bottom)	CL(Left)	CL(Right)
1	11.67	0	10	0	1.00	0.70	1.00	1.00

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	PASS (53.8%)	69.3	150.0	0	D+L	1
Bending Stress Y (psi)	PASS (0.5%)	1339.2	1345.5	5.83	D+L	1
Deflection (in)	PASS (13.8%)	0.503 (=L/278)	0.584 (=L/240)	5.83	D+L	
Bearing Stress (psi)	PASS (76.4%)	95.7	405.0	0	D+L	1

REACTIONS

V-(lbf) M-(lbf-ft)

Y axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	246	257	0	0	0	0	0	0	0	0	0
B	246	257	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Uniform (lbf/ft)	19.6	19.6	0	11.67	Dead	Y
Uniform (lbf/ft)	44	44	0	11.67	Live	Y
Uniform (lbf/ft)	20.4	20.4	0	11.67	Dead	Y
Self Weight (lbf/ft)	2.15	2.15	0	11.67	Dead	Y

PASS

DATE:	10/2/2020	COMPANY:	Helen Watts Engineering PLLC
VITRUVIUS BUILD:	Base	DESIGNED BY:	Helen Watts
CUSTOMER:		REVIEWED BY:	Helen Watts
PROJECT LOCATION:	,		
LEVEL:	Main Floor	LOADING:	ASD
LOCATION:	2x8 Floor Joist w plywood + steel plate and 5 ft of loading	CODE:	2018 International Building Code
TYPE:	FLOOR JOIST	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 1	(2) 1.5 X 7.25	24(in) O.C.	DRY	

2x8 Floor Joist w plywood + steel plate and 5 ft of loading DIAGRAM

BEAM PROPERTIES

Start (ft): 0 End (ft): 11.67 Member Slope: 0/12 Actual Length (ft): 11.67 O.C. Spacing(in): 24

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
21.75	95.27	4.08	4.29	2	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	975	625	150	1350	405	1500	550
Adjusted Values	1346	750	150	1417	405	1500	550
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.2	1.2	1	1.05	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1.15

BEAM DATA

		Unbraced Length (ft)		Beam End				
Span	Length (ft)	Top	Bottom	Elev. Diff (ft)	CL(Top)	CL(Bottom)	CL(Left)	CL(Right)
1	11.67	0	10	0	1.00	0.73	1.00	1.00

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (44.8%)**	82.9	150.0	0	D+L	1
Bending Stress Y (psi)	**PASS (13.9%)**	1159.0	1345.5	4.2	D+L	1
Deflection (in)	**PASS (29.5%)**	0.274 (=L/510)	0.389 (=L/360)	5.25	L	
Bearing Stress (psi)	**PASS (71.7%)**	114.4	405.0	0	D+L	1

REACTIONS

V-(lbf) M-(lbf-ft)

Y axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	258	943	0	0	0	0	0	0	0	0	0
B	258	257	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Uniform (lbf/ft)	19.6	19.6	0	11.67	Dead	Y
Uniform (lbf/ft)	20.4	20.4	0	11.67	Dead	Y
Uniform (lbf/ft)	240	240	0	5	Live	Y
Self Weight (lbf/ft)	4.29	4.29	0	11.67	Dead	Y

DATE:	10/2/2020	COMPANY:	Helen Watts Engineering PLLC
VITRUVIUS BUILD:	Base	DESIGNED BY:	Helen Watts
CUSTOMER:		REVIEWED BY:	Helen Watts
PROJECT LOCATION:	,		
LEVEL:	Main Floor	LOADING:	ASD
LOCATION:	Beam supporting travel lane of pallet jack	CODE:	2018 International Building Code
TYPE:	ROOF BEAM	NDS:	2018 NDS
MATERIAL:	STRUCTURAL COMPOSITE LUMBER		

Weyerhaeuser	2.0E Microlam LVL	(4) 1.5 X 11.25	DRY		

Beam supporting travel lane of pallet jack DIAGRAM

BEAM PROPERTIES

Start (ft): 0 End (ft): 12 Member Slope: 0/12 Actual Length (ft): 12 Roof Pitch: 0/12

Area	Ix	Iy	BSW	Lams	Cfn	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
67.5	711.91	12.66	19.69	4	7.35	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	2600	1895	285	2510	750	2000	1016.535
Adjusted Values	2600	1895	285	2510	750	2000	1017
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1

Bending Adjustment Factors C_V = 1.01 C_r = 1 Volume factor Is applied on a load combination basis And Is Not reflected in the adjusted values

BEAM DATA

		Unbraced Length (ft)		Beam End				
Span	Length (ft)	Top	Bottom	Elev. Diff (ft)	CL(Top)	CL(Bottom)	CL(Left)	CL(Right)
1	12	0	12	0	1.00	0.40	1.00	1.00

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (53.2%)**	133.3	285.0	12	D+L	1
Bending Stress Y (psi)	**PASS (15.9%)**	2205.0	2622.9	6	D+L	1
Deflection (in)	**PASS (49.3%)**	0.406 (=L/355)	0.800 (=L/180)	6	D+L	
Bearing Stress (psi)	**PASS (61.9%)**	285.7	750.0	0	D+L	1

REACTIONS

V-(lbf) M-(lbf-ft)

Y axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	2998	3002	0	0	0	0	0	0	0	0	0
B	2998	3002	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Uniform (lbf/ft)	480	480	0	12	Dead	Y
Uniform (lbf/ft)	1201	1201	3.5	8.5	Live	Y
Self Weight (lbf/ft)	19.69	19.69	0	12	Dead	Y

NOTES

ATTACHMENT E

PROFESSIONAL RESUME

BUILDING INSPECTION ENGINEERS
PROUDLY SERVING NORTH AMERICA SINCE 1957

Helen C. Watts, P.E.

Senior Engineer

Helen Watts practices structural engineering with PE licensure in four states, with over 40 years of experience in construction, facilities engineering, inspection, and structural design for repairs, new construction, and building modifications.

Her experience includes hundreds of residential and commercial building inspections, remediation and remodeling designs, forensic investigations, and design for new construction on commercial, industrial, condominium and residential properties, as well as construction management and inspection.

For over 12 years, she worked as a Principal at Helen Watts Engineering PLLC performing inspections and design for wood, timber, masonry, concrete, and steel structures.

Helen has taught a variety of courses to engineers and the trades, including developing a curriculum and teaching the first course of structural engineering for timber framers at KVCC, and teaching structural engineering for the PE preparation course for mechanical engineers.

EDUCATION AND PROFESSIONAL AFFILIATION

University of New Hampshire, Durham, NH – 1980, BS Civil Engineering
University of Maine, Orono, ME – 1983, 5th Year Certificate, Pulp and Paper Manufacturing
Licensed Professional Engineer: Maine, New Hampshire, Massachusetts, Hawaii
Certifications: NCEES, SECB, MaineDOT LPA
Memberships: Structural Engineers Association of Maine
Timber Guild Engineering Council
ASCE Fellow, Lead for 2 Areas for Maine Infrastructure Grade 2008 -
Society of Women Engineers
Pejepscot Terrace, Brunswick, ME – Chair of the Board
Author: The Graphic Handbook of the Pretty Good House (2013)
Volume 2, The Pretty Good House (2016)

WHY I DO WHAT I DO

I want to help every building be the best it can be, and every building owner get the most out of their building dollar. Buildings should be healthy, comfortable, robust and sustainable. My work impacts the productivity of the building occupants, the carbon footprint during construction and maintenance, and the bottom line of the owners. I love finding the little problems that can be big possibilities instead of bad surprises.

WHY CRITERIUM ENGINEERS

Criterium Engineers serves a wide variety of clients across the country, and I like the challenge of assisting Criterium Franchises. I also like the care taken in producing high-quality reports.

PROJECT HIGHLIGHTS

- Inspection and report on the Gedney House, Salem, MA, owned by Historic New England and built in 1665 – Structural adequacy, durability, and ideas for the use of the building as a museum of timber and wood construction methods.
- Hathorn Block, Bowdoinham, ME – Structural evaluation and repair planning, new masonry openings, plus structural design to bring 5 stories of 1849 timber framing up to modern building code floor loadings and to provide an elevated concrete deck.
- New private residence and cottage, Biddeford, Maine – Evaluation of existing retaining wall, and design and permitting of new retaining wall under new Maine Sand Dune regulations, structural design of two new buildings, weekly construction inspection through completion of structural framing.
- Horizontal boring machine, Portsmouth Naval Shipyard, Kittery, ME – Design of foundation and installation of the foundation and the horizontal boring machine in the Controlled Industrial Access area of the shipyard
- Portland House, Portland, ME – Inspection, development of repair plans and specifications, project contracting assistance and construction inspection, repairs to 3-level parking garage. Also, repairs to the masonry exterior, and planning of work for the handrail attachment to the balcony decks.
- Danforth Heights, Portland, ME – Investigation, report, repair planning, specifications and drawings, contracting assistance, construction inspection, repairs to masonry façade to stop water intrusion. Also, inspections of 43 units of low-income townhouses with reports for maintenance planning.

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