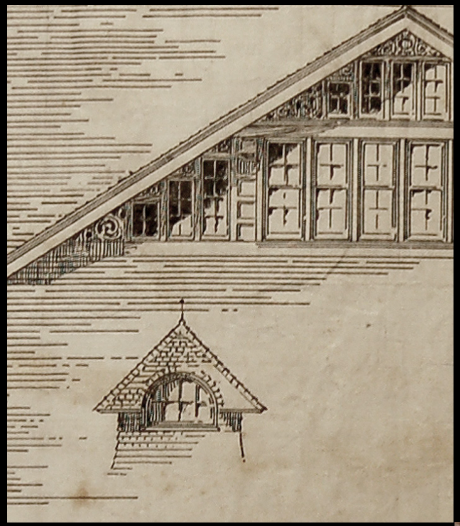

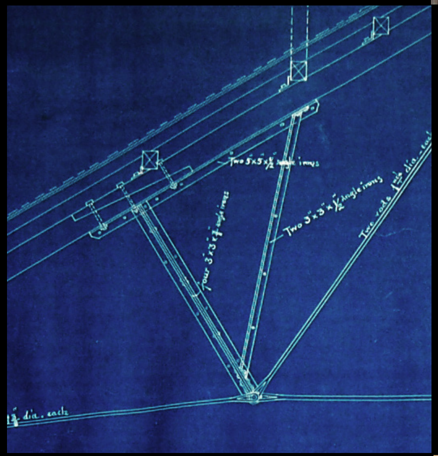

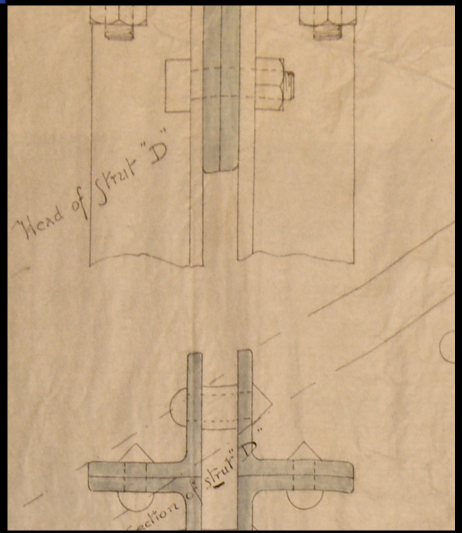

Structural Repair of the Breeding Barn at Shelburne Farms

Appendices

Structural Repair of the Breeding Barn at Shelburne Farms

Volume II: Appendices

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Sebastian Renfield, Layout and Design

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Contents

Executive Summary

Shelburne Farms is a 1400-acre working farm and National Historic Landmark District located on the eastern edge of Lake Champlain in Vermont, U.S.A. A model farm and country estate developed by founders Dr. William Seward and Lila Vanderbilt Webb, Shelburne Farms is a nationally significant cultural landscape typical of the picturesque country estates that appeared in the U.S. in the late nineteenth century. The architecture and landscape design represent significant achievements by architect Robert Henderson Robertson and landscape architect Frederick Law Olmsted, Sr. The farm is currently operated by a non-profit sustainability organization.

The Breeding Barn (1891), the center of Dr. Webb's effort to develop an improved horse breed, consists of a timber-framed main block 107 feet wide by 418 feet long, with a two-story annex. The riding ring at the center of the building, approximately 72 feet wide and 375 feet long, is spanned with composite trusses based on a design by Camille Polonceau having timber top chords with wrought iron braces and ties.

After decades of neglect, the barn is the focal point of a multi-phase stabilization and repair project. This completion report describes the structural repair of the barn that took place from 2009-10, which posed several interesting challenges. Analysis of the principal truss indicated overstresses in iron ties. Augmentation or replacement of the ties was unacceptable because of the adverse effect on historical integrity. Furthermore, decayed valley members in each of the large dormer pairs that dominate the roof required extensive repair work. Because of difficulties associated with removing such long timbers (36-54 feet), it was necessary to make most of the repairs *in situ* and without removing the roof covering.

In an effort to maintain the historic character of the barn, the multi-disciplinary project team conducted an investigation to discover the nature and condition of materials and connections and assign realistic design values, using laser scanning, resistance drilling, strength testing, and metallographic analysis. Modeling, load testing, and plane-frame analysis were used to determine the stress distribution in roof frame elements. Through modeling and analysis, it was determined that factors of safety for each of the principal elements of the riding ring truss were acceptable, and that the focus of the stabilization and repair project would be on repairing deteriorated elements and reinstating those that, for one reason or another, had been removed.

A modest testing program allowed the project team to assess the effectiveness of various *in situ* repairs. The investigation led to the development of repair strategies for roof frame elements that included the scarfing of new timbers, and inserting engineered lumber (by segmental infill) to replace decayed material. Repair designs achieved a balance between risk and integrity to ensure public safety while respecting the historic materials and design, and preserving the Breeding Barn within the cultural landscape of Shelburne Farms.

This report is divided into two volumes, the first consisting of a narrative description of the structural repair of the Breeding Barn; the second volume includes the results of lab analyses, consultants' reports, and the drawings produced for the structural repair of the barn. Volume 1 presents a brief history of the farm and its development in the late-19th century, the history of repairs and alterations made to the Breeding Barn, and conservation planning for the future re-use of the barn and surrounding buildings and landscape. This is followed by a description of the Breeding Barn and its chief structural components with respect to condition assessment, materials testing, structural analysis, design, and repair implementation. The narrative focuses on foundation stonework, characterization of iron and timber, and the assessment and repair of perimeter wall woodwork; aisle roof, wall and floor frames; riding ring columns; and riding ring roof frame.

Repairs are presented in greater detail with respect to location, the individual elements affected, and repair geometry in the as-built drawings prepared for this report. As-built drawings are included in Volume 2. The project also involved review of architect Robert Henderson Robertson's original drawings of the barn, as well as development of HABS-level drawings, and a set of design drawings for guiding the repair of the building, all of which are bound in this volume. Volume 2 also includes appendices devoted to the geotechnical survey, analysis of historic mortar, characterization of historic iron, the wood assessment, repair mockups and testing results, structural modeling and analysis, and the primary materials used in the repair of the building. The assessment, testing, structural analysis, and repair decisions made for stabilization of the Breeding Barn serve as a demonstration of sound preservation technology practices, and are the topic of several published papers and conference presentations. These are compiled in Appendix K.

Section I: Drawings

Appendix A: As-Built Drawings

The following appendix includes graphic documentation of the structural repairs made to the Breeding Barn. The drawings contain a higher level of descriptive detail with respect to location, the individual elements affected, and particular repair geometry than can be included in the written report narrative. The drawings are grouped according to the areas treated, including perimeter walls; aisle roof, wall, and floor frames; riding ring columns; and riding ring roof.

The as-built drawings were completed under the supervision of the project manager and are based on field measurements collected by his staff, along with field reports and repair details supplied by the timber-frame contractor. Due to illness, the project engineer was not able to participate directly in their preparation, and acted in an advisory capacity only.

THE STRUCTURAL REPAIR OF THE BREEDING BARN AT

SHELBURNE FARMS

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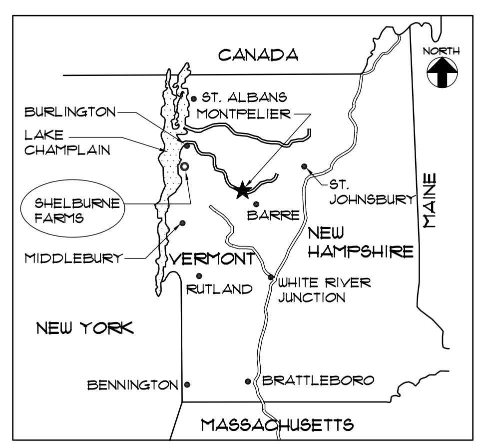
INDEX OF SHEETS (29 SHEETS)

NOTES:
ALL DIMENSIONS AND RELATIONSHIPS IN THIS DRAWING ARE APPROXIMATE AND ARE BASED ON FIELD MEASUREMENTS BY SMITH ALVAREZ SIENKIEWYCZ ARCHITECTS IN MARCH AND AUGUST 2004 AND DRAWINGS PROVIDED BY SHELBURNE FARMS, DATED FEBRUARY 16, 1995, DRAWINGS PROVIDED BY DCF ENGINEERING, INC., DATED MARCH 31, 2009, AS WELL AS DRAWINGS DELINEATED FOR THE HISTORIC AMERICAN ENGINEERING RECORD, NATIONAL PARK SERVICE, LAUREN CORTINEZ, OLIVER COX, KAREN HUGHES, AND FELICIA SANTIAGO.



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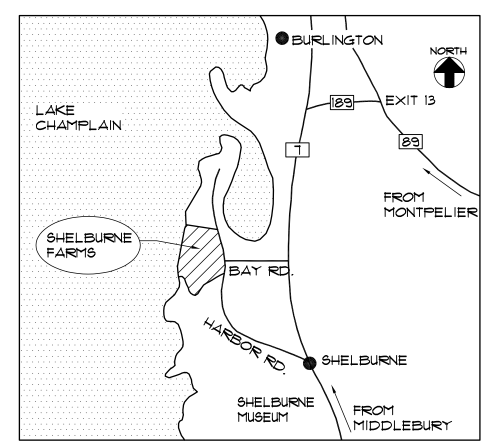
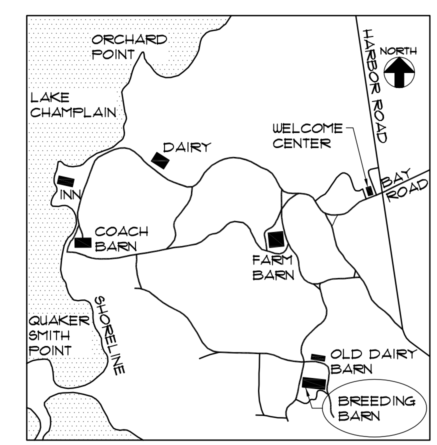


VICINITY MAP



SHELBURNE MAP

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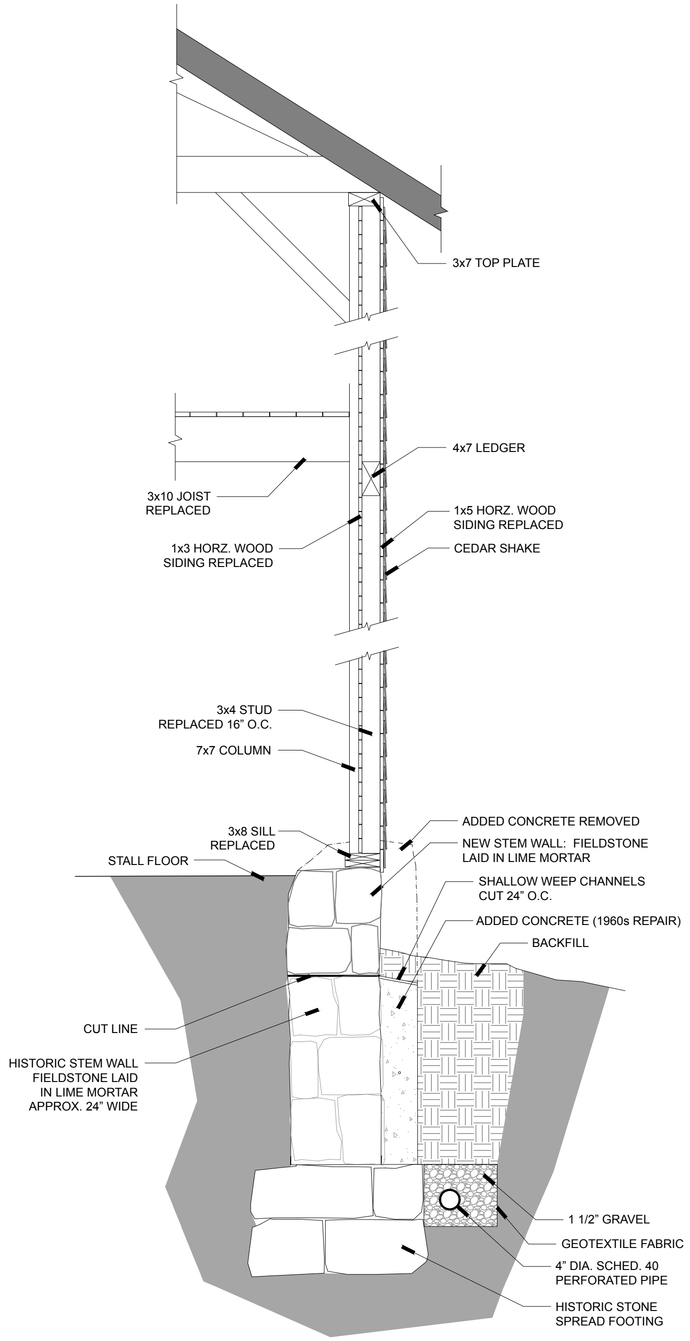
THE STRUCTURAL REPAIR OF THE BREEDING BARN AT
SHELBURNE FARMS
1611 HARBOR ROAD, SHELBURNE, VT 05482

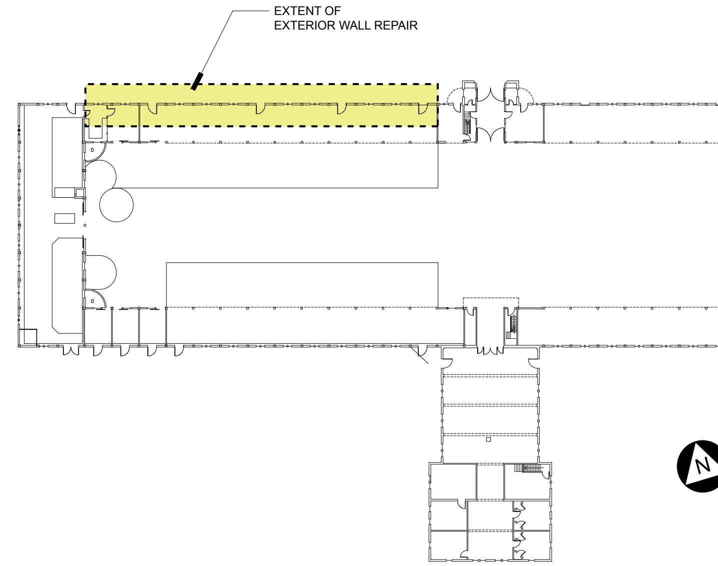
DRAWINGS PREPARED BY: KERI L. STEVENSON

30 MARCH, 2011

1



A1 / 2 EXTERIOR WALL SECTION AT REPAIR
SCALE: 3/4" = 1'-0"



B1 / 2 FOUNDATION REPAIR, KEYED FIRST FLOOR PLAN
SCALE: 1/32" = 1'-0"

NOTES

-REPAIR TYPICALS:

- REMOVAL OF ABOVE-GRADE PORTIONS OF CONCRETE COUNTER WALL
- CONSTRUCTION OF NEW STONE STEM WALL, FIELDSTONE LAID IN LIME MORTAR
- POLYETHYLENE BARRIER INSTALLED BETWEEN STONEWORK AND EXISTING CONCRETE FLOOR
- SHALLOW DRAINAGE WEEPS CUT IN TOP OF CONCRETE FOOTING
- NEW UNDERDRAIN ALONG NW WALL, INCLUDING 4" DIA. SCHED. 40 PERFORATED PIPE TIED TO EXISTING DRAIN SYSTEM, POROUS FILL, AND GEOTEXTILE FABRIC WRAP
- NEW LOCUST SILLS
- COLUMN SCARFS AT EXISTING SCARF LOCATIONS
- STUD SISTERS/SCARFS AS NEEDED
- SCARF REPAIRS OF AISLE FLOOR LEDGER AS NEEDED
- REPLACEMENT IN-KIND OF WALL SHEATHING AS NEEDED

-SEE SHEET 20 FOR GENERAL REPAIR DETAILS

-SEE SHEET 3 FOR DETAILS OF AN EXTERIOR WALL REPAIR

Shelburne Farms Breeding Barn
Foundation Repairs, Section and Keyed First Floor Plan

DRAWINGS PREPARED BY: KERI L. STEVENSON

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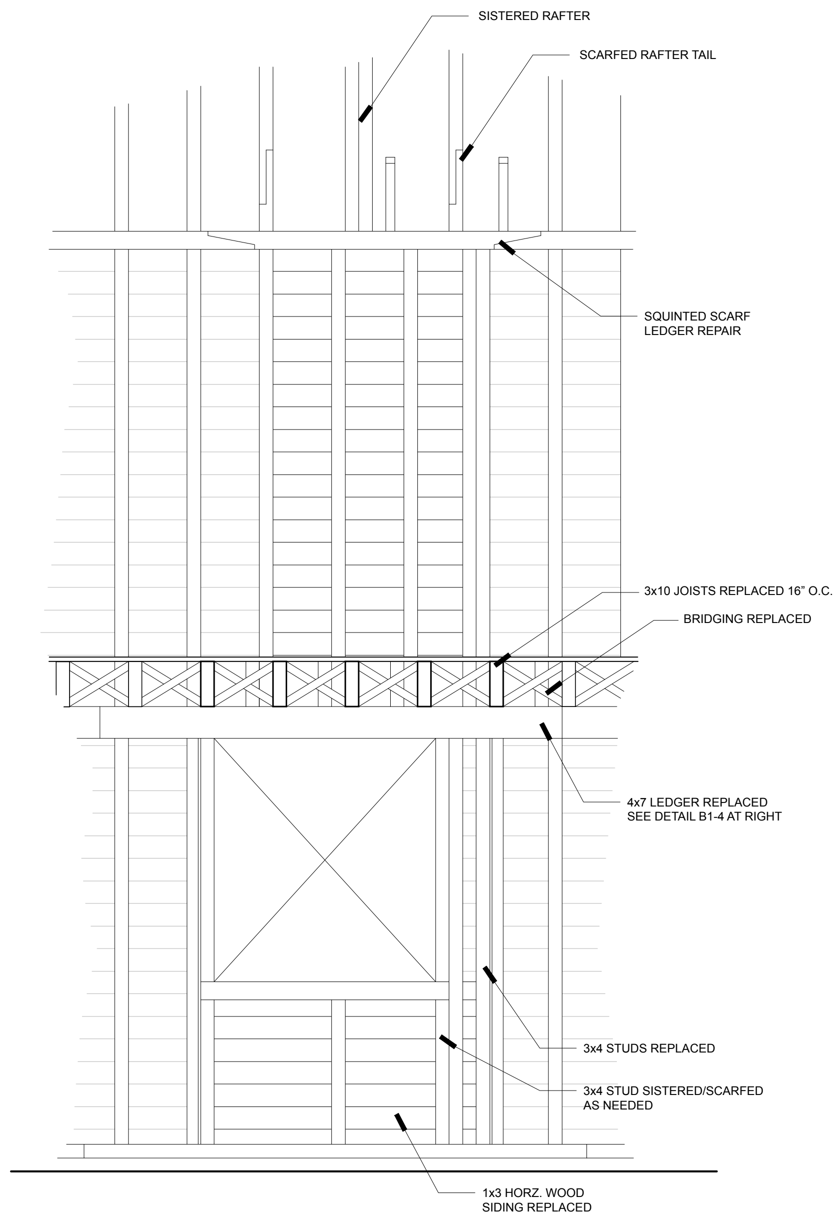
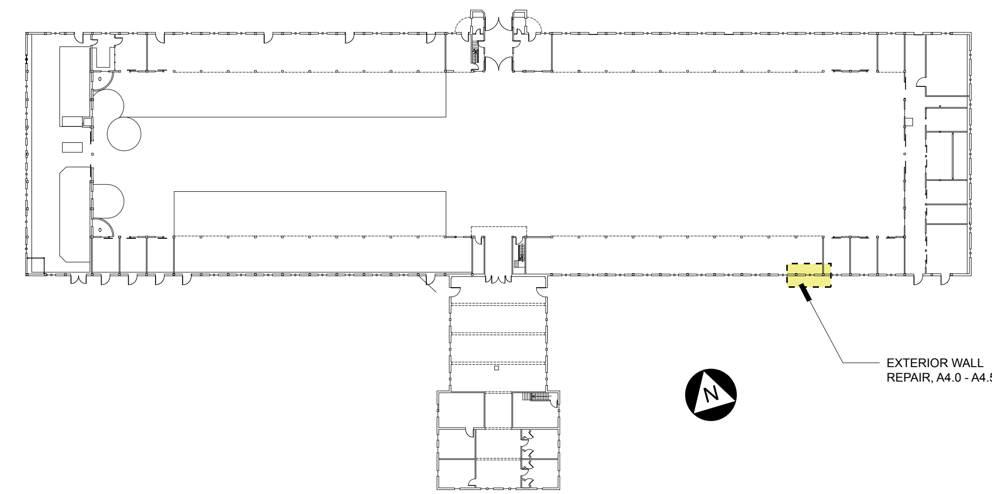
THE STRUCTURAL REPAIR OF THE BREEDING BARN AT
SHELBURNE FARMS
1611 HARBOR ROAD, SHELBURNE, VT 05482

30 MARCH, 2011

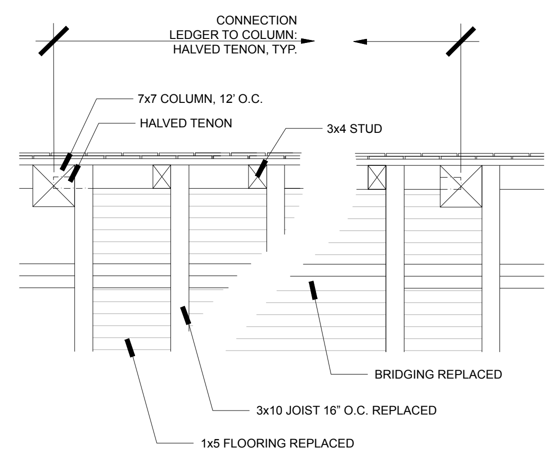
2



A1 3 EXTERIOR WALL REPAIR, A4.0 - A4.5, ELEVATION
SCALE: 3/4" = 1'-0"



B2 3 EXT. WALL REPAIR, KEYED FIRST FLOOR PLAN
SCALE: 1/32" = 1'-0"



B1 3 LEDGER REPAIR, PLAN DETAIL *WORM'S EYE VIEW
SCALE: 1" = 1'-0"

NOTES

-REPAIR TYPICALS:

- NEW LOCUST SILLS
- COLUMN SCARFS AT EXISTING SCARF LOCATIONS
- STUD SISTERS/SCARFS AS NEEDED
- SCARF REPAIRS OF AISLE FLOOR LEDGER AS NEEDED
- REPLACEMENT IN-KIND OF WALL SHEATHING AS NEEDED
- 3x10 FLOOR JOISTS REPLACED AS NEEDED
- JOIST BRIDGING REPLACED AS NEEDED

-SEE SHEET 20 FOR GENERAL REPAIR DETAILS

-SEE SHEET 2 FOR FOUNDATION REPAIR DETAILS

-SEE SHEETS 4-5 FOR AISLE FLOOR REPAIR DETAILS

Shelburne Farms Breeding Barn
Exterior Wall Repairs, Section and Keyed First Floor Plan

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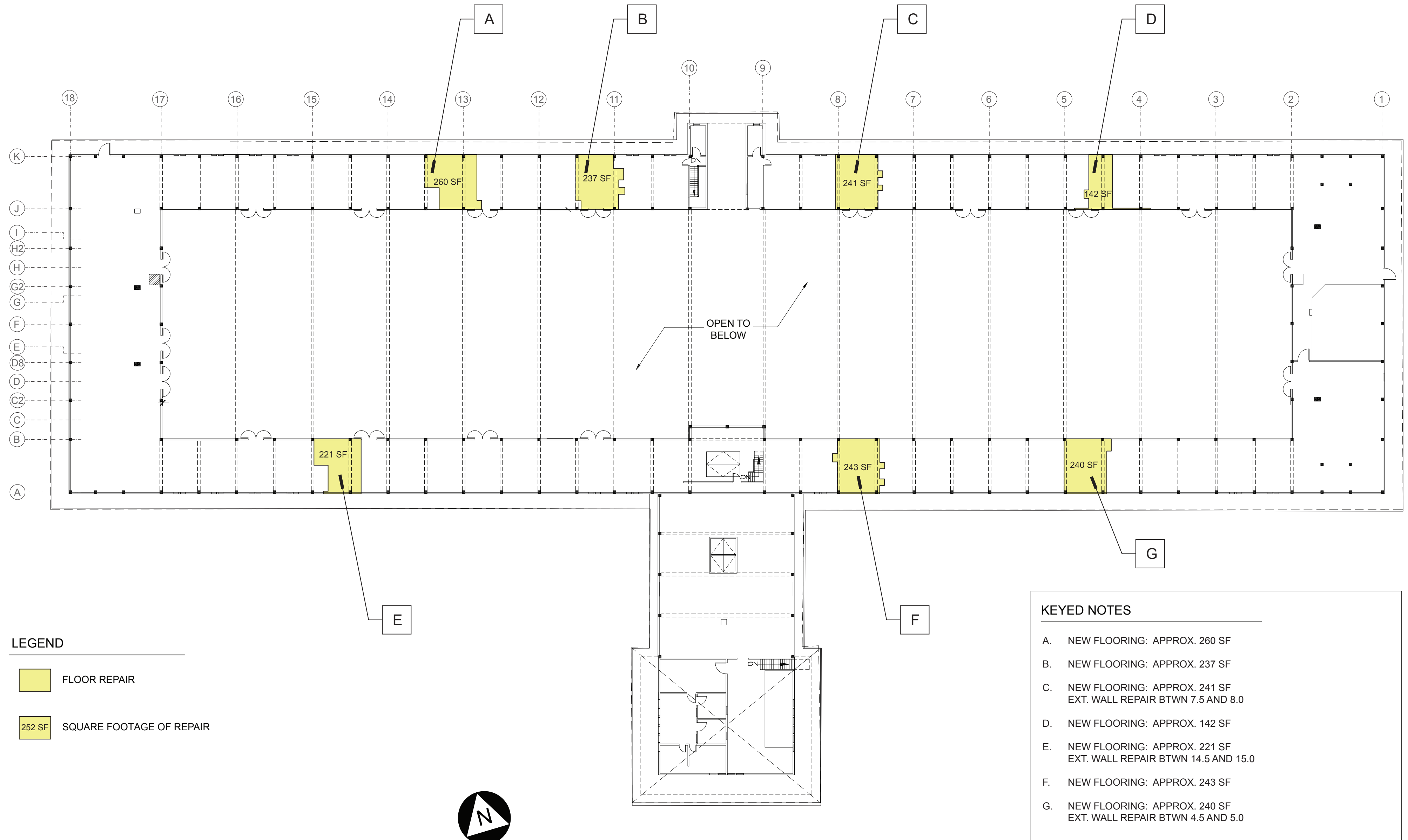
DRAWINGS PREPARED BY: KERI L. STEVENSON

30 MARCH, 2011

3



LEGEND

FLOOR REPAIR

252 SF SQUARE FOOTAGE OF REPAIR

KEYED NOTES

A. NEW FLOORING: APPROX. 260 SF

B. NEW FLOORING: APPROX. 237 SF

C. NEW FLOORING: APPROX. 241 SF
EXT. WALL REPAIR BTWN 7.5 AND 8.0

D. NEW FLOORING: APPROX. 142 SF

E. NEW FLOORING: APPROX. 221 SF
EXT. WALL REPAIR BTWN 14.5 AND 15.0

F. NEW FLOORING: APPROX. 243 SF

G. NEW FLOORING: APPROX. 240 SF
EXT. WALL REPAIR BTWN 4.5 AND 5.0

-SEE SHEET 5 FOR FLOOR REPAIR DETAILS

-SEE SHEET 3 FOR EXTERIOR WALL REPAIR DETAILS

Shelburne Farms Breeding Barn

Aisle Floor Repairs, Keyed Second Floor Plan

SCALE: 1/16" = 1'-0"
UNLESS OTHERWISE NOTED
0 4 10 20 40

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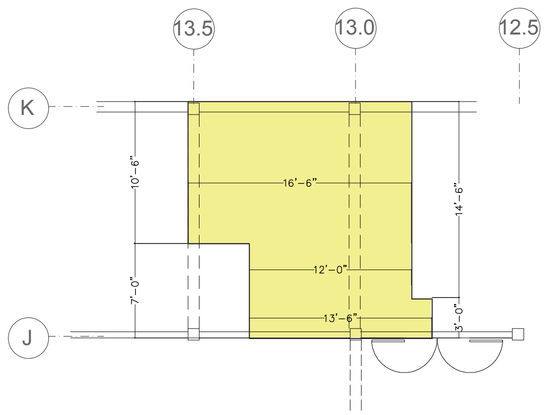
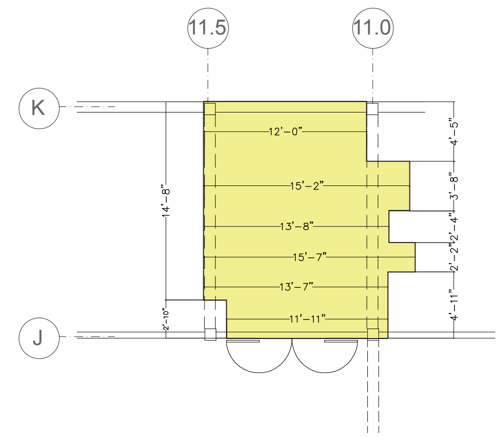
DRAWINGS PREPARED BY: KERI L. STEVENSON

30 MARCH, 2011

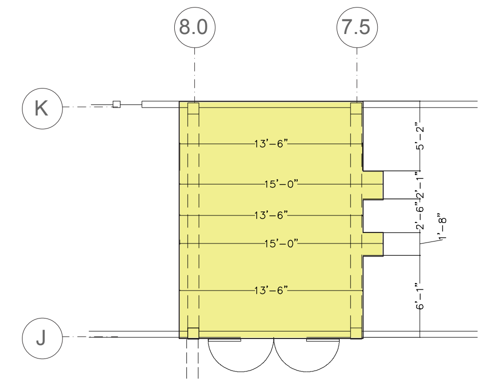
4



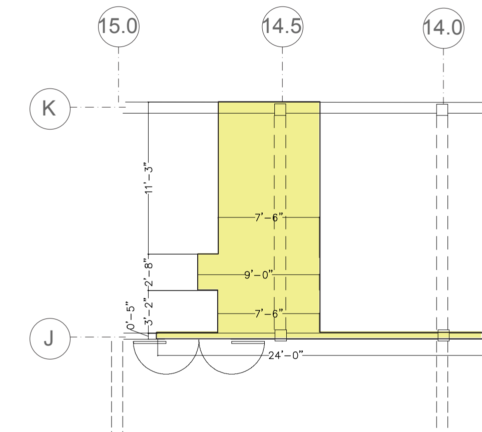
A REPAIR AREA, 12.5 - 13.5 JK
APPROX. 260 SF



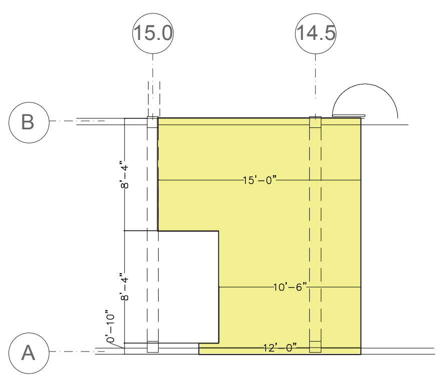
B REPAIR AREA, 11.0 - 11.5 JK
APPROX. 237 SF



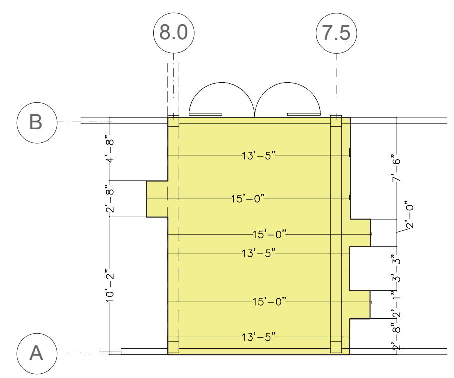
C REPAIR AREA, 7.5 - 8.0 JK
APPROX. 241 SF



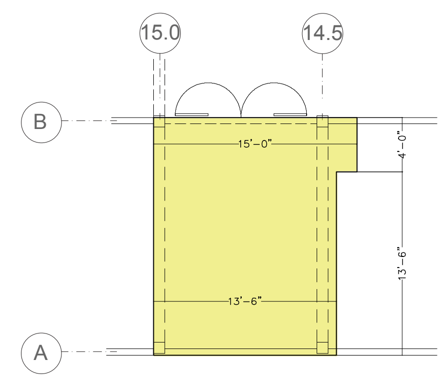
D REPAIR AREA, 3.5 - 5.0 JK
APPROX. 142 SF



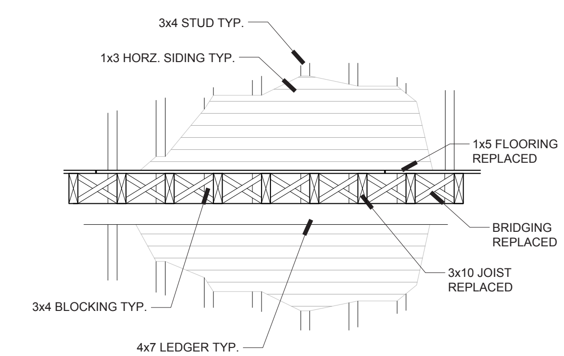
E REPAIR AREA, 14.5 - 15.0 AB
APPROX. 221 SF



F REPAIR AREA, 7.5 - 8.0 AB
APPROX. 243 SF



G REPAIR AREA, 14.5 - 15.0 AB
APPROX. 221 SF



H SECTION THOUGH TYP. REPAIR
SCALE: 1/2" = 1'-0"

NOTES

-REPAIR TYPICALS:

- SCARF REPAIRS OF AISLE FLOOR LEDGER AS NEEDED
- REPLACEMENT IN-KIND OF JOISTS AND BRIDGING AS NEEDED
- REPLACEMENT IN-KIND OF FLOOR DECKING AS NEEDED

Shelburne Farms Breeding Barn
Aisle Floor Repairs, Dimensioned Repair Details

SCALE: 3/16" = 1'-0"
UNLESS OTHERWISE NOTED
0 2 4 10 20

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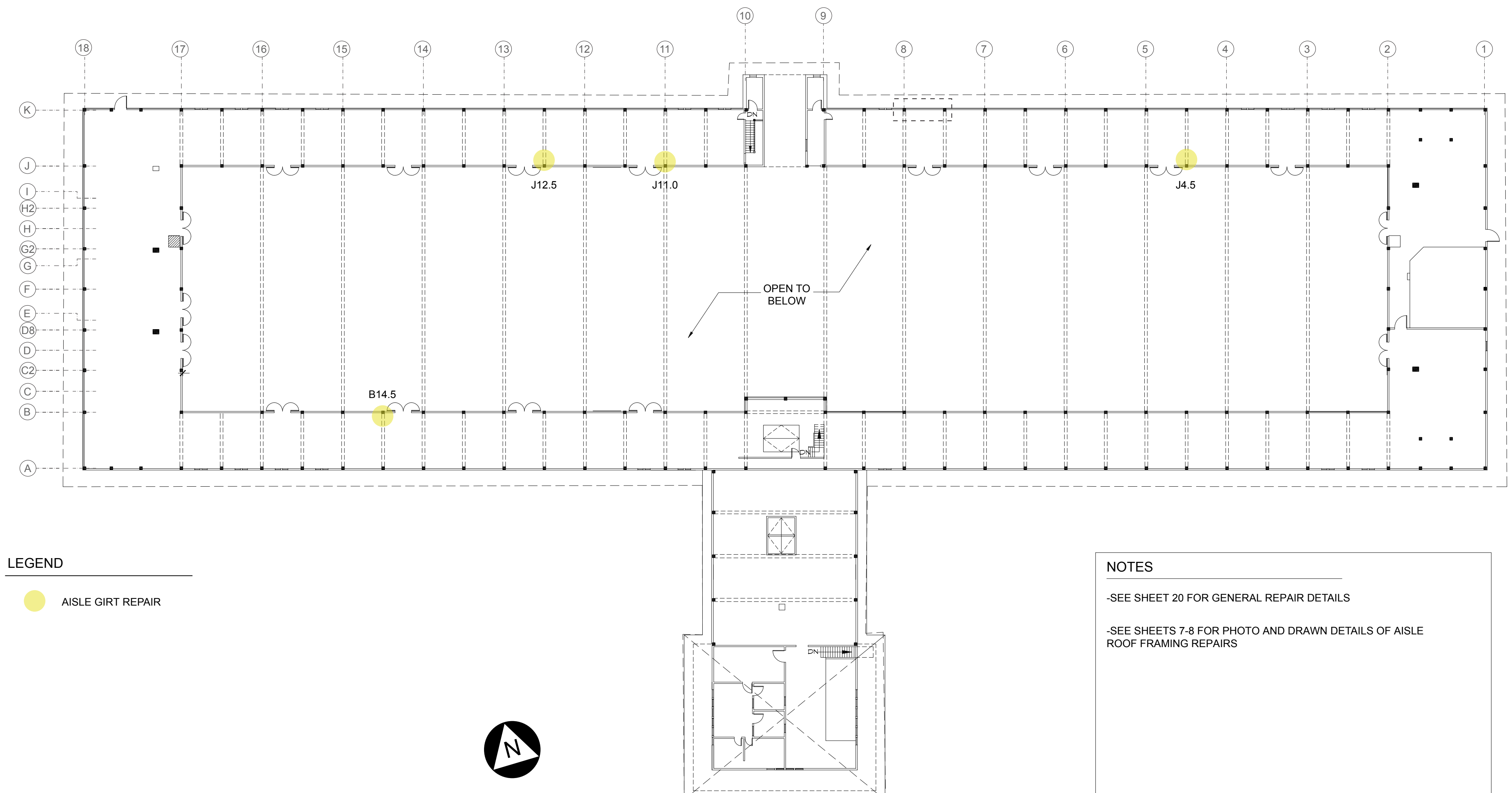
30 MARCH, 2011

5



LEGEND

AISLE GIRT REPAIR

NOTES

-SEE SHEET 20 FOR GENERAL REPAIR DETAILS

-SEE SHEETS 7-8 FOR PHOTO AND DRAWN DETAILS OF AISLE ROOF FRAMING REPAIRS

Shelburne Farms Breeding Barn
Aisle Roof Framing Repairs, Keyed Second Floor Plan

SCALE: 1/16" = 1'-0"
UNLESS OTHERWISE NOTED
0 4 10 20 40

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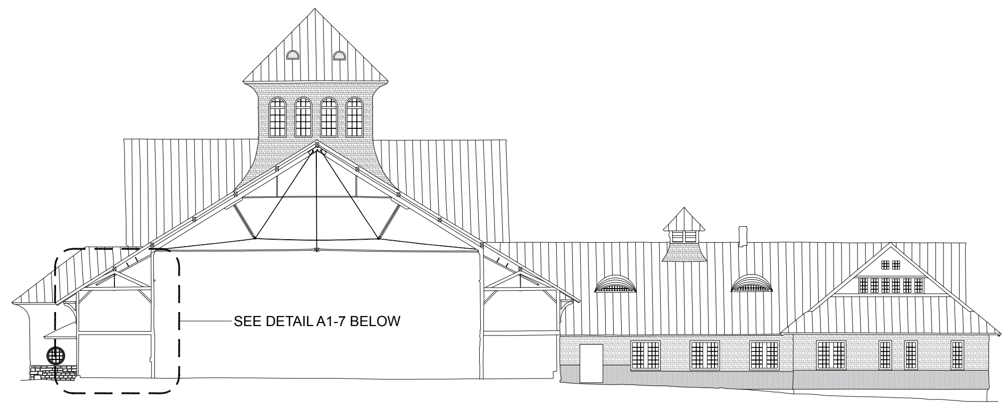
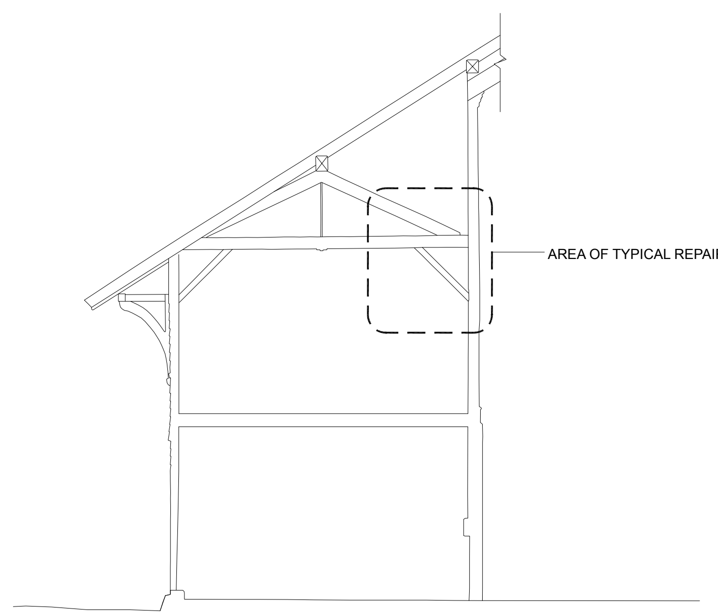
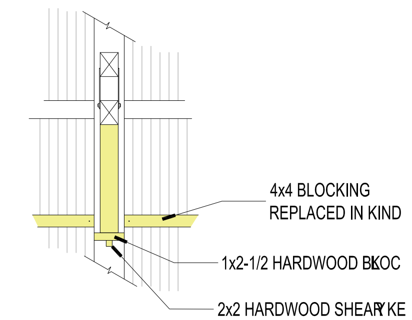
6



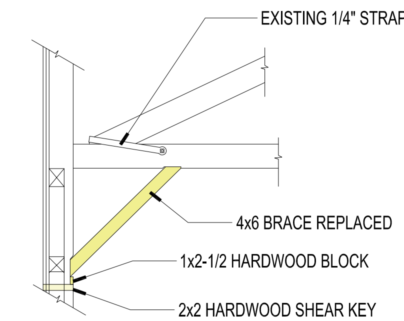
A2/7 TRANVERSE SECTION OF BREEDING BARN
SCALE: 1/16" = 1'-0"



A1/7 DETAIL OF A2-7 ABOVE: AREA OF TYP. REPAIR
SCALE: 1/4" = 1'-0"



B14.5 SOUTH ELEVATION
SCALE: 1/2" = 1'-0"



B14.5 WEST ELEVATION
SCALE: 1/2" = 1'-0"

B14.5 PHOTO DETAIL, SOUTH
N.T.S.

B14.5 PHOTO DETAIL, WEST
N.T.S.

LEGEND

ROOF FRAMING REPAIR

Shelburne Farms Breeding Barn
Aisle Roof Framing Repairs, Details, 1 of 2

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THE STRUCTURAL REPAIR OF THE BREEDING BARN AT
SHELBURNE FARMS
1611 HARBOR ROAD, SHELBURNE, VT 05482

DRAWINGS PREPARED BY: KERI L. STEVENSON

30 MARCH, 2011

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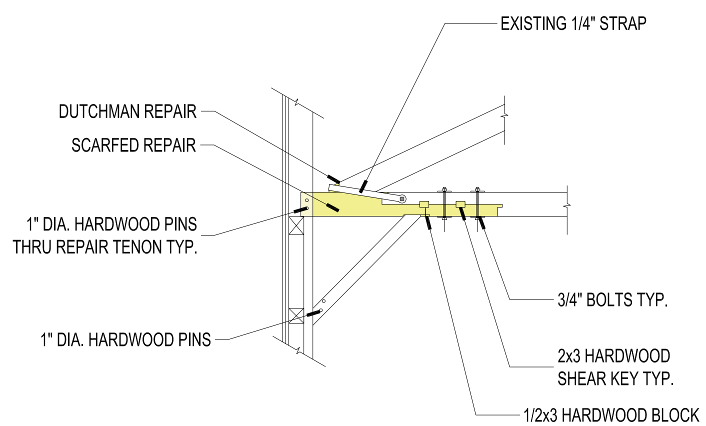
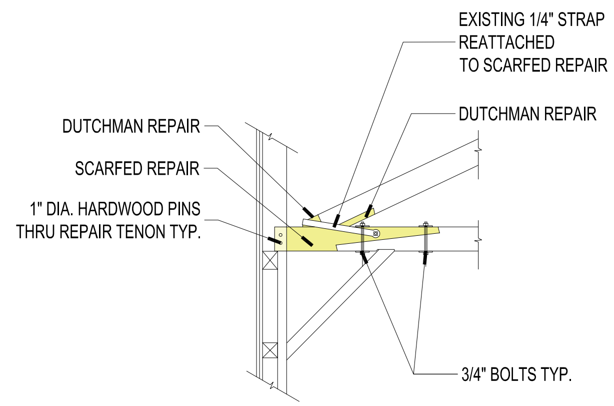
LEGEND

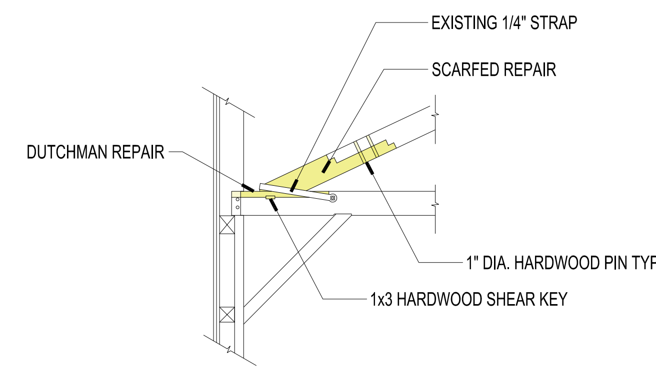
ROOF FRAMING REPAIR



J11.0 EAST ELEVATION
SCALE: 1/2" = 1'-0"



J4.5 EAST ELEVATION
SCALE: 1/2" = 1'-0"



J12.5 EAST ELEVATION
SCALE: 1/2" = 1'-0"

J11.0 PHOTO DETAIL, EAST
N.T.S.

J11.0 PHOTO DETAIL, WEST
N.T.S.

J4.5 PHOTO DETAIL, EAST
N.T.S.

J4.5 PHOTO DETAIL, WEST
N.T.S.

J12.5 PHOTO DETAIL, EAST
N.T.S.

J12.5 PHOTO DETAIL, WEST
N.T.S.

Shelburne Farms Breeding Barn
Aisle Roof Framing Repairs, Details, 2 of 2

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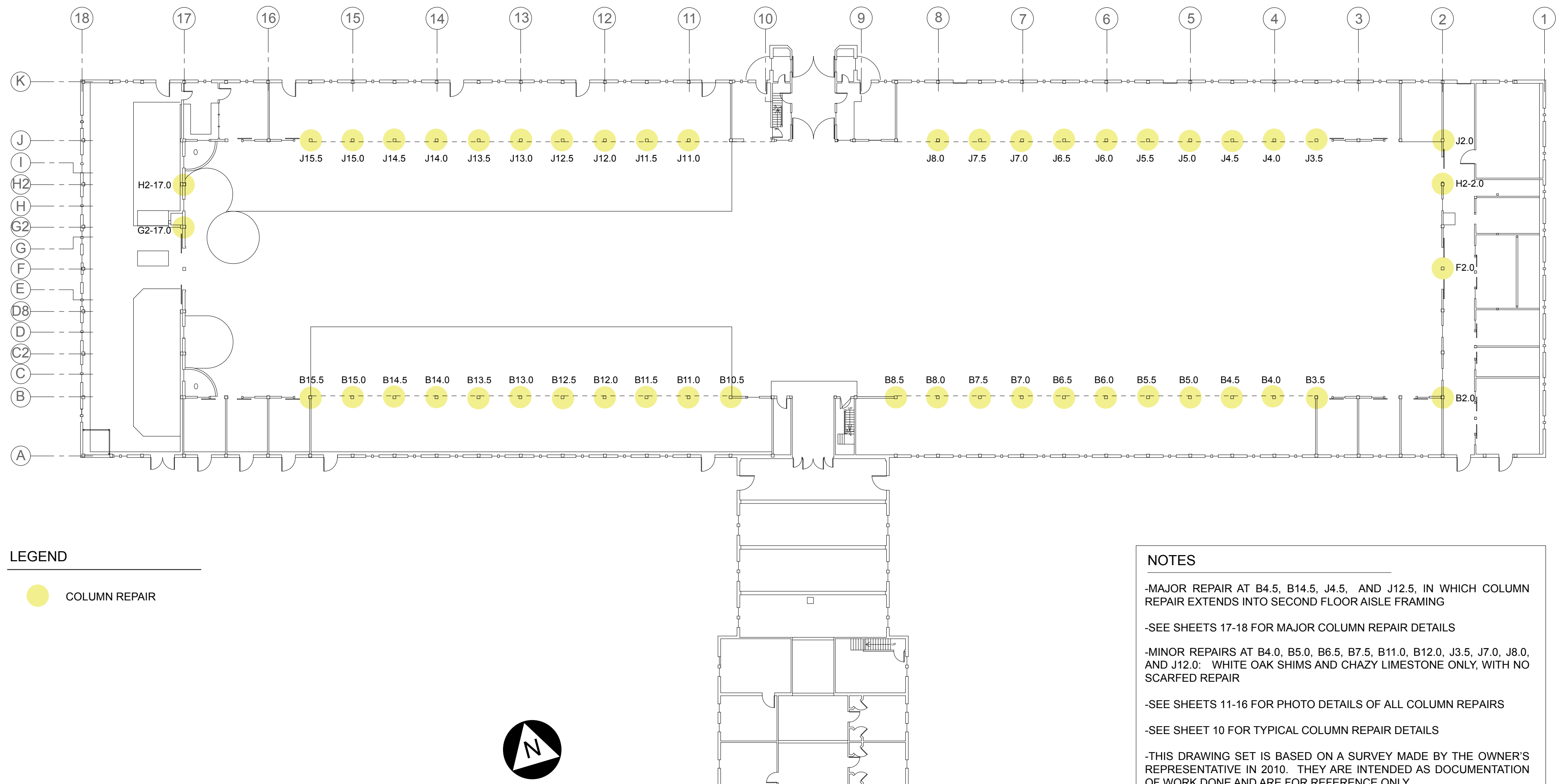

LEGEND

COLUMN REPAIR

NOTES

-MAJOR REPAIR AT B4.5, B14.5, J4.5, AND J12.5, IN WHICH COLUMN REPAIR EXTENDS INTO SECOND FLOOR AISLE FRAMING

-SEE SHEETS 17-18 FOR MAJOR COLUMN REPAIR DETAILS

-MINOR REPAIRS AT B4.0, B5.0, B6.5, B7.5, B11.0, B12.0, J3.5, J7.0, J8.0, AND J12.0: WHITE OAK SHIMS AND CHAZY LIMESTONE ONLY, WITH NO SCARFED REPAIR

-SEE SHEETS 11-16 FOR PHOTO DETAILS OF ALL COLUMN REPAIRS

-SEE SHEET 10 FOR TYPICAL COLUMN REPAIR DETAILS

-THIS DRAWING SET IS BASED ON A SURVEY MADE BY THE OWNER'S REPRESENTATIVE IN 2010. THEY ARE INTENDED AS DOCUMENTATION OF WORK DONE AND ARE FOR REFERENCE ONLY.

Shelburne Farms Breeding Barn
Column Repairs, Keyed First Floor Plan

SCALE: 1/16" = 1'-0"
UNLESS OTHERWISE NOTED
0 4 10 20 40

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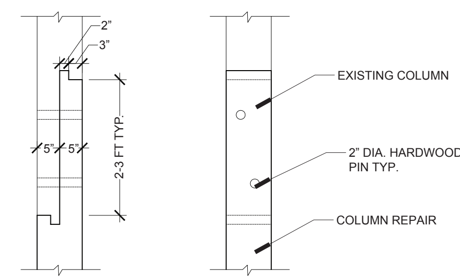
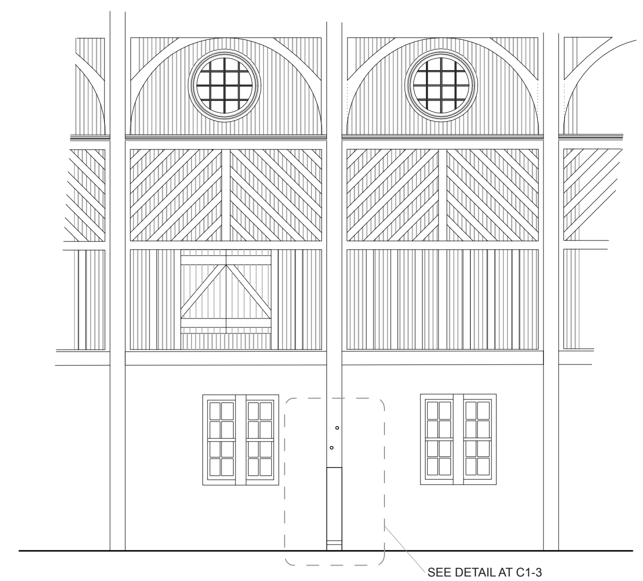
POST NO.	DESCRIPTION	REPAIR LENGTH*
B2.0	COVERED UP WITH METAL, ASSUME 8" OF DECAY	0 (12")
B2.5	GOOD CONDITION, FULL LENGTH	NO REPAIR
B3.0	GOOD CONDITION, FULL LENGTH	NO REPAIR
B3.5	DECAYED AT BASE, UP 10"	0 (16")
B4.0	DECAYED AT BASE, UP 6"	S + CL ONLY
B4.5	POST BOTTOM HAS BEEN REPLACED BUT THE LENGTH IS 2" TOO SHORT	117" (153")
B5.0	SOFT ON THE OUTSIDE UP 6", NO DECAY	S + CL ONLY
B5.5	DECAYED AT THE BASE, UP 6"	40" (69")
B6.0	DECAYED AT THE BASE, UP 10"	35" (65")
B6.5	DECAYED AT THE BASE, UP 10"	30" (63")
B7.0	SOFT ON THE OUTSIDE UP 2", FAIR CONDITION	S + CL ONLY
B7.5	DECAYED ON BOTTOM, CORNERS BROKEN OFF, REPAIR WITH 6FT LONG NEW TIMBER	32" (62")
B8.0	DECAYED AT THE BASE, UP 8"	S + CL ONLY
B8.5	DECAYED AT THE BASE, UP 8"	0" (27")
B9.0	GOOD CONDITION, FULL LENGTH	NO REPAIR
B9.5a	EXCELLENT CONDITION, FULL LENGTH	NO REPAIR
B9.5b	EXCELLENT CONDITION, FULL LENGTH	NO REPAIR
B10.0	EXCELLENT CONDITION, FULL LENGTH	NO REPAIR
B10.5	DECAYED AT THE BASE, UP 10"	20" (36")
B11.0	DECAYED, UP 8"	S + CL ONLY
B11.5	DECAYED ON BOTTOM, CORNERS BROKEN OFF, REPAIR WITH 5FT LONG NEW TIMBER	24" (60")
B12.0	DECAYED AT THE BASE, UP 6"	S + CL ONLY
B12.5	DECAYED AT THE BASE, UP 8"	0 (18")
B13.0	DECAYED AT THE BASE, UP 8"	36" (56")
B13.5	DECAYED AT THE BASE, UP 8"	37" (57")
B14.0	DECAYED AT THE BASE, UP 8"	36" (56")
B14.5	POST BOTTOM HAS BEEN REPLACED BUT THE LENGTH IS 2" TOO SHORT	156" (192")
B15.0	DECAYED AT THE BASE, UP 8"	38" (58")
B15.5	DECAYED AT THE BASE, UP 10"	0 (24")
B16.0	GOOD CONDITION, FULL LENGTH	NO REPAIR
B16.5	GOOD CONDITION, FULL LENGTH	NO REPAIR
B17.0	COVERED UP WITH METAL, ASSUME 4" TO 6" OF DECAY	NO REPAIR
C2-17.0	GOOD CONDITION, FULL LENGTH	NO REPAIR
D8-17.0	FAIR CONDITION, FULL LENGTH	NO REPAIR
F17.0	FAIR CONDITION, FULL LENGTH	NO REPAIR
G2-17.0	DECAYED AT THE BASE, UP 10"	20" (48")
H2-17.0	DECAYED AT THE BASE, UP 12"	9" (16")
J17.0	COVERED WITH METAL, ASSUME FAIR CONDITION, NO DECAY	NO REPAIR
J16.5	GOOD CONDITION, FULL LENGTH	NO REPAIR
J16.0	GOOD CONDITION, FULL LENGTH	NO REPAIR
*REPAIR LENGTH IS NOTED AS "HEIGHT OF BOTTOM SHOULDER OF REPAIR(HEIGHT OF TOP SHOULDEROF REPAIR)		
** "S + CL ONLY" DENOTES THAT REPAIR CONSISTS OF WHITE OAK SHIMS AND CHAZY LIMESTONE AT COLUMN BASE ONLY		

POST NO.	DESCRIPTION	REPAIR LENGTH*
J15.5	DECAYED AT BASE, UP 8"	29" (52")
J15.0	DECAYED AT BASE, UP 8"	39" (66")
J14.5	DECAYED ON BOTTOM, CRACKED ON CORNERS, REPAIR WITH 6FT LONG NEW TIMBER	40" (67")
J14.0	DECAYED ON BOTTOM, CRACKED ON CORNERS, REPAIR WITH 7FT LONG NEW TIMBER	41" (61")
J13.5	DECAYED ON BOTTOM, CRACKED ON CORNERS, REPAIR WITH 6FT LONG NEW TIMBER	39" (59")
J13.0	DECAYED AT BASE, UP 6"	53" (32")
J12.5	POST BOTTOM HAS BEEN REPLACED BUT THE LENGTH IS 1" TOO SHORT	118" (154")
J12.0	DECAYED AT BASE, UP 6"	S + CL ONLY
J11.5	DECAYED AT BASE, UP 8"	44" 64")
J11.0	DECAYED AT BASE, UP 2"	59" (37")
J10.5	POST HAS BEEN CUT OFF AT BOTTOM, 1'-9" SHORTER THAN FULL LENGTH, CONCRETE UNDERNEATH	NO REPAIR
J10.0	DECAYED AT BASE, UP 4"	NO REPAIR
J9.5a	FAIR CONDITION, FULL LENGTH	NO REPAIR
J9.5b	FAIR CONDITION, FULL LENGTH	NO REPAIR
J9.0	GOOD CONDITION, FULL LENGTH	NO REPAIR
J8.5	DECAYED AT BASE, UP 8"	NO REPAIR
J8.0	DECAYED AT BASE, UP 8"	S + CL ONLY
J7.5	DECAYED AT BASE, UP 10"	57" (35")
J7.0	DECAYED AT BASE, UP 8"	S + CL ONLY
J6.5	DECAYED, UP 8"	35" (56")
J6.0	DECAYED AT BASE, UP 10"	24" (45")
J5.5	DECAYED ON BOTTOM, CRACKED ON CORNERS, REPAIR WITH 4FT LONG NEW TIMBER	44" (64")
J5.0	DECAYED AT BASE, UP 8"	62" (40")
J4.5	POST BOTTOM HAS BEEN REPLACED	117" (153")
J4.0	FAIR CONDITION, FULL LENGTH	S + CL ONLY
J3.5	DECAYED AT BASE, UP 10"	0 (31")
J3.0	SOFT ON OUTSIDE UP 4" BUT GOOD CONDITION, FULL LENGTH	NO REPAIR
J2.5	GOOD CONDITION, FULL LENGTH	NO REPAIR
J2.0	GOOD CONDITION, FULL LENGTH	0 (16")
H8-2.0	FAIR CONDITION, FULL LENGTH	NO REPAIR
G2-2.0	GOOD CONDITION, FULL LENGTH	NO REPAIR
F2.0	DECAYED AT BASE, UP 2"	0 (14")
D8-2.0	EXCELLENT CONDITION, FULL LENGTH	NO REPAIR
C2-2.0	EXCELLENT CONDITION, FULL LENGTH	NO REPAIR
A1.0-A3.5	SILLS ARE DECAYED BUT POSTS ARE GOOD	SEE SHEET 11
A4.0-A9.0	CONCRETE PLACED 2' UP ON INSIDE OF POSTS AND SILLS, ASSUME SOME DECAY	SEE SHEET 11
A10.0-A15.0	CONCRETE PLACED 2' UP ON INSIDE OF POSTS AND SILLS, ASSUME SOME DECAY	SEE SHEET 11
A15.5-A17.5	SLAB FLOOR CONCRETE OVER SILLS AGAINST POSTS, ASSUME SILLS DECAYED SOME POST DECAY	SEE SHEET 11
K17.5-K10.0	CONCRETE PLACED OVER SILL ONTO POSTS, ASSUME SILLS ARE DECAYED, SOME POST DECAY	SEE SHEET 11
K9.0-K1.0	REPAIRED WITH (2X) 2x6 PT SILL PLATES WITH SILL SEAL AND NEW CONCRETE, SOME DECAY ON POSTS	SEE SHEET 11
* REPAIR LENGTH IS NOTED AS "HEIGHT OF BOTTOM SHOULDER OF REPAIR(HEIGHT OF TOP SHOULDEROF REPAIR)		
** "S + CL ONLY" DENOTES THAT REPAIR CONSISTS OF WHITE OAK SHIMS AND CHAZY LIMESTONE AT COLUMN BASE ONLY		

A1 / 10 TABLE OF COLUMN REPAIR DETAILS



B1 / 10 TYPICAL COLUMN REPAIR
SCALE: 3/4" = 1'-0"

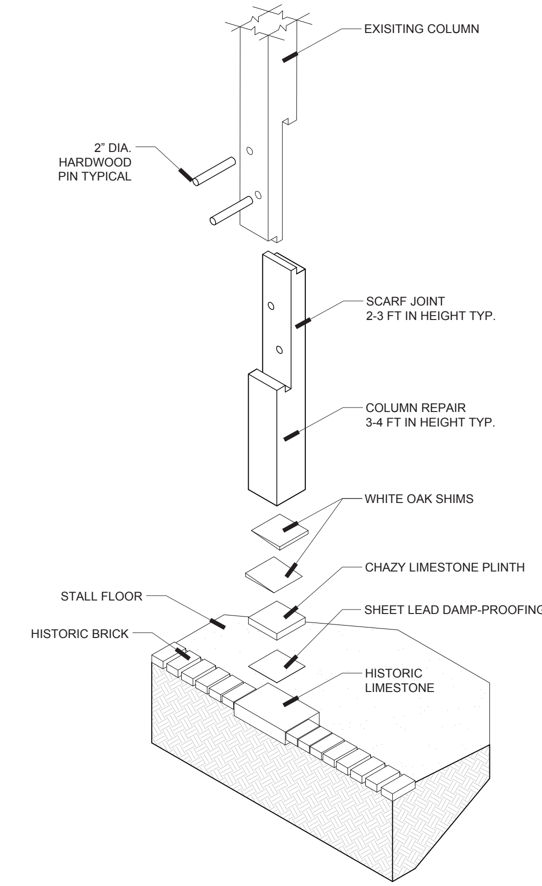


B1 / 10 AREA OF TYPICAL COLUMN REPAIR
SCALE: 1/4" = 1'-0"

NOTES

-REPAIR TYPICALS:

- NEW COLUMN BASES SCARFED INTO EXISTING COLUMNS TO REPLACE DECAYED MATERIAL AND TO ELEVATE THE COLUMN BASE
- THE SCARF FORM USED FOR MOST OF THE REPAIRS REPLICATES AN HISTORIC FORM FOUND IN THE BUILDING
- REPLACEMENT PIECES TYPICALLY AT LEAST 2 FEET LONG BELOW THE LOWEST SHOULDER, WITH BLADES AT LEAST 2 FEET LONG TO RESIST BUCKLING
- BLADES PINNED WITH 2" DIA. HARDWOOD PINS
- ADDITION OF A LIMESTONE PLINTH WITH SHEET LEAD DAMP-PROOFING AND WHITE OAK SHIMS



C1 / 10 DETAIL OF B1-3, TYP. COLUMN REPAIR: EXPLODED ISOMETRIC VIEW
SCALE: 3/4" = 1'-0"

Shelburne Farms Breeding Barn
Column Repairs, Repair Details

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SHELBURNE FARMS
1611 HARBOR ROAD, SHELBURNE, VT 05482

30 MARCH, 2011

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B2.0 PHOTO DETAILS
N.T.S.

B3.5 PHOTO DETAILS
N.T.S.

B4.0 PHOTO DETAILS
N.T.S.

B4.5 PHOTO DETAILS
N.T.S.

B5.0 PHOTO DETAILS
N.T.S.

B5.5 PHOTO DETAILS
N.T.S.

B6.0 PHOTO DETAILS
N.T.S.

B6.5 PHOTO DETAILS
N.T.S.

B7.0 PHOTO DETAILS
N.T.S.

Shelburne Farms Breeding Barn
Column Repairs, Photo Details, 1 of 6

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30 MARCH, 2011

11

B7.5 PHOTO DETAILS
N.T.S.

B8.0 PHOTO DETAILS
N.T.S.

B8.5 PHOTO DETAILS
N.T.S.

B10.5 PHOTO DETAILS
N.T.S.

B11.0 PHOTO DETAILS
N.T.S.

B11.5 PHOTO DETAILS
N.T.S.

B12.0 PHOTO DETAILS
N.T.S.

B12.5 PHOTO DETAILS
N.T.S.

B13.0 PHOTO DETAILS
N.T.S.

Shelburne Farms Breeding Barn

Column Repairs, Photo Details, 2 of 6

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B13.5 PHOTO DETAILS
N.T.S.

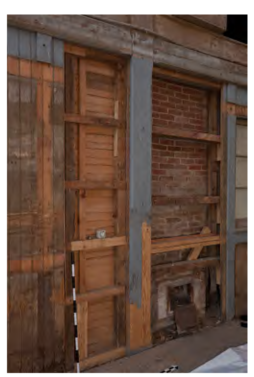
B14.0 PHOTO DETAILS
N.T.S.

B14.5 PHOTO DETAILS
N.T.S.

B15.0 PHOTO DETAILS
N.T.S.

B15.5 PHOTO DETAILS
N.T.S.

F2.0 PHOTO DETAILS
N.T.S.



G2-17.0 PHOTO DETAILS
N.T.S.

H2-2.0 PHOTO DETAILS
N.T.S.

H2-17.0 PHOTO DETAILS
N.T.S.

Shelburne Farms Breeding Barn
Column Repairs, Photo Details, 3 of 6

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J2.0 PHOTO DETAILS
N.T.S.

J3.5 PHOTO DETAILS
N.T.S.

J4.0 PHOTO DETAILS
N.T.S.

J4.5 PHOTO DETAILS
N.T.S.

J5.0 PHOTO DETAILS
N.T.S.

J5.5 PHOTO DETAILS
N.T.S.

J6.0 PHOTO DETAILS
N.T.S.

J6.5 PHOTO DETAILS
N.T.S.

J7.0 PHOTO DETAILS
N.T.S.

Shelburne Farms Breeding Barn
Column Repairs, Photo Details, 4 of 6

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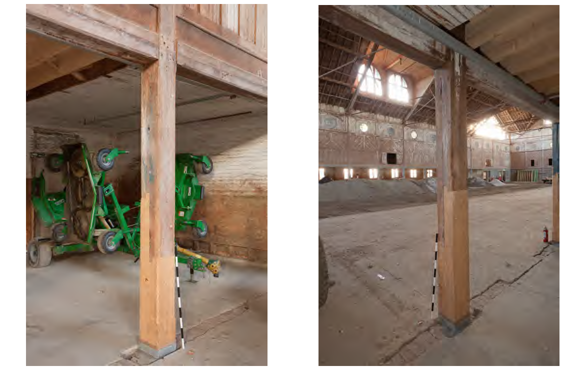
30 MARCH, 2011

14

J7.5 PHOTO DETAILS
N.T.S.

J8.0 PHOTO DETAILS
N.T.S.



J11.0 PHOTO DETAILS
N.T.S.

J11.5 PHOTO DETAILS
N.T.S.

J12.0 PHOTO DETAILS
N.T.S.

J12.5 PHOTO DETAILS
N.T.S.

J13.0 PHOTO DETAILS
N.T.S.

J13.5 PHOTO DETAILS
N.T.S.

J14.0 PHOTO DETAILS
N.T.S.

Shelburne Farms Breeding Barn
Column Repairs, Photo Details, 5 of 6

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30 MARCH, 2011

15

J14.5 PHOTO DETAILS
N.T.S.

J15.0 PHOTO DETAILS
N.T.S.

J15.5 PHOTO DETAILS
N.T.S.

Shelburne Farms Breeding Barn
Column Repairs, Photo Details, 6 of 6

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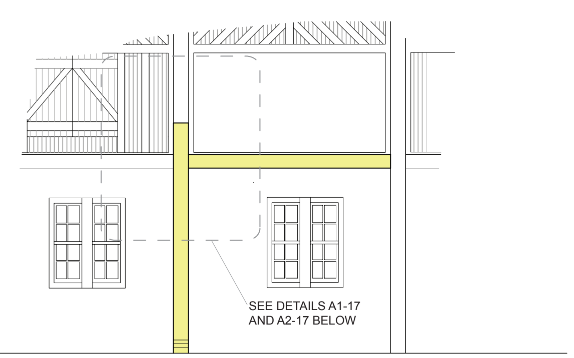
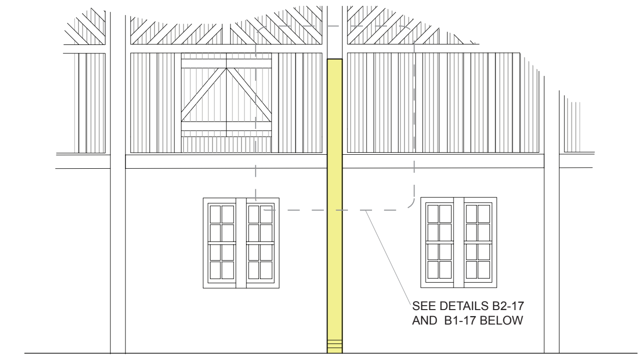
16



A3 17 B4.5: REPAIR IN CONTEXT
SCALE: 1/4" = 1'-0"



B3 17 B14.5: REPAIR IN CONTEXT
SCALE: 1/4" = 1'-0"

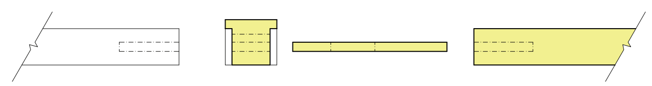
NOTES

-SEE SHEETS 11-16 FOR PHOTO DETAILS OF ALL COLUMN REPAIRS

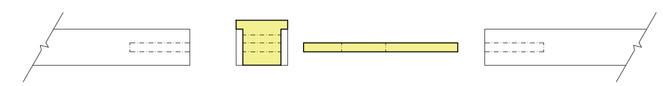
-SEE SHEET 10 FOR TYPICAL COLUMN REPAIR DETAILS

LEGEND

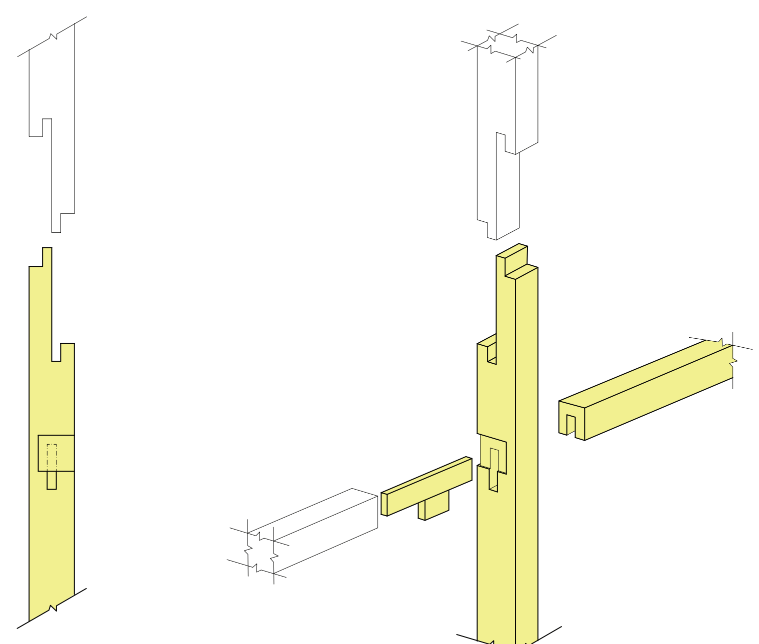
COLUMN REPAIR



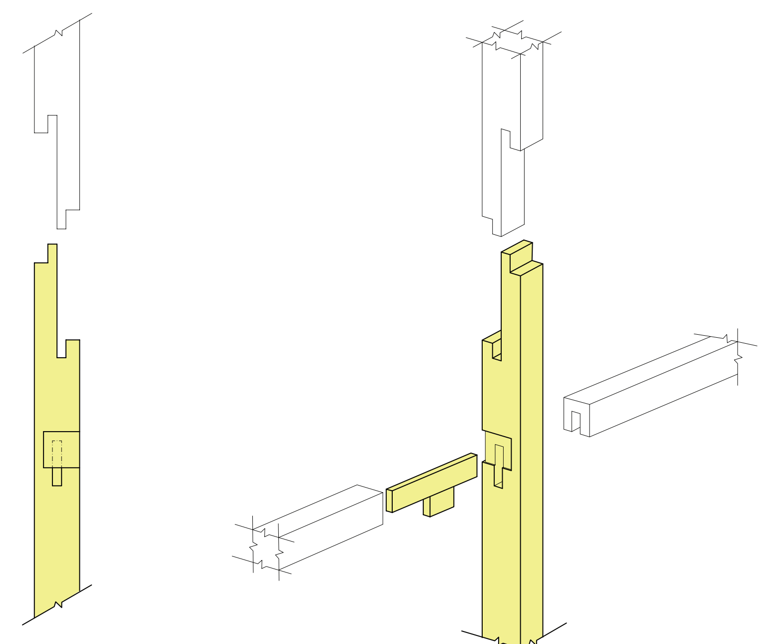
A2 17 B4.5: DETAIL OF REPAIR, PLAN
SCALE: 3/4" = 1'-0"



B2 17 B14.5: DETAIL OF REPAIR, PLAN
SCALE: 3/4" = 1'-0"



A1 17 B4.5: DETAIL OF REPAIR, ELEVATIONS AND AXONOMETRIC
SCALE: 3/4" = 1'-0"



B1 17 B14.5: DETAIL OF REPAIR, ELEVATIONS AND AXONOMETRIC
SCALE: 3/4" = 1'-0"

Shelburne Farms Breeding Barn
Major Column Repairs, Details

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THE STRUCTURAL REPAIR OF THE BREEDING BARN AT
SHELBURNE FARMS
1611 HARBOR ROAD, SHELBURNE, VT 05482

DRAWINGS PREPARED BY: KERI L. STEVENSON

30 MARCH, 2011

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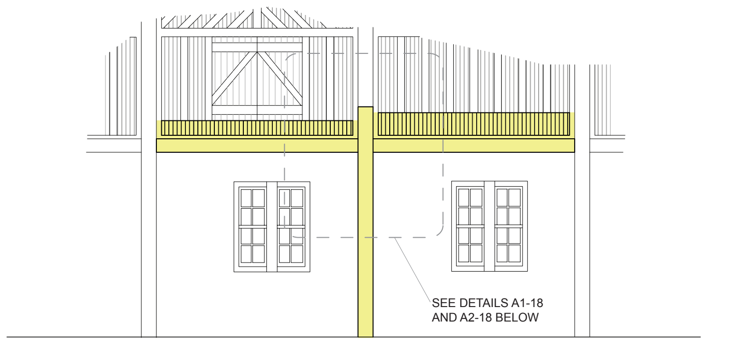


A3 18 J4.5: REPAIR IN CONTEXT
SCALE: 1/4" = 1'-0"

B3 18 J12.5: REPAIR IN CONTEXT
SCALE: 1/4" = 1'-0"

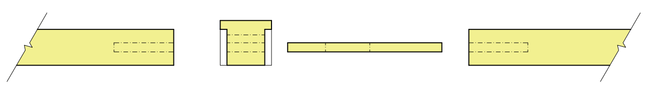
NOTES

-SEE SHEETS 11-16 FOR PHOTO DETAILS OF ALL COLUMN REPAIRS

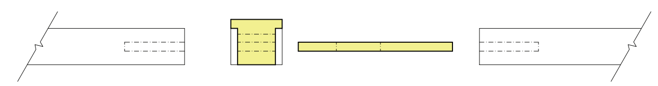
-SEE SHEET 10 FOR TYPICAL COLUMN REPAIR DETAILS

LEGEND

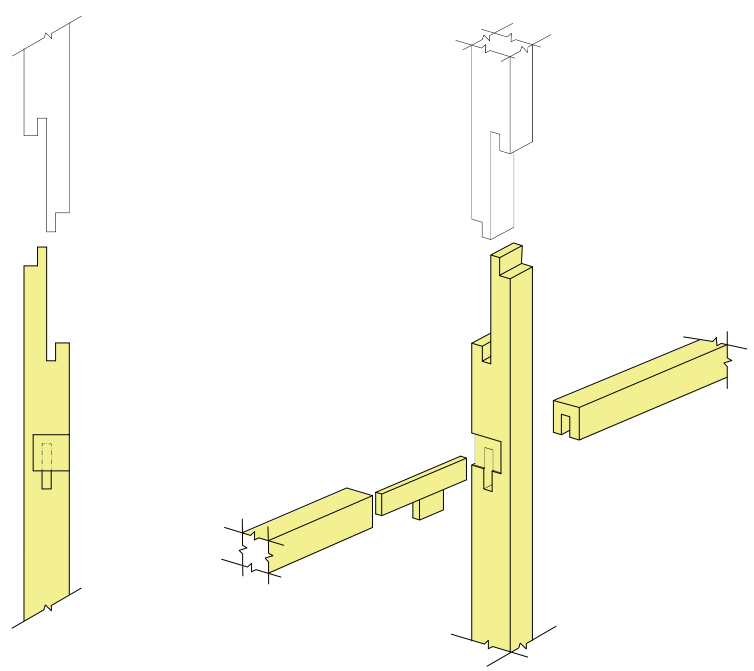
COLUMN REPAIR



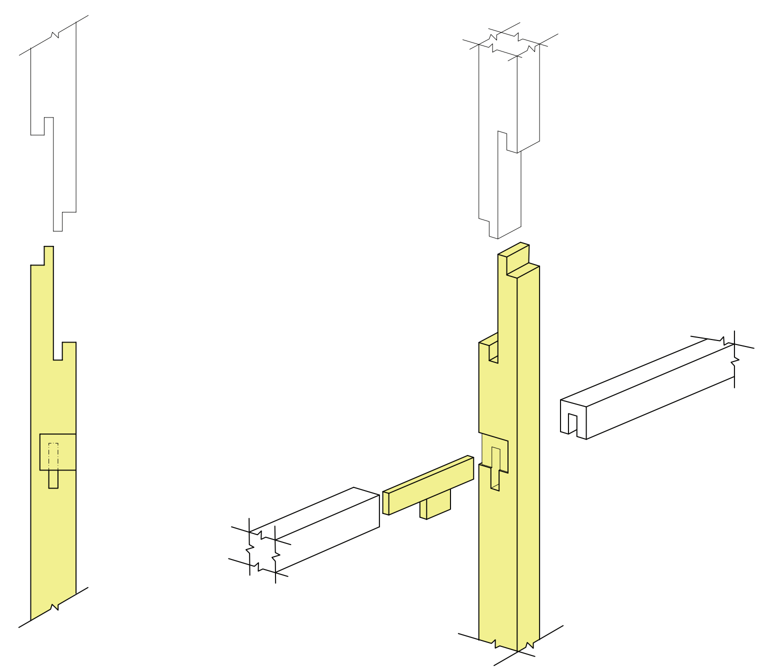
A2 18 J4.5: DETAIL OF REPAIR, PLAN
SCALE: 3/4" = 1'-0"



B2 18 J12.5: DETAIL OF REPAIR, PLAN
SCALE: 3/4" = 1'-0"



A1 18 J4.5: DETAIL OF REPAIR, ELEVATIONS AND AXONOMETRIC
SCALE: 3/4" = 1'-0"



B1 18 J12.5: DETAIL OF REPAIR, ELEVATIONS AND AXONOMETRIC
SCALE: 3/4" = 1'-0"

Shelburne Farms Breeding Barn
Major Column Repairs, Details

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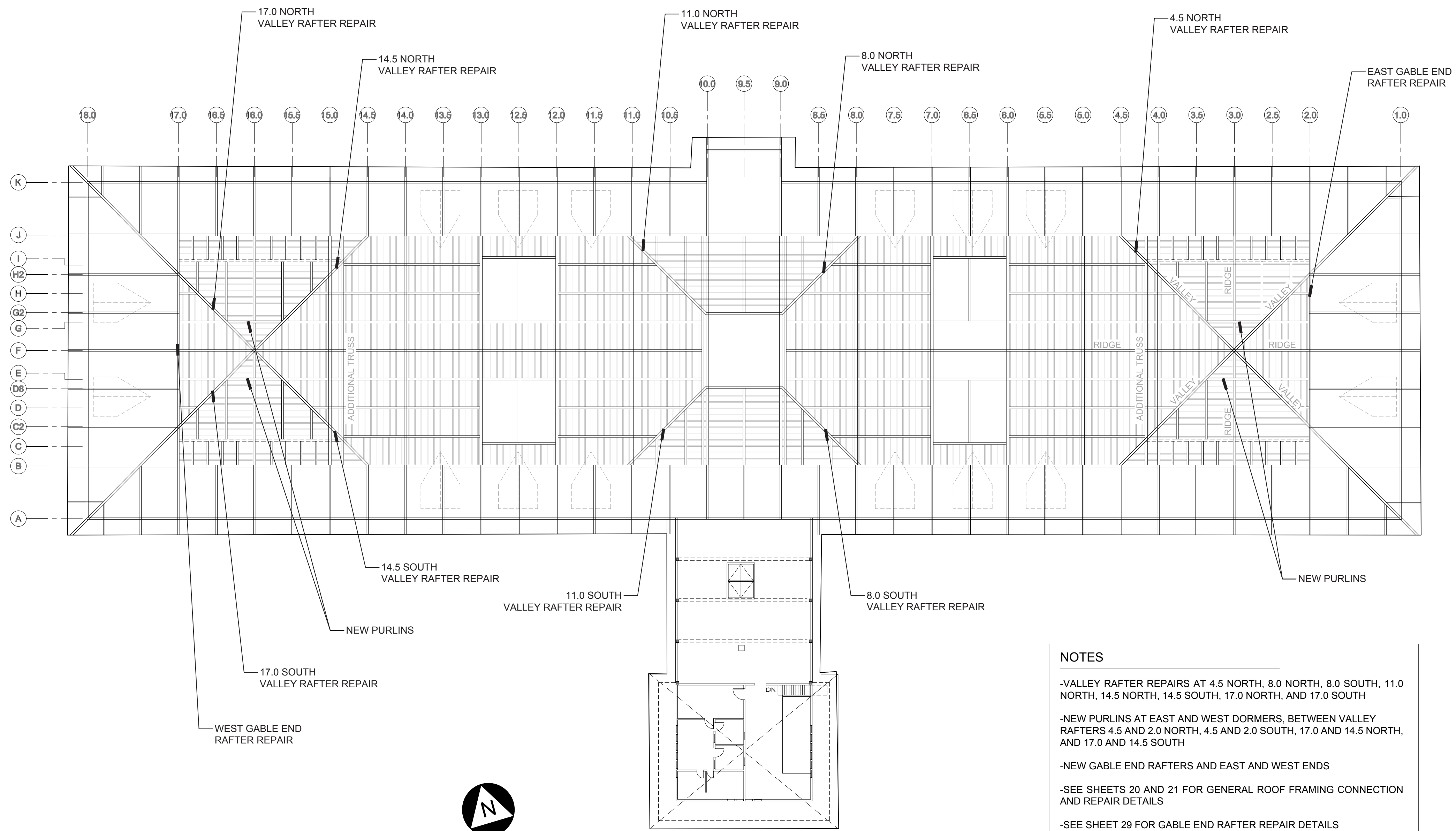

NOTES

-VALLEY RAFTER REPAIRS AT 4.5 NORTH, 8.0 NORTH, 8.0 SOUTH, 11.0 NORTH, 14.5 NORTH, 14.5 SOUTH, 17.0 NORTH, AND 17.0 SOUTH

-NEW PURLINS AT EAST AND WEST DORMERS, BETWEEN VALLEY RAFTERS 4.5 AND 2.0 NORTH, 4.5 AND 2.0 SOUTH, 17.0 AND 14.5 NORTH, AND 17.0 AND 14.5 SOUTH

-NEW GABLE END RAFTERS AND EAST AND WEST ENDS

-SEE SHEETS 20 AND 21 FOR GENERAL ROOF FRAMING CONNECTION AND REPAIR DETAILS

-SEE SHEET 29 FOR GABLE END RAFTER REPAIR DETAILS

-SEE SHEETS 22-28 FOR VALLEY RAFTER REPAIR DETAILS

Shelburne Farms Breeding Barn
Roof Framing Repairs, Keyed Roof Framing Plan

SCALE: 1/16" = 1'-0" UNLESS OTHERWISE NOTED 0 4 10 20 40

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DRAWINGS PREPARED BY: KERI L. STEVENSON

30 MARCH, 2011

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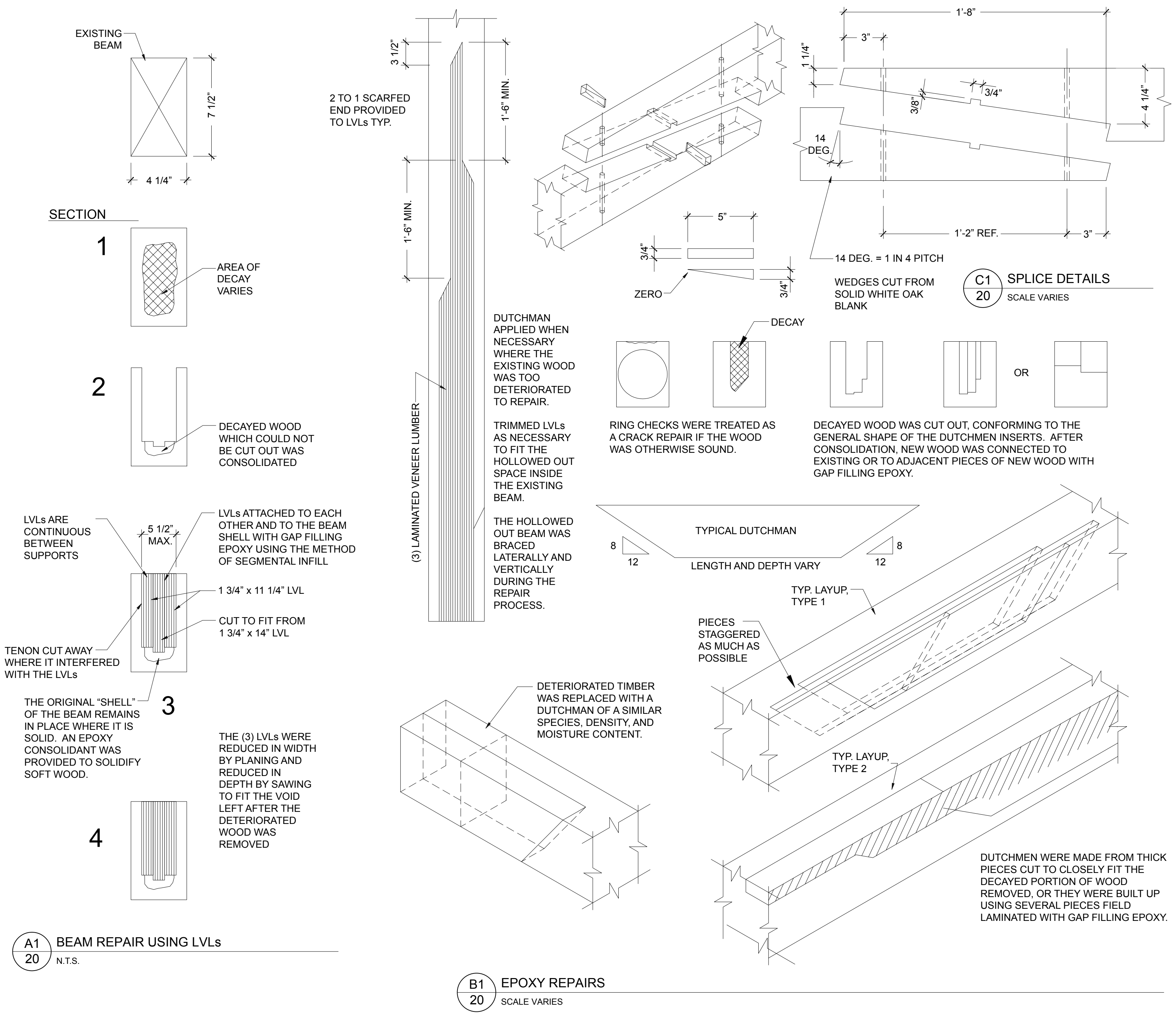


EPOXY REPAIR

REPAIR BY SEGMENTAL INFILL

EPOXY SYSTEMS USED FOR TIMBER REPAIR ARE DIFFERENT THAN THOSE USED FOR CONCRETE.

A PIECE SIMILAR TO A DUTCHMAN INSERT CAN BE USED AS A FILLER. THE ADHESIVE USED CAN BE A GAP FILLING EPOXY TO SECURE A FILLER PIECE MADE FROM WOOD SALVAGED FROM ELSEWHERE IN THE BUILDING.

ALL DETERIORATED WOOD PLUS A MINIMUM OF ONE QUARTER INCH SOUND WOOD WAS REMOVED BY MAKING CUTS WITH A RECIPROCATING SAW OR A ROUTER. FACES OF CUT-OUT AREAS WERE LEFT SQUARE AND PARALLEL. A REPLACEMENT BLOCK WAS CUT TO FIT SNUGLY BUT NOT TIGHT WITH NOT MORE THAN 1/8" GAP AT ANY MEETING SURFACE. THE ENDS OF PIECES JOINED WERE SLOPED OR SCARFED.

EPOXY WAS MIXED ACCORDING TO MANUFACTURER'S DIRECTIONS AND PRECAUTIONS.

ALL MATING SURFACES OF BOTH THE WOOD MEMBER AND THE REPAIR BLOCK WERE FULLY COATED WITH THE MIXED EPOXY. REPAIR BLOCK WAS PLACED INTO THE CUT-OUT AREA ENSURING THAT ALL COATED FACES WERE IN FULL CONTACT WITH NO VOIDS OR GAPS. BLOCK WAS TAPPED INTO PLACE AND CLAMPED FOR A MINIMUM OF 48 HOURS. EXCESS ADHESIVE WAS REMOVED FROM THE GLUE LINE, AND THE NEW AREA WAS LEDT UNDISTURBED DURING THE SETTING PERIOD.

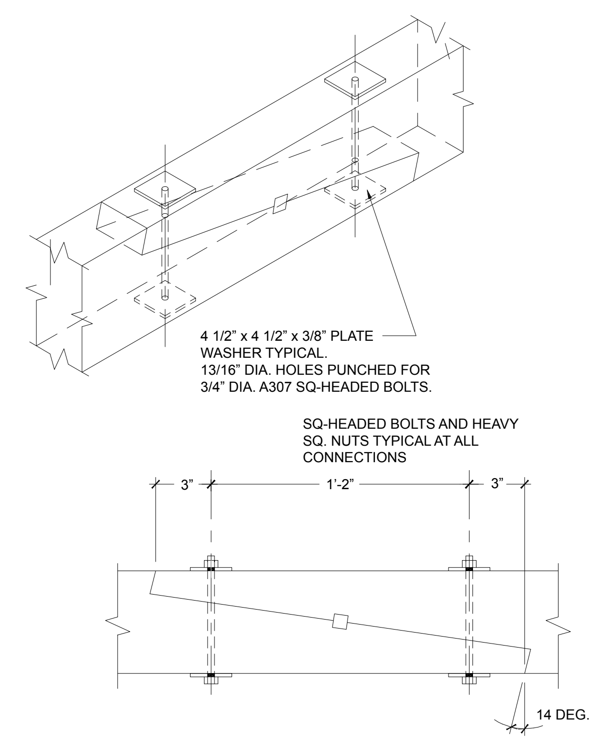
AFTER CURING, EXCESS GLUE WAS EITHER TOOLED OR SANDED AWAY.

CRACK REPAIR

LOOSE WOOD WAS MADE SECURE USING EPOXY AS A CRACK REPAIR ADHESIVE.

SEASON CRACKS (CHECKS) DID NOT REQUIRE REPAIR.

EPOXY CONSOLIDANT WAS MIXED ACCORDING TO THE MANUFACTURER'S INSTRUCTIONS. CONSOLIDANT WAS INSTALLED IN SPONGY WOOD AND IN DRILLED HOLES USING EITHER APPLICATORS SUPPLIED BY THE MANUFACTURER OR PLASTIC SQUEEZE BOTTLES WITH SMALL DIA. NOZZLES AND LEFT UNDISTURBED TO CURE.



Shelburne Farms Breeding Barn
Repair Details

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30 MARCH, 2011

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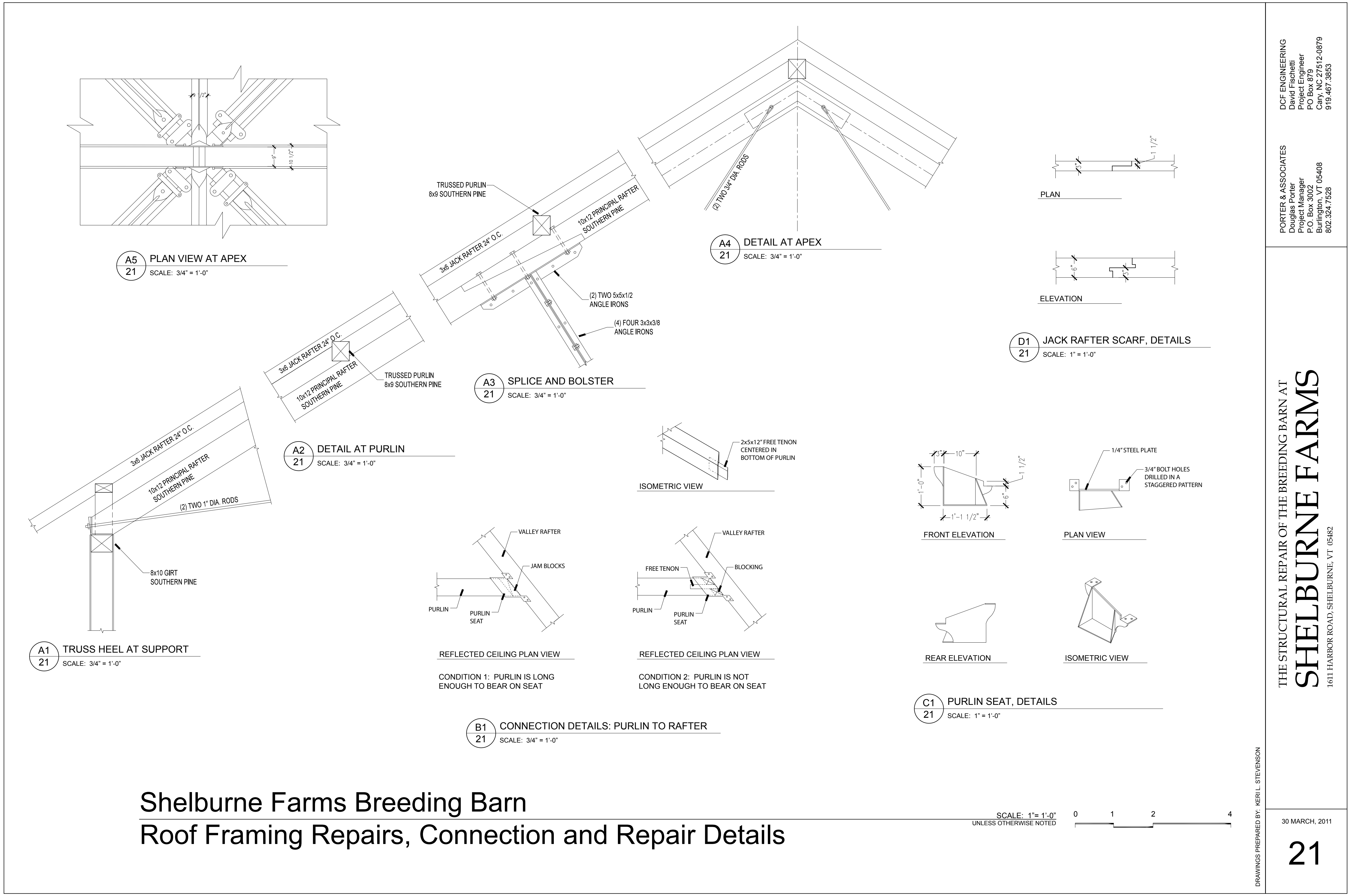
A5 PLAN VIEW AT APEX
21 SCALE: 3/4" = 1'-0"
TRUSSED PURLIN 8x9 SOUTHERN PINE
3x6 JACK RAFTER 24" O.C.
10x12 PRINCIPAL RAFTER SOUTHERN PINE
(2) TWO 5x5x1/2 ANGLE IRONS
(4) FOUR 3x3x3/8 ANGLE IRONS
A3 SPLICE AND BOLSTER
21 SCALE: 3/4" = 1'-0"
(2) TWO 3/4" DIA. RODS
A4 DETAIL AT APEX
21 SCALE: 3/4" = 1'-0"
PLAN
ELEVATION
D1 JACK RAFTER SCARF, DETAILS
21 SCALE: 1" = 1'-0"
A2 DETAIL AT PURLIN
21 SCALE: 3/4" = 1'-0"
(2) TWO 1" DIA. RODS
8x10 GIRT SOUTHERN PINE
A1 TRUSS HEEL AT SUPPORT
21 SCALE: 3/4" = 1'-0"
2x5x12" FREE TENON CENTERED IN BOTTOM OF PURLIN
ISOMETRIC VIEW
VALLEY RAFTER
JAM BLOCKS
PURLIN
PURLIN SEAT
REFLECTED CEILING PLAN VIEW
CONDITION 1: PURLIN IS LONG ENOUGH TO BEAR ON SEAT
FREE TENON
BLOCKING
REFLECTED CEILING PLAN VIEW
CONDITION 2: PURLIN IS NOT LONG ENOUGH TO BEAR ON SEAT
B1 CONNECTION DETAILS: PURLIN TO RAFTER
21 SCALE: 3/4" = 1'-0"
1/4" STEEL PLATE
3/4" BOLT HOLES DRILLED IN A STAGGERED PATTERN
FRONT ELEVATION
PLAN VIEW
REAR ELEVATION
ISOMETRIC VIEW
C1 PURLIN SEAT, DETAILS
21 SCALE: 1" = 1'-0"
Shelburne Farms Breeding Barn
Roof Framing Repairs, Connection and Repair Details
SCALE: 1"= 1'-0"
UNLESS OTHERWISE NOTED
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30 MARCH, 2011
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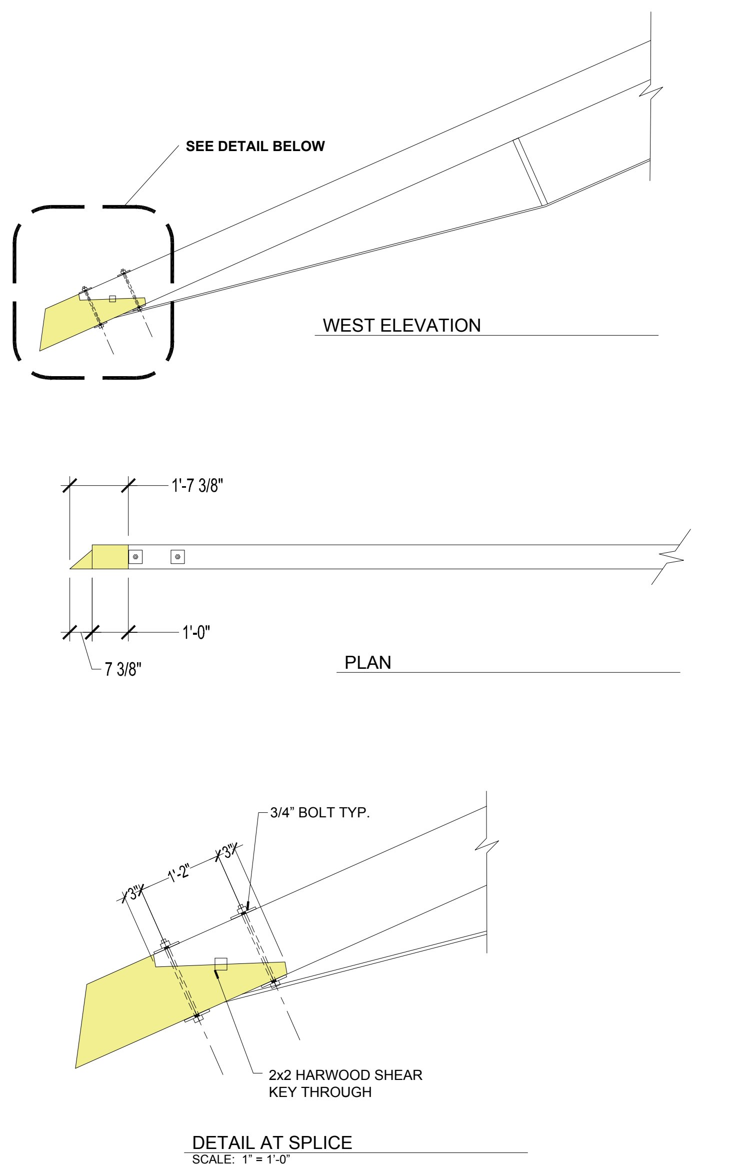
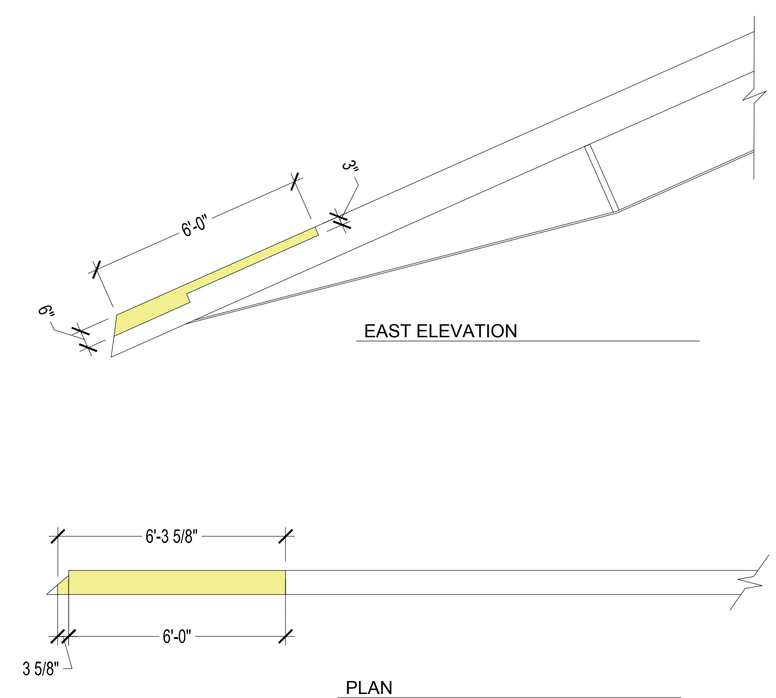
LEGEND

VALLEY RAFTER REPAIR



A1 22 VALLEY RAFTER REPAIR, 4.5 NORTH

EXTENT OF REPAIR:
SQUINTED SPLICE
MECHANICAL FASTENERS



B1 22 VALLEY RAFTER REPAIR, 8.0 NORTH

EXTENT OF REPAIR:
DUTCHMAN REPAIR IN LOWER PORTION OF VALLEY RAFTER
SPRUCE DUTCHMAN REPAIR EXTENDS THROUGH MEMBER

Shelburne Farms Breeding Barn
Roof Framing Repairs, Valley Rafter Details

SCALE: 1/2" = 1'-0"
UNLESS OTHERWISE NOTED

0 1 2 4 6

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30 MARCH, 2011

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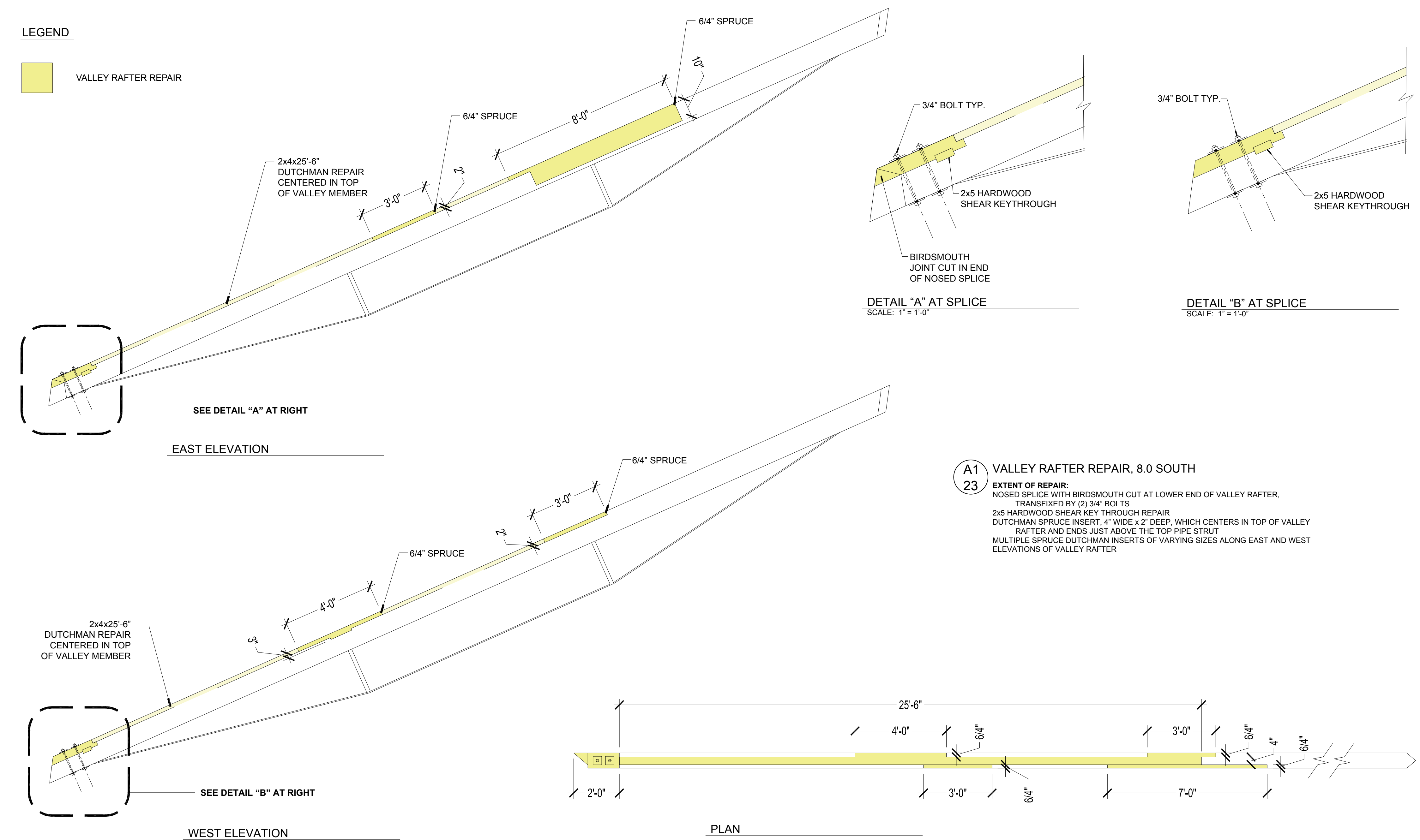

Shelburne Farms Breeding Barn
Roof Framing Repairs, Valley Rafter Details

SCALE: 1/2" = 1'-0"
UNLESS OTHERWISE NOTED
0 1 2 4 6

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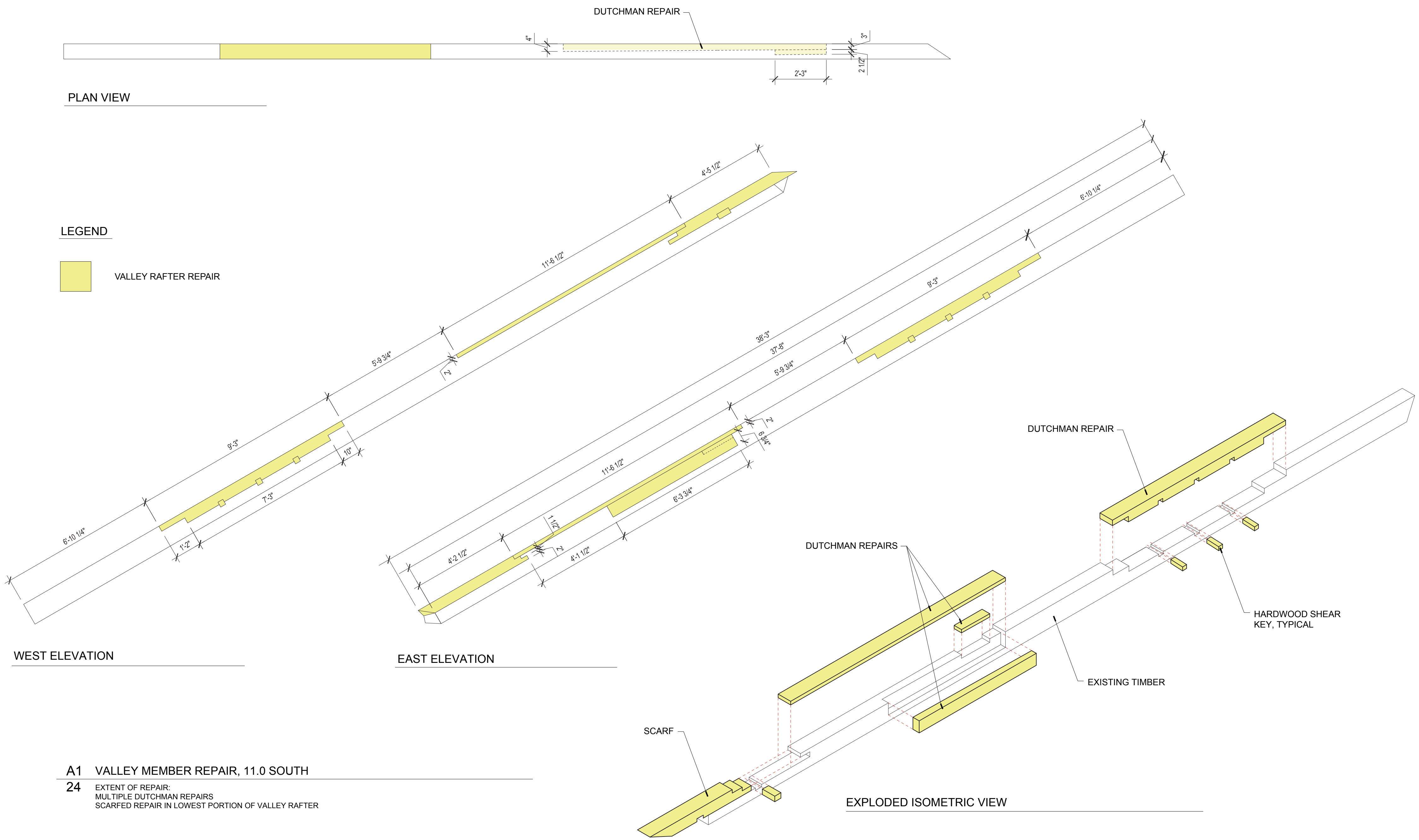
23



A1 VALLEY MEMBER REPAIR, 11.0 SOUTH
24 EXTENT OF REPAIR:
MULTIPLE DUTCHMAN REPAIRS
SCARFED REPAIR IN LOWEST PORTION OF VALLEY RAFTER

Shelburne Farms Breeding Barn
Roof Framing Repairs, Valley Rafter Details

SCALE: 1/2" = 1'-0"
UNLESS OTHERWISE NOTED

0 1 2 4 6

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30 MARCH, 2011

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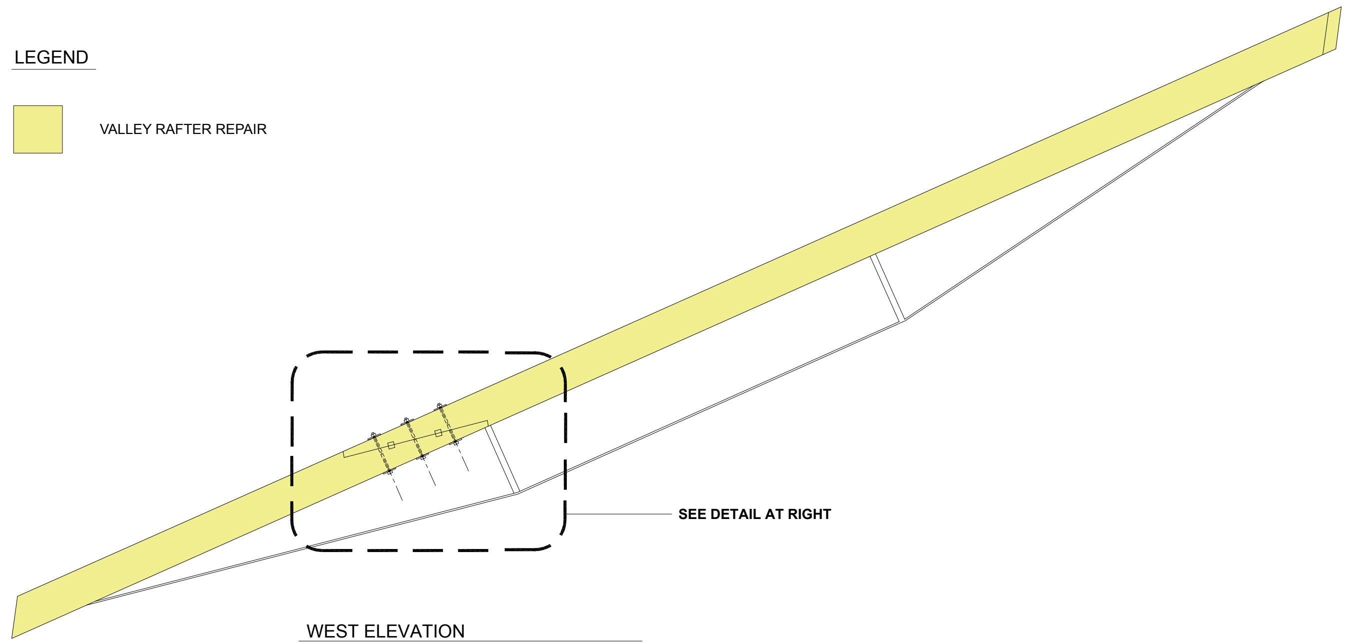
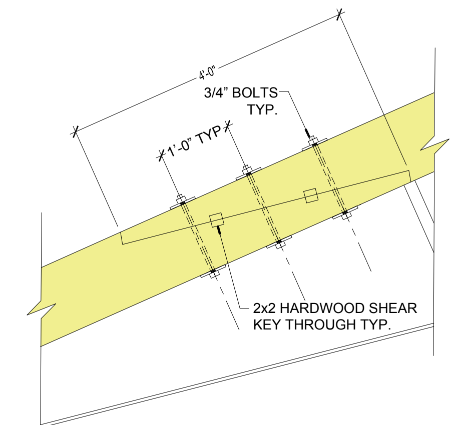
LEGEND

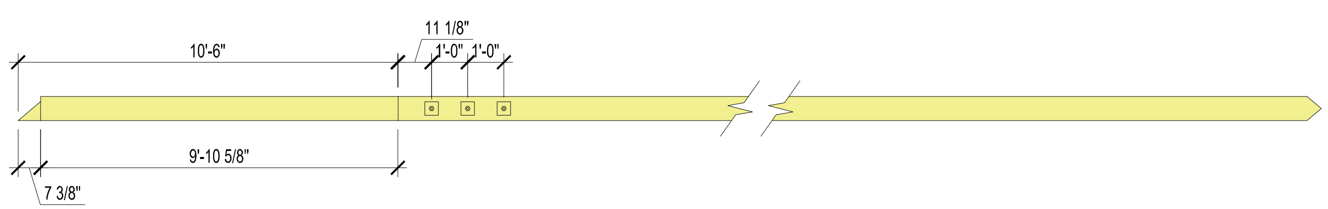
VALLEY RAFTER REPAIR



WEST ELEVATION



DETAIL AT SPLICE
SCALE: 1" = 1'-0"



PLAN

A1 / 25 VALLEY RAFTER REPAIR, 11.0 NORTH

EXTENT OF REPAIR:
- COMPLETE REPLACEMENT OF VALLEY RAFTER IN TWO PARTS
- SCARF BREAKS ON QUEENPOST AND 4'-0" BELOW
- REPAIR FASTENED WITH (3) 3/4" BOLTS, 1'-0" O.C.

Shelburne Farms Breeding Barn
Roof Framing Repairs, Valley Rafter Details

SCALE: 1/2" = 1'-0"
UNLESS OTHERWISE NOTED

0 1 2 4 6

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30 MARCH, 2011

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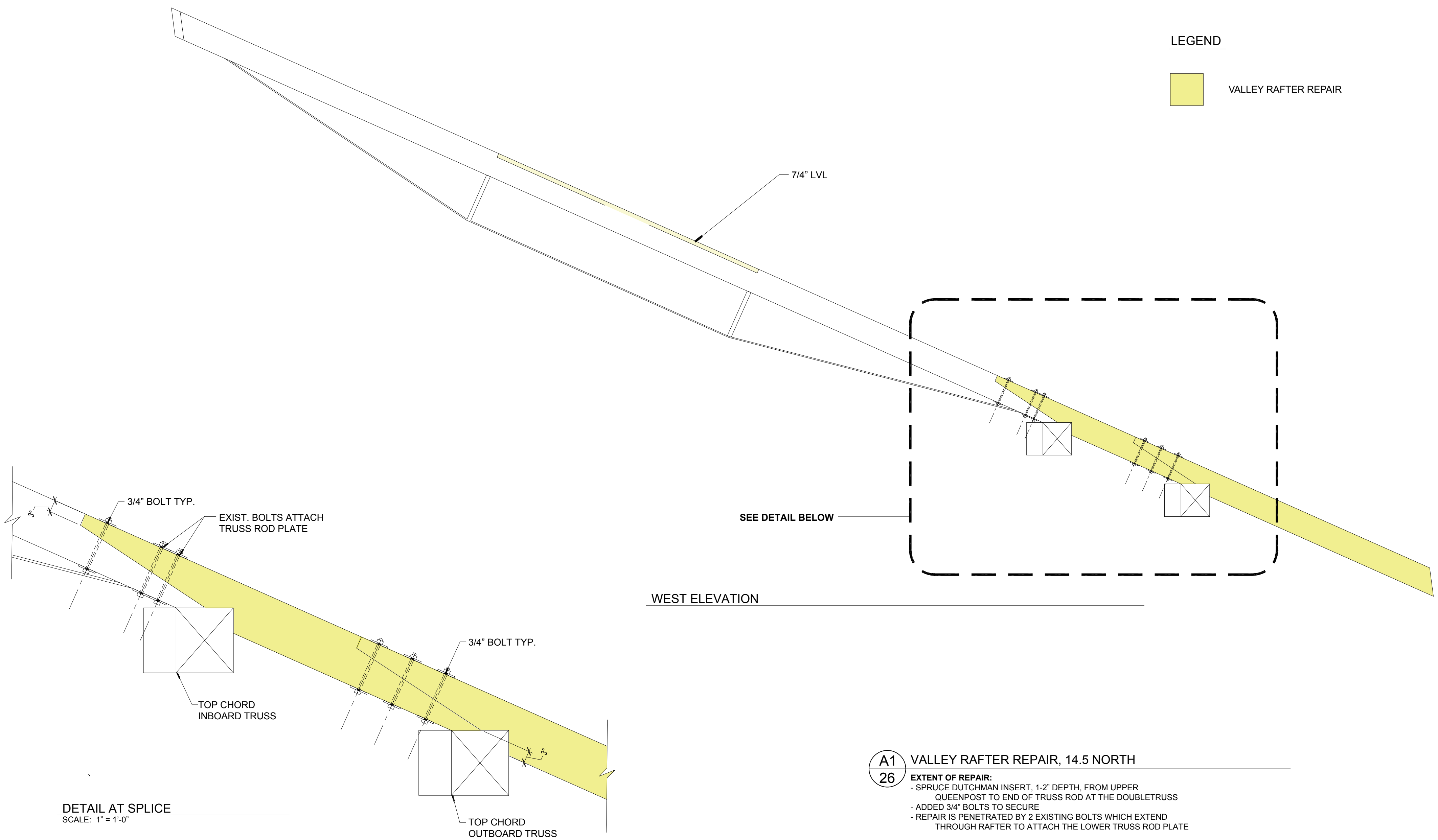

Shelburne Farms Breeding Barn
Roof Framing Repairs, Valley Rafter Details

SCALE: 1/2" = 1'-0"
UNLESS OTHERWISE NOTED

0 1 2 4 6

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30 MARCH, 2011

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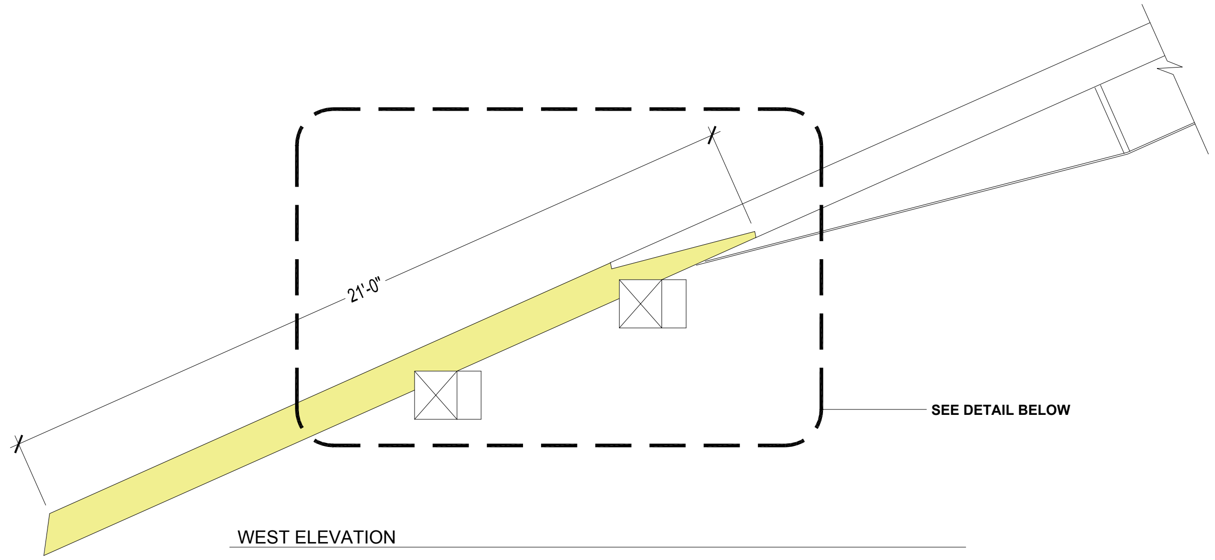
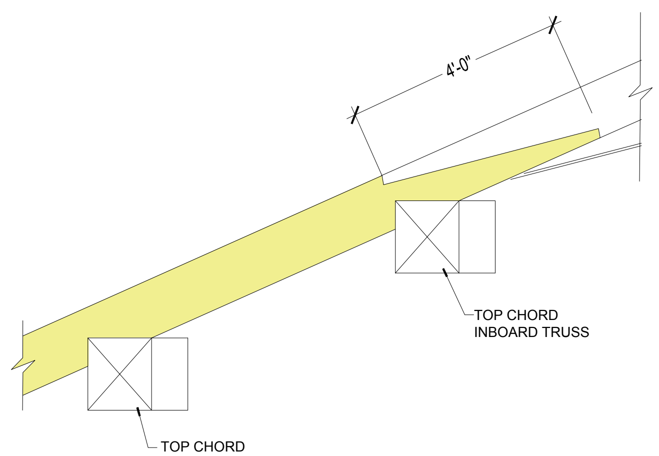


WEST ELEVATION



DETAIL AT SPLICE
SCALE: 1" = 1'-0"

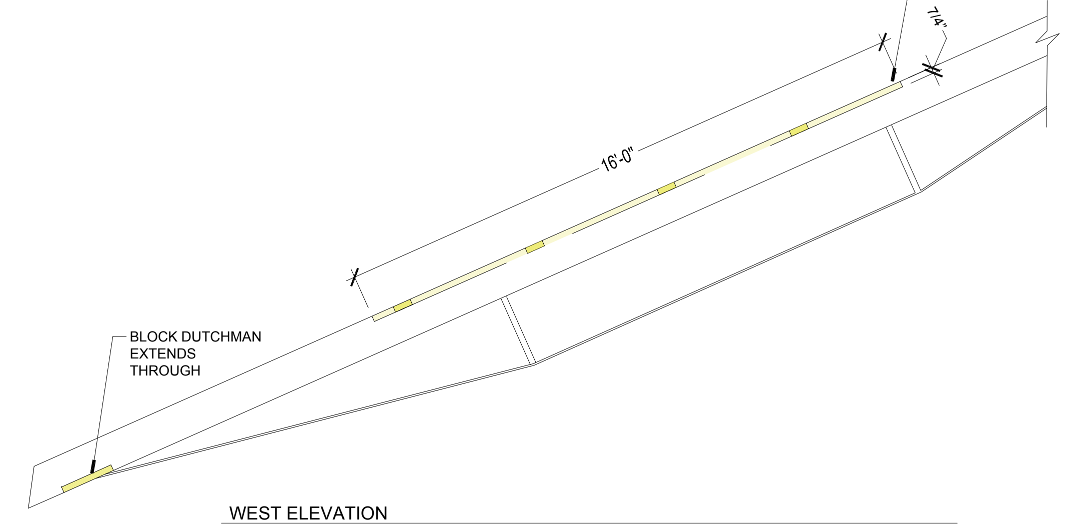
PHOTO DETAIL AT OUTBOARD TRUSS
N.T.S.

A1
27
VALLEY RAFTER REPAIR, J14.5

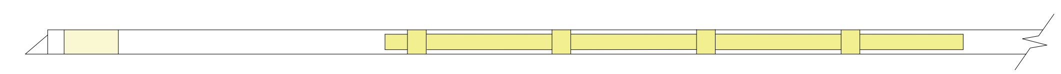
EXTENT OF REPAIR:
- SCARF REPAIR IN LOWER PORTION OF VALLEY MEMBER
- SCARF CENTERS OVER INBOARD TRUSS
- BLOCK INSTALLED IN TOP OF OUTBOARD TRUSS CHORD - SEE PHOTO FOR ADDITIONAL DETAIL

LEGEND

VALLEY RAFTER REPAIR



WEST ELEVATION



PLAN

B1
27
VALLEY RAFTER REPAIR, 17.0 NORTH

EXTENT OF REPAIR:
- LVL DUTCHMAN INSERT ALONG TOP CENTER OF VALLEY MEMBER
- MULTIPLE DUTCHMAN INSERTS ALONG TOP OF MEMBER WHERE JACK RAFTERS INTERSECT WITH VALLEY RAFTER
- BLOCK DUTCHMAN EXTENDING THROUGH MEMBER AT BOTTOM TRUSS ROD TERMINUS

Shelburne Farms Breeding Barn
Roof Framing Repairs, Valley Rafter Details

SCALE: 1/2" = 1'-0"
UNLESS OTHERWISE NOTED

0 1 2 4 6

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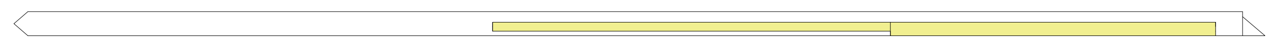
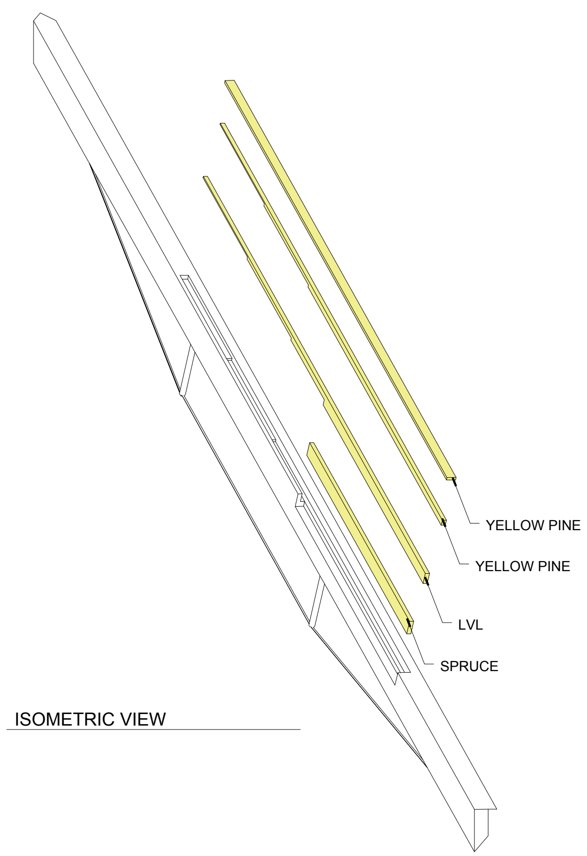


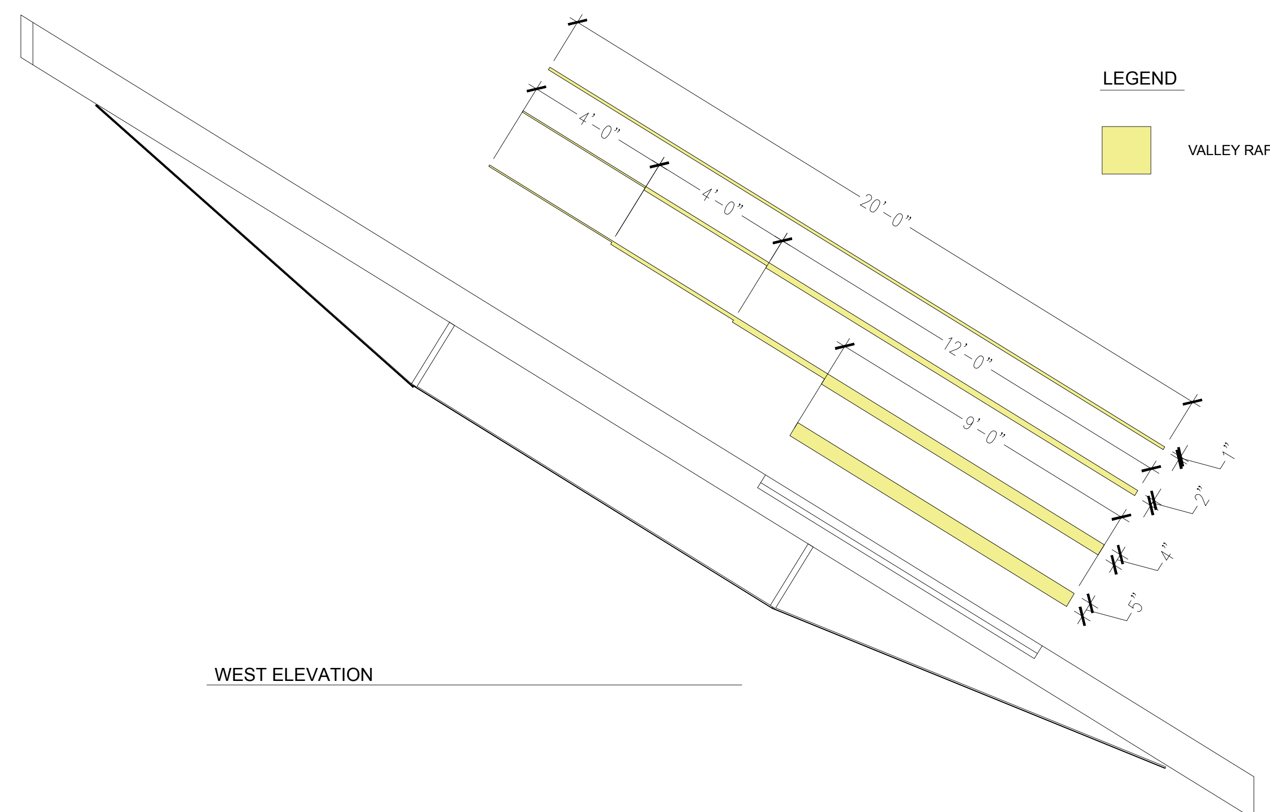
PLAN

A1 / 28 VALLEY RAFTER REPAIR, 17.0 SOUTH

EXTENT OF REPAIR:
- MULTIPLE DUTCHMAN INSERTS
- NEW WOOD CONNECTED TO EXISITING WOOD AND TO ADJACENT NEW WOOD WITH GAP FILLING EPOXY
- REPAIR CONTOURED TO FIT DECAY POCKETS
- DUTCHMAN INSERTS ARE LVLs, YELLOW PINE, AND SPRUCE - SEE DRAWING FOR LOCATIONS



ISOMETRIC VIEW



WEST ELEVATION

LEGEND

VALLEY RAFTER REPAIR

Shelburne Farms Breeding Barn
Roof Framing Repairs, Valley Rafter Details

SCALE: 1/2" = 1'-0"
UNLESS OTHERWISE NOTED

0 1 2 4 6

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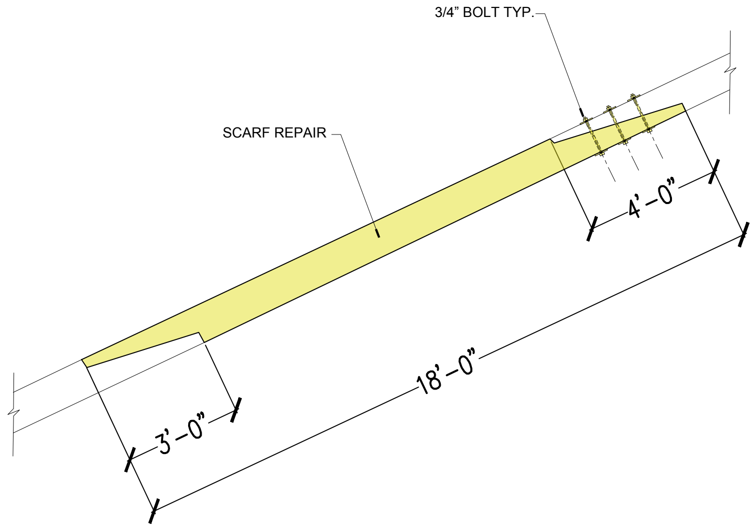
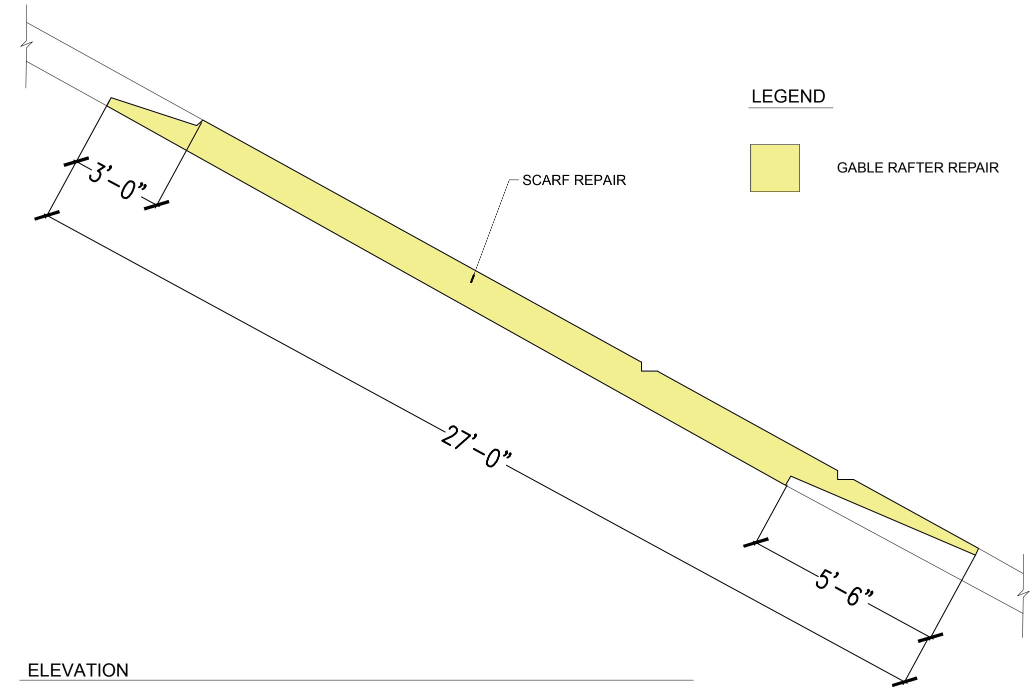


ELEVATION



LEGEND

GABLE RAFTER REPAIR

ELEVATION

PHOTO ELEVATION
APPROX. SCALE: 3/16" = 1'-0"

PHOTO ELEVATION
APPROX. SCALE: 3/16" = 1'-0"

A1/29 GABLE END RAFTER REPAIR, EAST

EXTENT OF REPAIR:
SQUINTED SCARF APPROX. 18'-0"
SOUTHERN SCARF, APPROX. 3'-0"
NORTHERN SCARF, APPROX. 4'-0"
MECHANICAL FASTENERS

B1/29 GABLE END RAFTER REPAIR, WEST

EXTENT OF REPAIR:
SQUINTED SCARF, APPROX. 27'-0"
SOUTHERN SCARF, APPROX. 3'-0"
NORTHERN SCARF, APPROX. 5'-6"

Shelburne Farms Breeding Barn
Roof Framing Repairs, Gable End Rafter Repairs

SCALE: 3/8" = 1'-0"
UNLESS OTHERWISE NOTED

0 1 2 4 6

DRAWINGS PREPARED BY: KERI L. STEVENSON

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30 MARCH, 2011

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Appendix B: Archival Drawings

Shelburne Farms is fortunate to have in its archives a large volume of material related to the construction of the Farm, including many of architect Robertson's sketches, drawings, painted delineations, and specifications; written orders for materials, bills of sale, and ledger entries concerning the purchase of construction materials; and letters, diaries, articles from contemporary newspapers and periodicals, and other written accounts of construction activities on the Farm. As part of the investigation of the Breeding Barn, the project team reviewed the original plans to determine the impact on the analysis of various elements, and field-verified the dimensions of individual elements against those drawings.

In this appendix, Robertson's surviving drawings of the Breeding Barn have been assembled along with shop drawings from Post & McCord and an order submitted to the company for ironwork . Robertson's structural designs for the barn seem to have developed over time, and that development is illustrated in the drawing set.

In the material from Post & McCord, the order includes cast shoes (truss heel connections), center rods, struts, and lower chord elements for 14 trusses, along with truss rods, bridles, and mounting brackets for purlins, and ironwork for aisle trusses. The cross-brace ties are not included on this order. This order form is accompanied by what appear to be shop drawings of the truss rods and aisle truss ironwork (which appear on the order) dated Aug 29, 1890.

This order seems to correspond to a group of drawings, all of which seem to have been drawn by the same draftsman (based on lettering style), that perhaps predate Robertson's intention to double the trusses at the lantern location and include cross-brace ties as part of the roof framing. These drawings include a plan of the Exercising Ring (showing truss locations), a Transverse Section Detail, Transverse and Longitudinal Sections, and a Roof Plan, One Half. The plans at this time did not include the annex.

A plan of the Main Roof Construction, One Half (which apparently postdates the group of drawings already described) shows cross-brace ties between trusses, and additional trusses at the lantern; these were conspicuously absent in the first group of drawings. This later drawing illustrates cross-brace ties terminating at a plate or shoe attached to the aisle side of the timber plates of the riding ring walls. The annex does not appear in this plan. A detail drawing of the Centre Block or Centre Ring Buckle (a cross-brace tie connection detail) appears to have been drawn by the same draftsman.

A second Transverse Section detail illustrates two additional trusses for the lantern location, as well as the cross-brace ties. The draftsman appears to be the same person responsible for the first group of drawings, but because the additional lantern trusses and the cross-brace ties were omitted from the first group, this drawing may represent a later development in the structural design. For the lantern trusses, a second strut has been added at the lantern-purlin location, and all of the iron bottom chord elements have been upsized. It is interesting that the center tie is upsized only 1/8-inch, indicating perhaps that the designer intended that lateral stresses be resisted primarily by the cross-brace ties.

A drawing of the north elevation is not labeled, and so is hard to compare to the other drawings (with respect to authorship). This drawing is not "as-built", and omits doorways on either side of the main entrance.

Sheets with annex drawings and chimney details seem to have been prepared by the same draftsman; their place in the drawing chronology is unknown. Similarly, a sheet devoted to the framing of the west aisle seems to have been done by a draftsman not otherwise represented in the drawing set.

Nearly all of the drawings give the address of Robertson's offices at 121 East 23rd Street in New York. It is believed that Robertson moved from this address in 1891, and so the time period over which this set of drawings was developed was probably relatively short, perhaps just two years. Given the absence of cross-brace ties from some of the drawings, and the overstresses predicted for lower chord elements by computer models focused on the truss alone, it is tempting to suppose that R. H. Robertson added the cross-brace tie rods to the basic truss configuration sometime during construction, thus reducing horizontal and vertical deflection and reducing the stresses in both the original wrought iron elements and timber top chord. The addition of ¾-inch thick by 3-inch high cross-brace ties to the building provides another path for the tensile force to be resisted.

Order No. .... *102 Broadway, New York,* *188*

POST & McCORD,

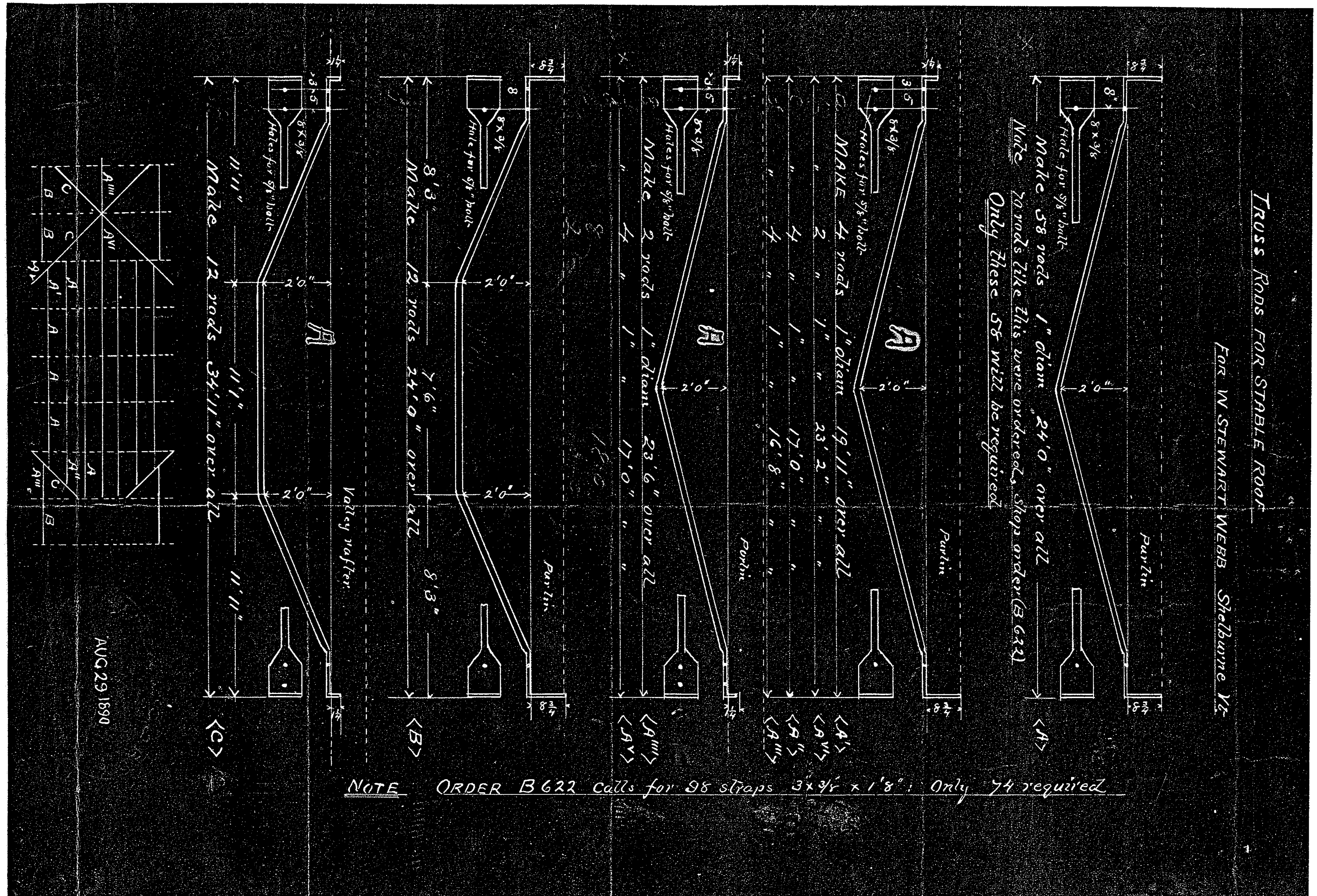
Civil Engineers, Bridge Builders and Contractors in Iron,

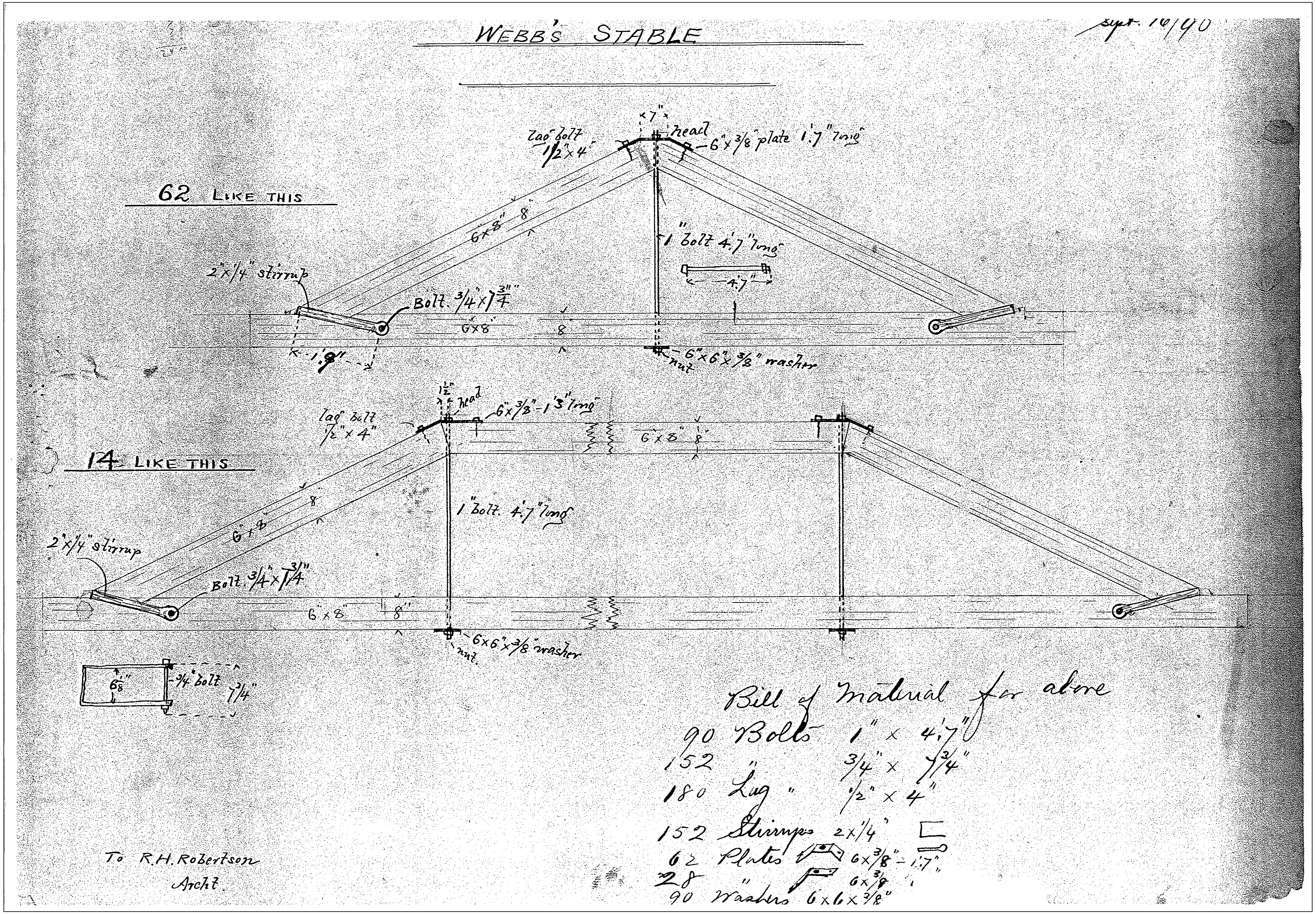
BILL OF FINISHED MATERIAL

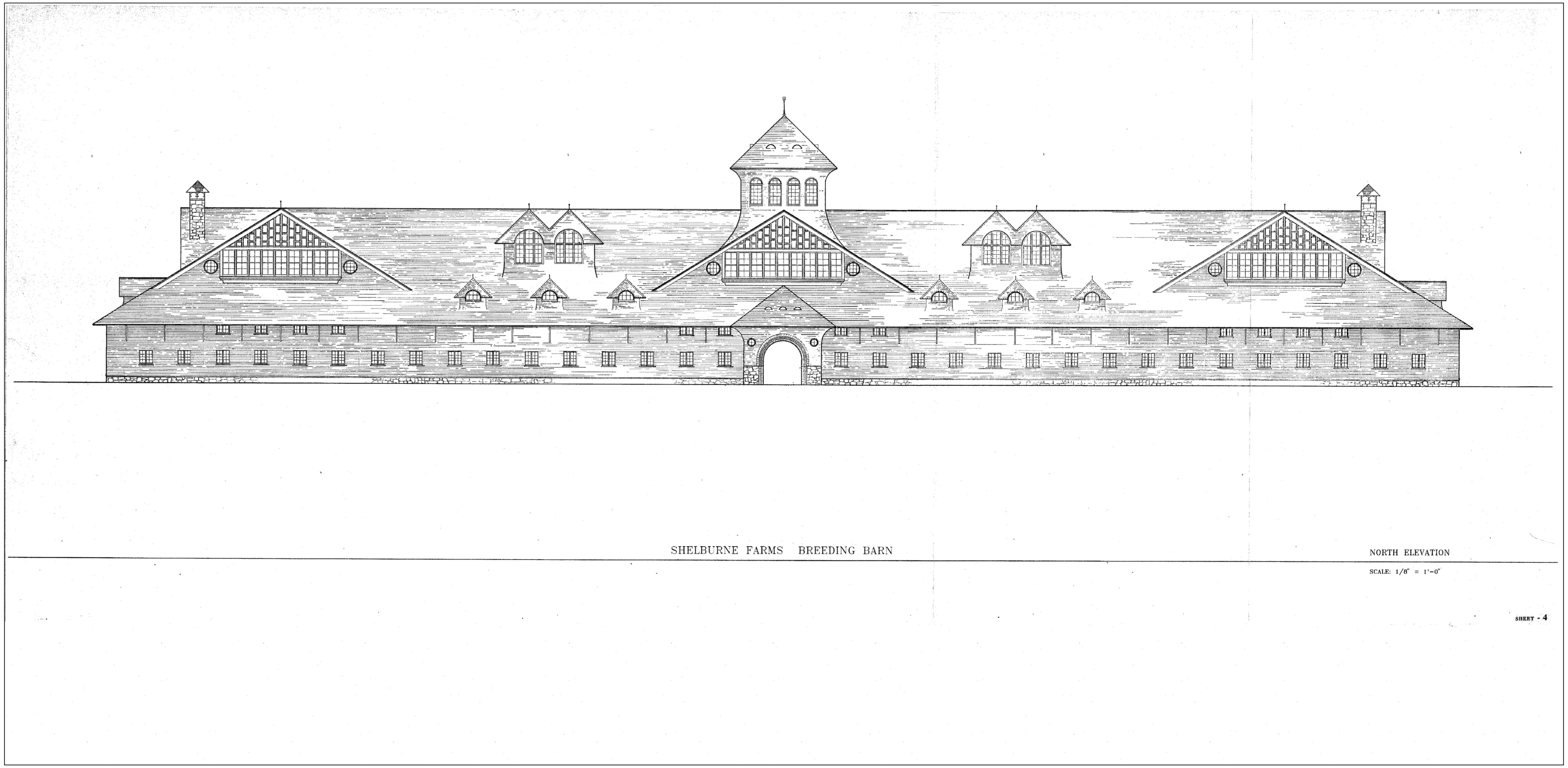
For STABLE ROOF FOR W.SEWARD WEBB,SHELBOHNE,VERMONT

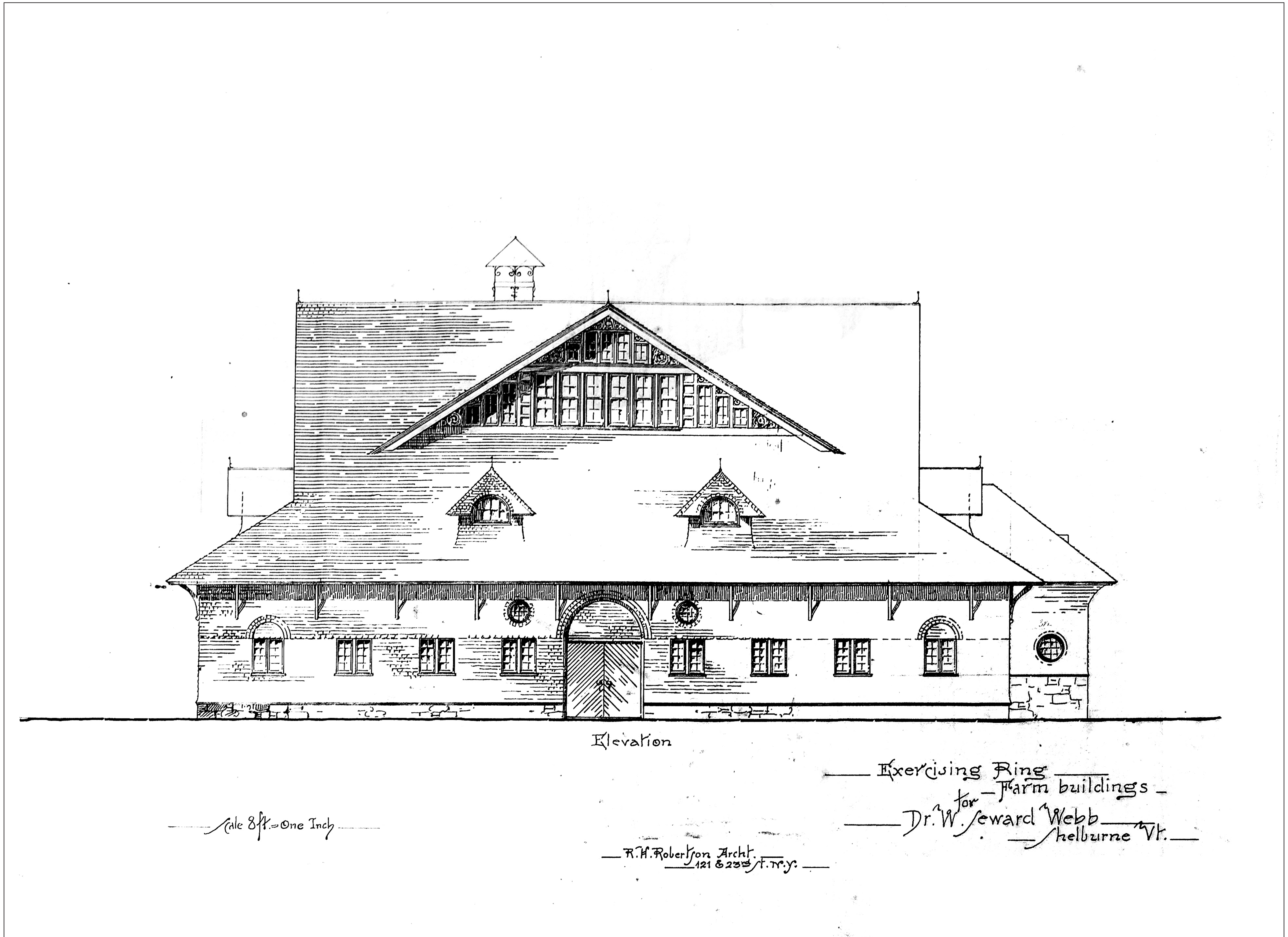
To

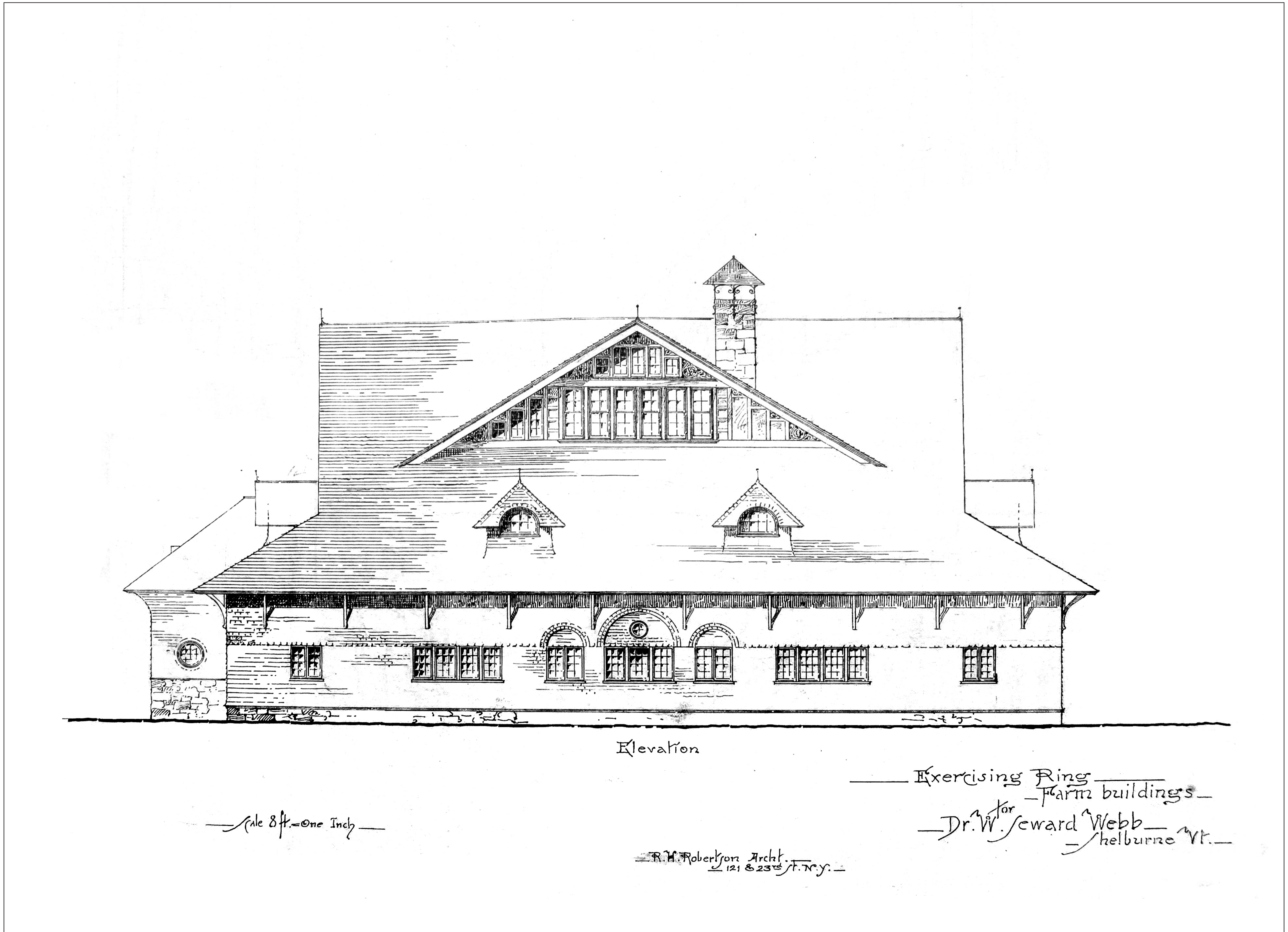
No. of Pieces.	DESCRIPTION.	LENGTH. Feet	Inches.	REMARKS.
28	Struts	9	4	
14	Shoes 7			
28	Rafter Plates 9x5/8			
28	Cast Shoes 12			
84	Cast Shoes 68			
126	3" G.P.Struts	2	0	
74	Straps 3 x 5/8	1	8	
14	Rods (C) 1-1/2" Rd	27	5	c to c
56	Rods (B) do 3/4" Rd	23	5	c to c 13
56	Rods (A) 1" Rd	23	9	c to end
58	Truss Rods (A) 1" Rd	24	0	Over all 40
4	Truss Rods (A) " 1" Rd	19	11	Over all
2	Truss Rods (A6) " 1" Rd	23	2	Over all
4	Truss Rods (A~~1~~2) " 1" Rd	17	0	Over all
4	Truss Rods (A3) " 1" Rd	16	8	Over all
2	Truss Rods (A4) " 1" Rd	23	6	Over all
4	Truss Rods (A5) " 1" Rd	17	0	Over all
12	Truss Rods (B) 1" Rd	24	0	Over all
12	Truss Rods (C) 1" Rd	34	11	Over all
28	Bolts 7/8 Rd,7-1/2" bet Head & Nut			1 Wrt Wash to eac
28	Bolts ~~1/2~~ 1-1/2 Rd,13" bet Hd & Nut			1 Wt Wash to each
168	Bolts 5/8 x 14" ~~Rat Hd & Wash~~			
168	Bolts 5/8 x 12-1/2"			
48	Bolts 5/8 x 11-3/4"			
214	Bolts 5/8 x 11"			
278	Wrt Washers for 5/8 Bolts			
620	Lag Screws 1/2 x 6"			
40	Bridle Irons 3 x 1/2			
2	Plates 16" x 1/2" x 16"			

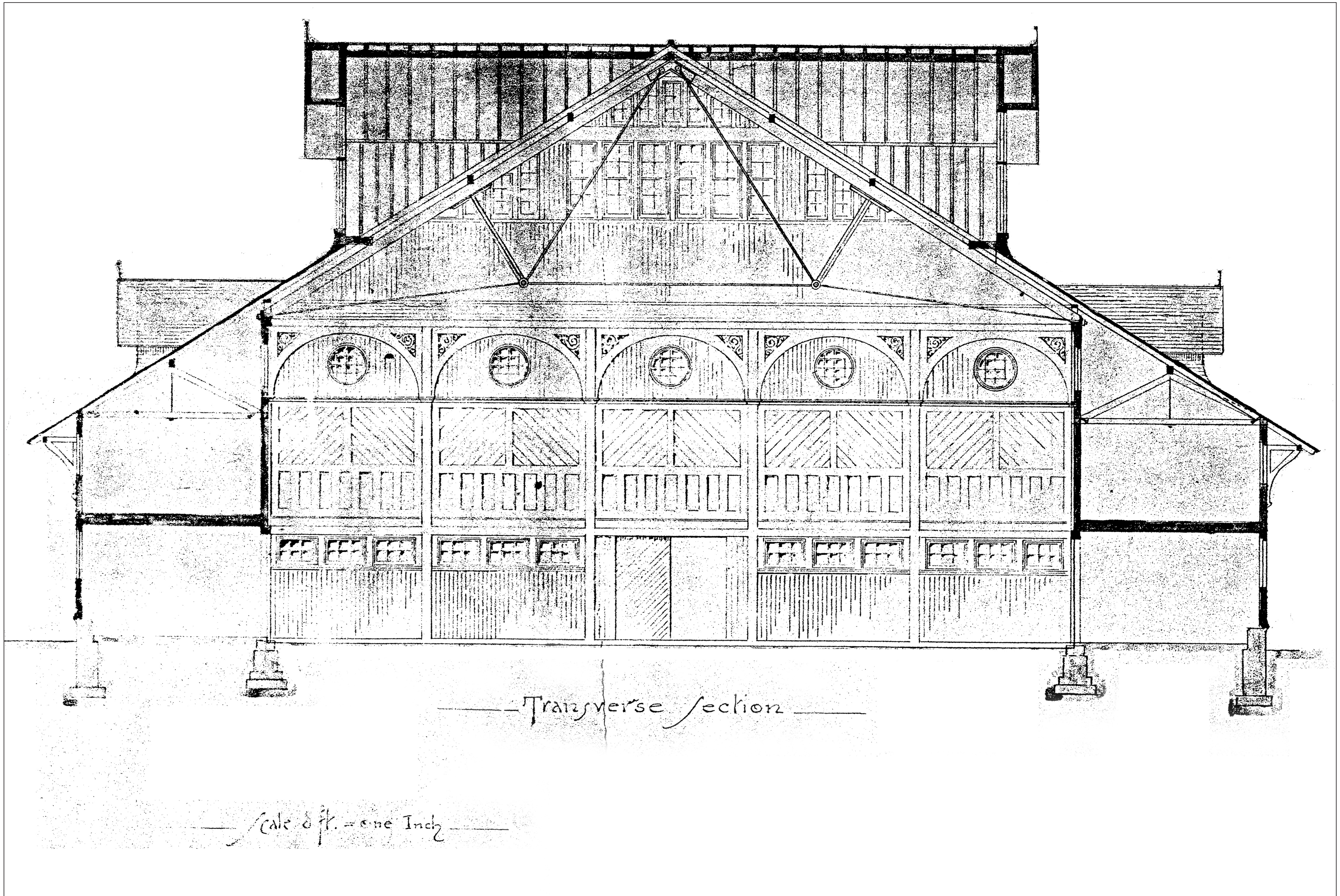

TRUSS RODS FOR STABLE ROOF
FOR W. STEWART WEBB Shelburne Vt.
Make 58 rods 1" diam 24'0" over all
Only these 58 will be required
MAKE 4 rods 1" diam 19'11" over all
Make 2 rods 1" diam 23'6" over all
Make 12 rods 24'0" over all
Make 12 rods 34'11" over all
Valley rafter
Purlin
NOTE ORDER B622 calls for 98 straps 3x3/8 x 1'8": Only 74 required
AUG 29 1890

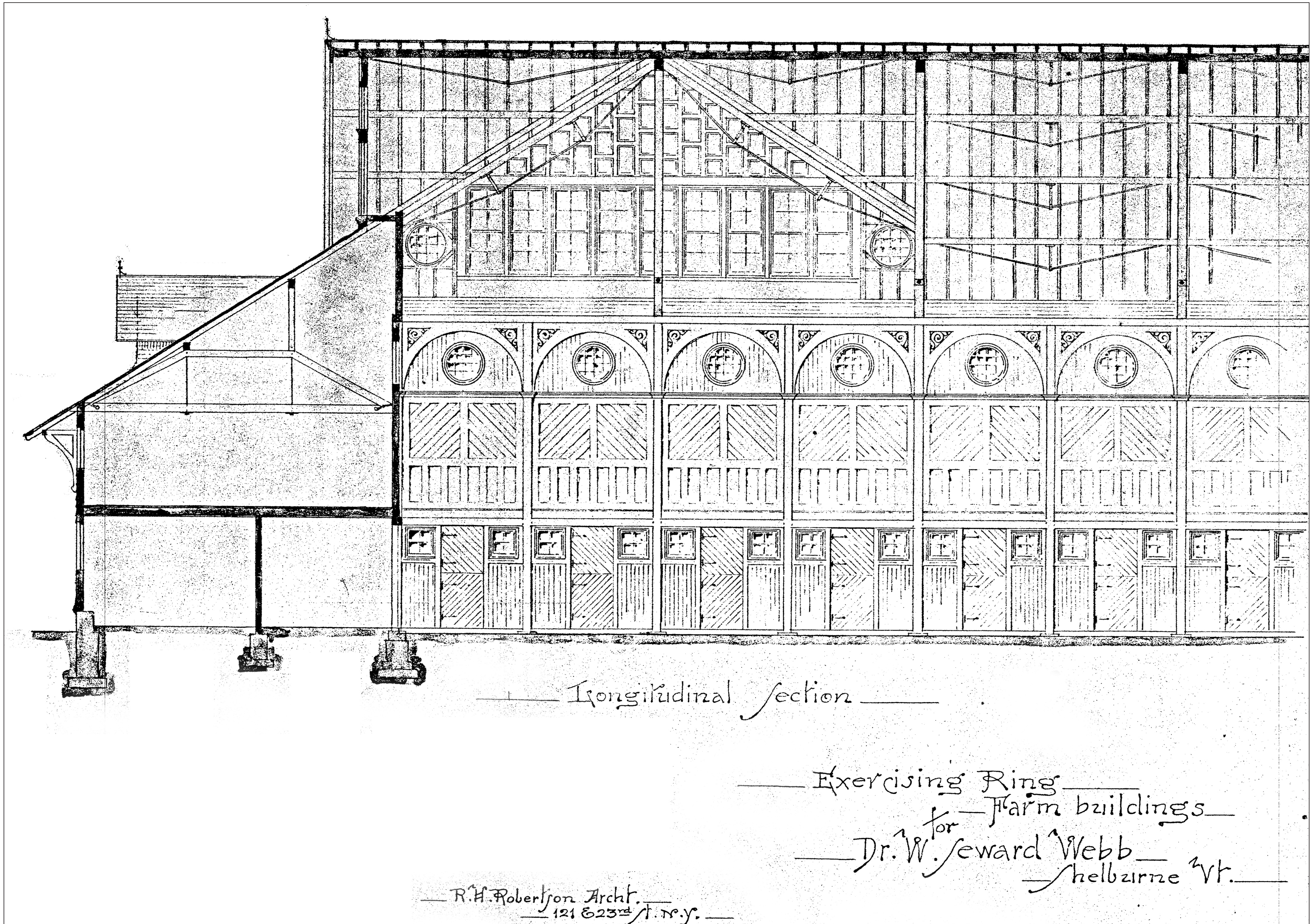

WEBB'S STABLE
Sept. 16/90
62 LIKE THIS
14 LIKE THIS
Lag bolt 1/2"x4"
head
6"x3/8" plate 1'7" long
6x8" 8"
1" bolt 4'7" long
4'7"
2"x1/4" stirrup
Bolt. 3/4"x7 3/4"
6x8 8
1'8"
6"x6"x3/8" washer
nut
1 1/2" head
6x3/8"-1'3" long
6"x8" 8
6x6"x3/8" washer
6 5/8"
3/4" bolt 7 1/4"
To R.H. Robertson Archt.
Bill of Material for above
90 Bolts 1" x 4'7"
152 " 3/4" x 7 3/4"
180 Lag " 1/2" x 4"
152 Stirrups 2x1/4"
62 Plates 6x3/8" - 1'7"
28 " 6x3/8
90 Washers 6x6x3/8"

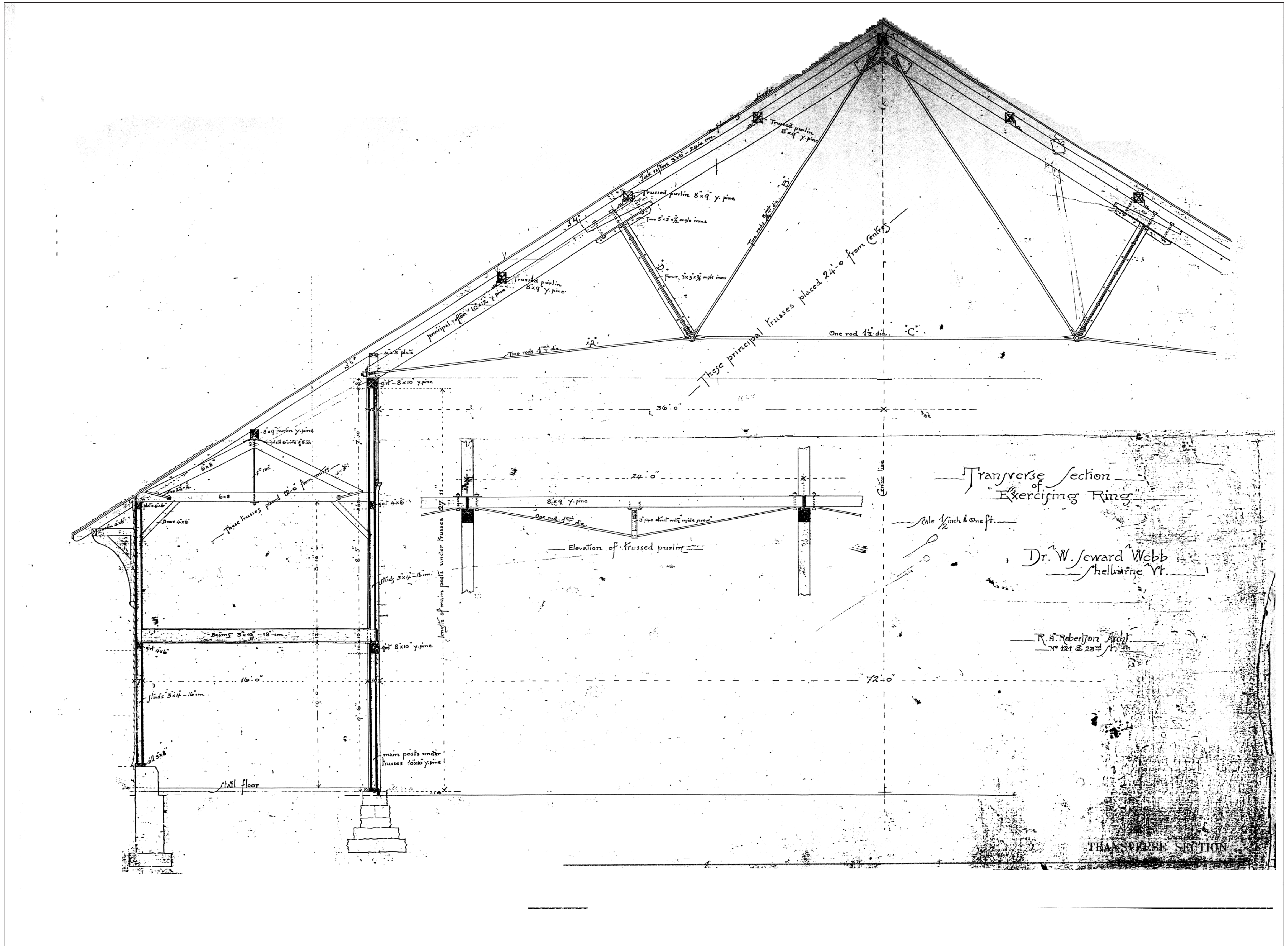

SHELBURNE FARMS BREEDING BARN
NORTH ELEVATION
SCALE: 1/8" = 1'-0"
SHEET - 4

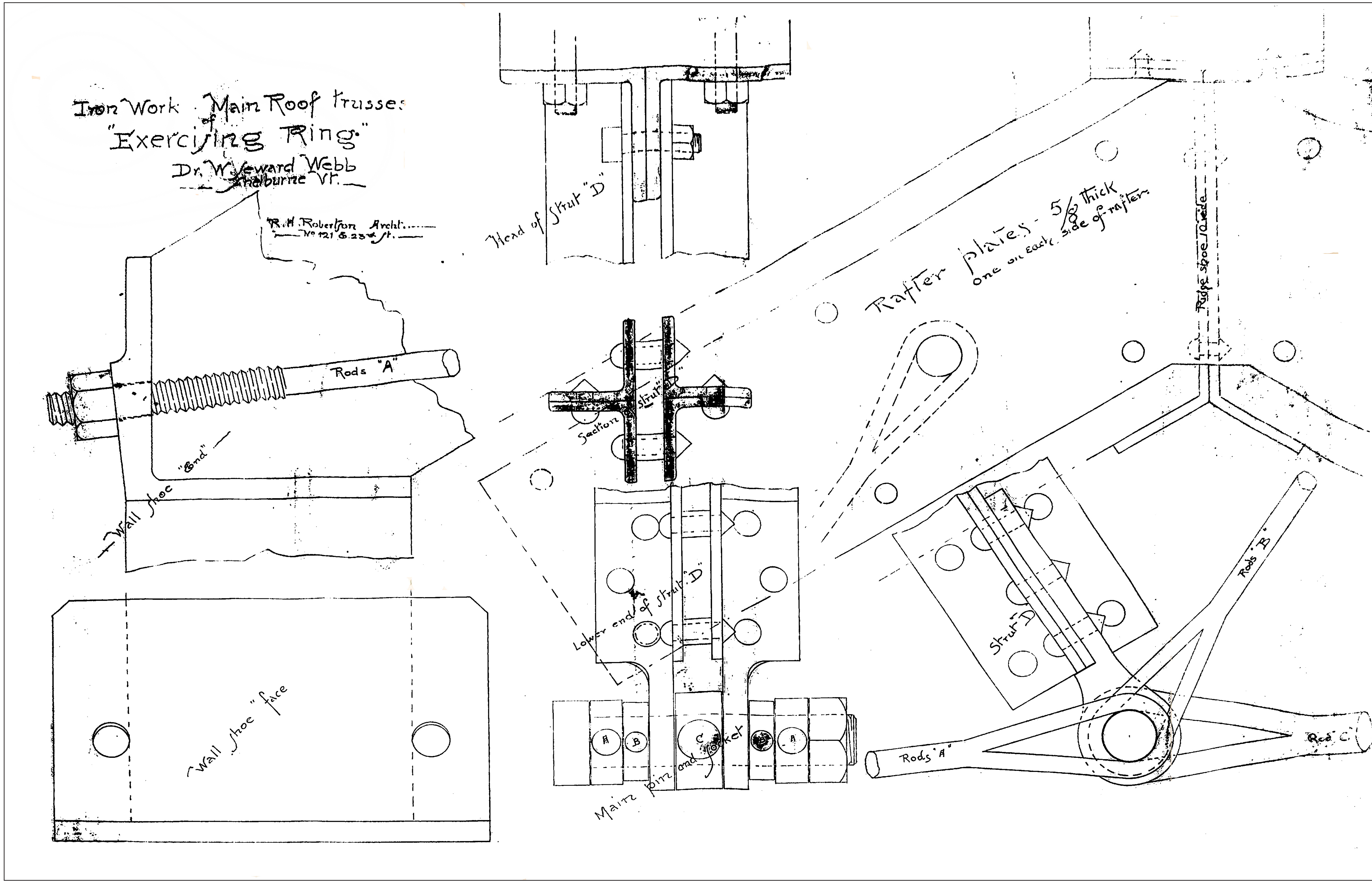

Elevation
Exercising Ring
Farm buildings
for
Dr. W. Seward Webb
Shelburne Vt.
Scale 8ft. = One Inch
R. H. Robertson Archt.
121 & 23rd St. N.Y.

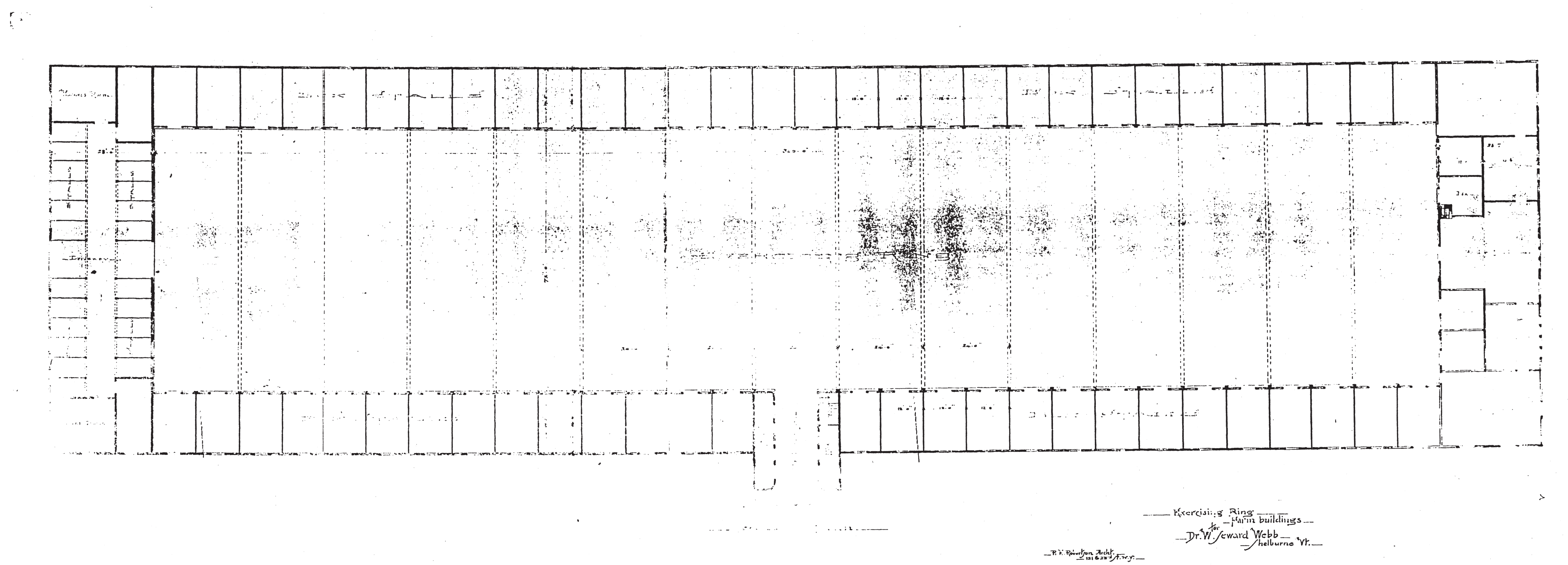

Elevation
Exercising Ring
Farm buildings
for
Dr. W. Seward Webb
Shelburne Vt.
Scale 8 ft. = One Inch
R. H. Robertson Archt.
121 E 23rd St. N.Y.


Transverse Section
Scale 8 ft. = one Inch


Longitudinal Section
Exercising Ring
Farm buildings
for
Dr. W. Seward Webb
Shelburne Vt.
R. H. Robertson Archt.
121 E 23rd St. N.Y.


Transverse Section of "Exercising Ring"
Scale 1/2 inch & One ft.
Dr. W. Seward Webb
Shelburne Vt.
R. H. Robertson Archt
No 121 E 23rd St.
These principal trusses placed 24'-0 from Centres
These trusses placed 12'-0 from centres
Elevation of trussed purlin
One rod 1½ dia.
Two rods 1" dia.
36'-0
72'-0
24'-0
16'-0
Stall floor
TRANSVERSE SECTION


Iron Work Main Roof trusses of "Exercising Ring"
Dr. W. Seward Webb
Shelburne Vt.
R. H. Robertson Archt.
No. 121 E. 23rd St.
Head of Strut "D"
Rafter plates - 5/8 thick
one on each side of rafter
Ridge shoe ¼ scale
Rods "A"
Section Strut
Wall shoe "end"
Lower end of strut "D"
Strut "D"
Rods "B"
Wall shoe "face"
Main pin and socket
Rods "A"
Rod "C"

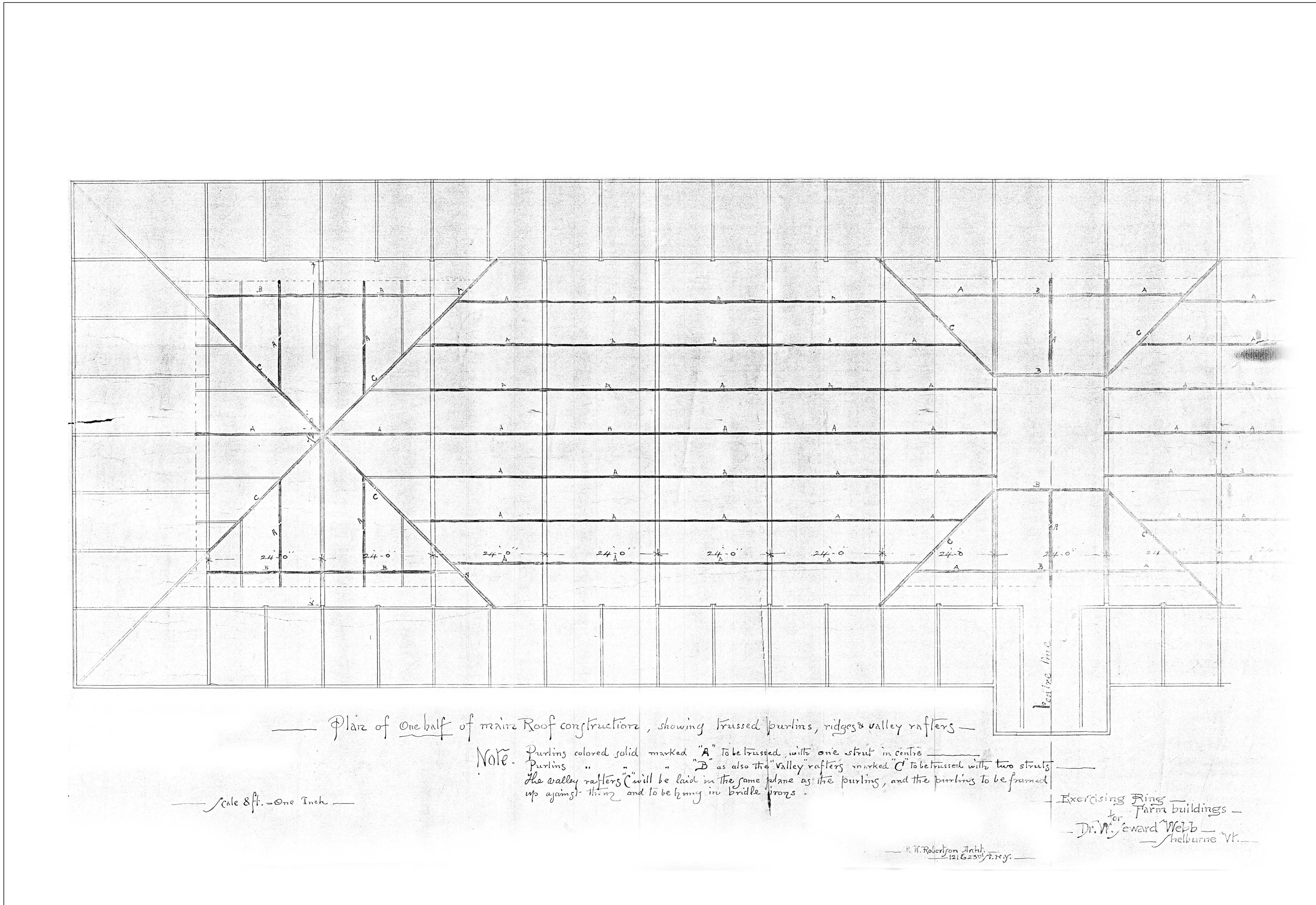


SHELBURNE FARMS BREEDING BARN

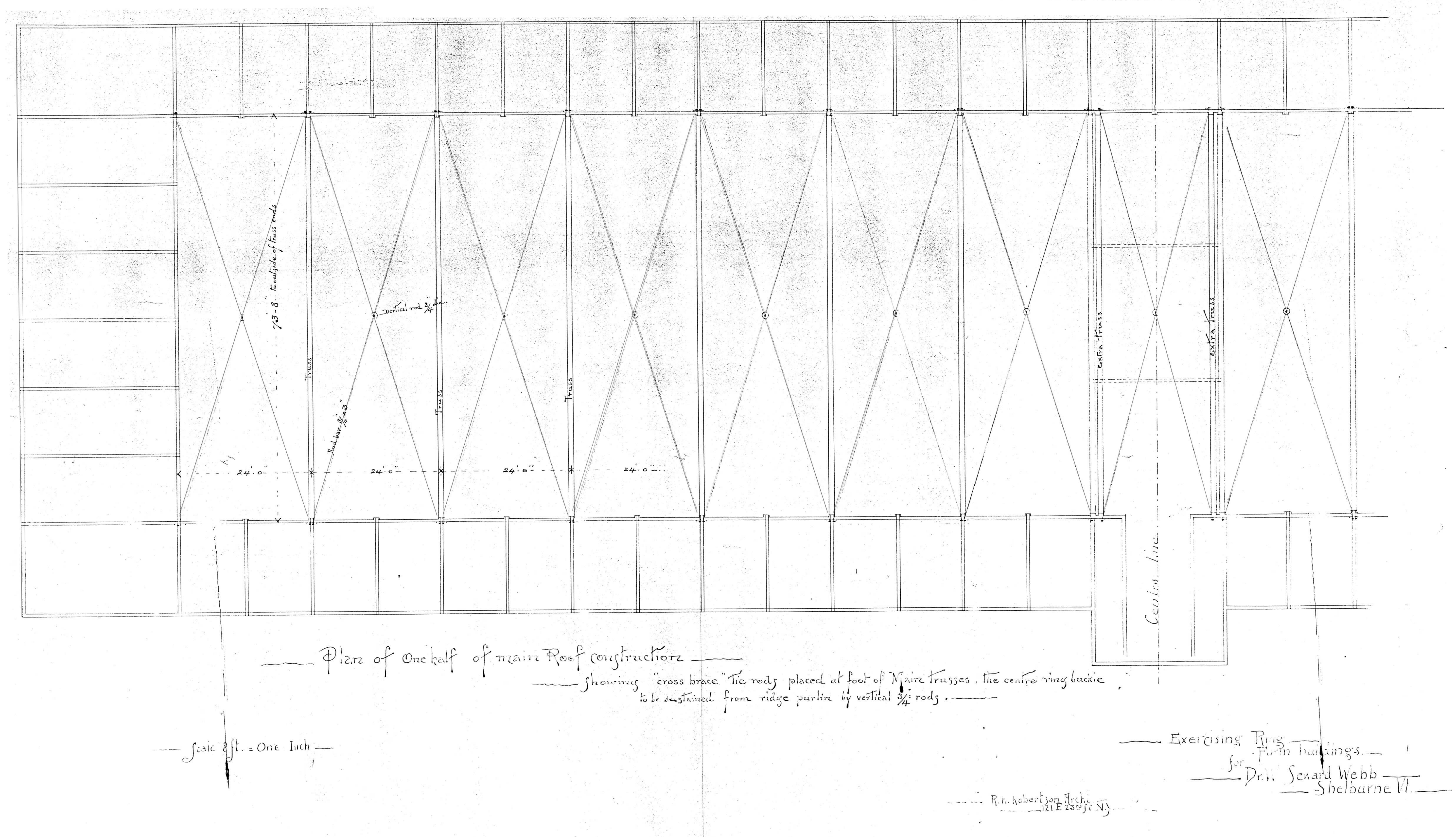
PLAN

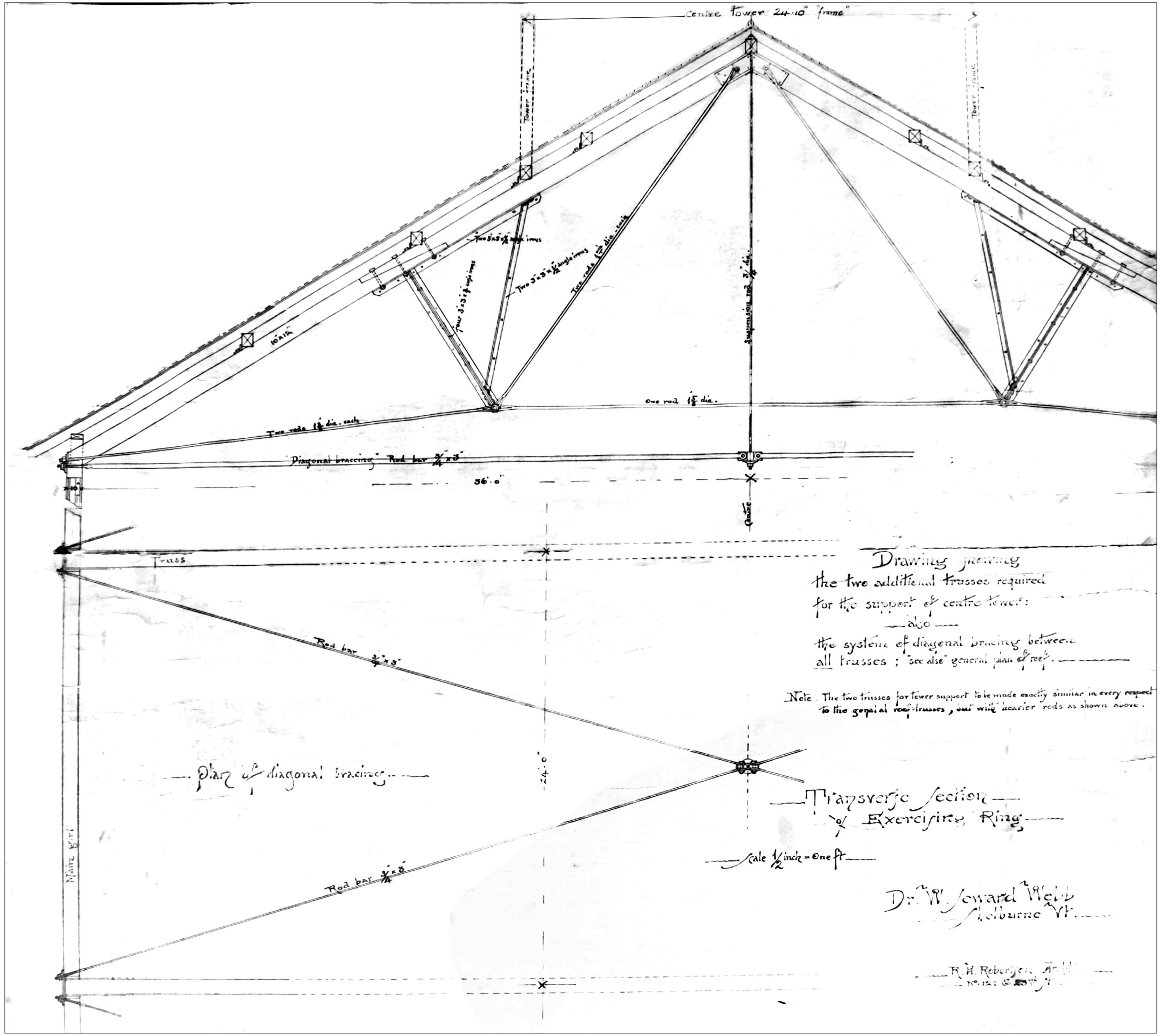
SCALE: 1/16" = 1'-0"

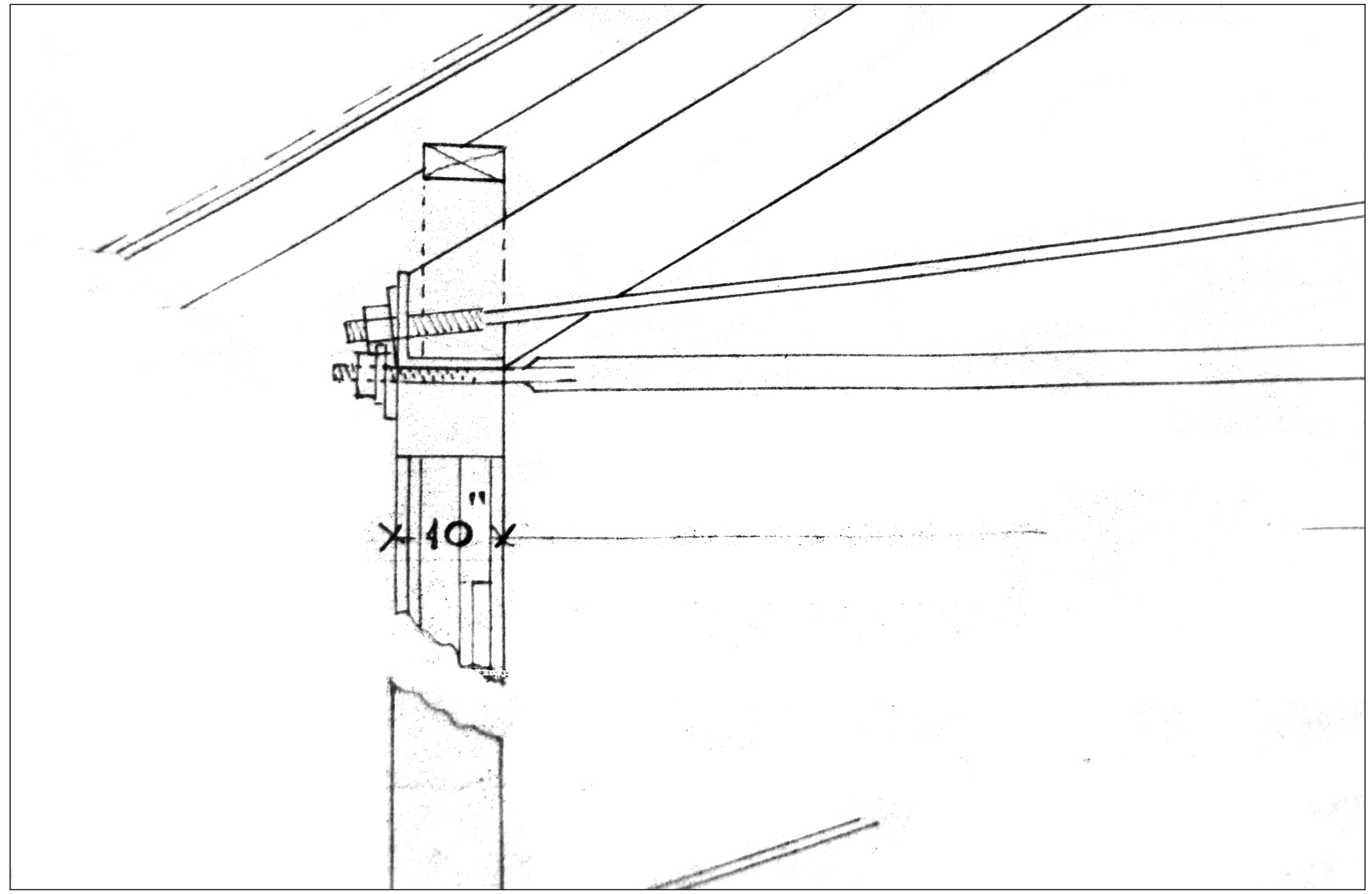
SHEET - 2

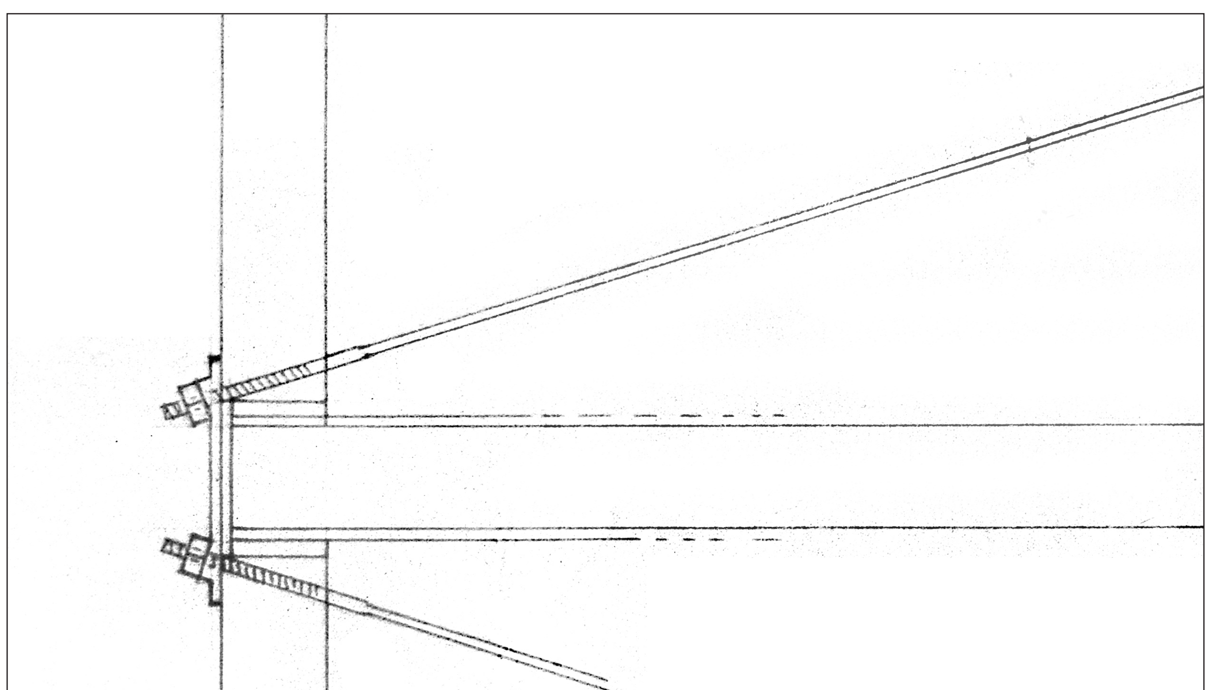


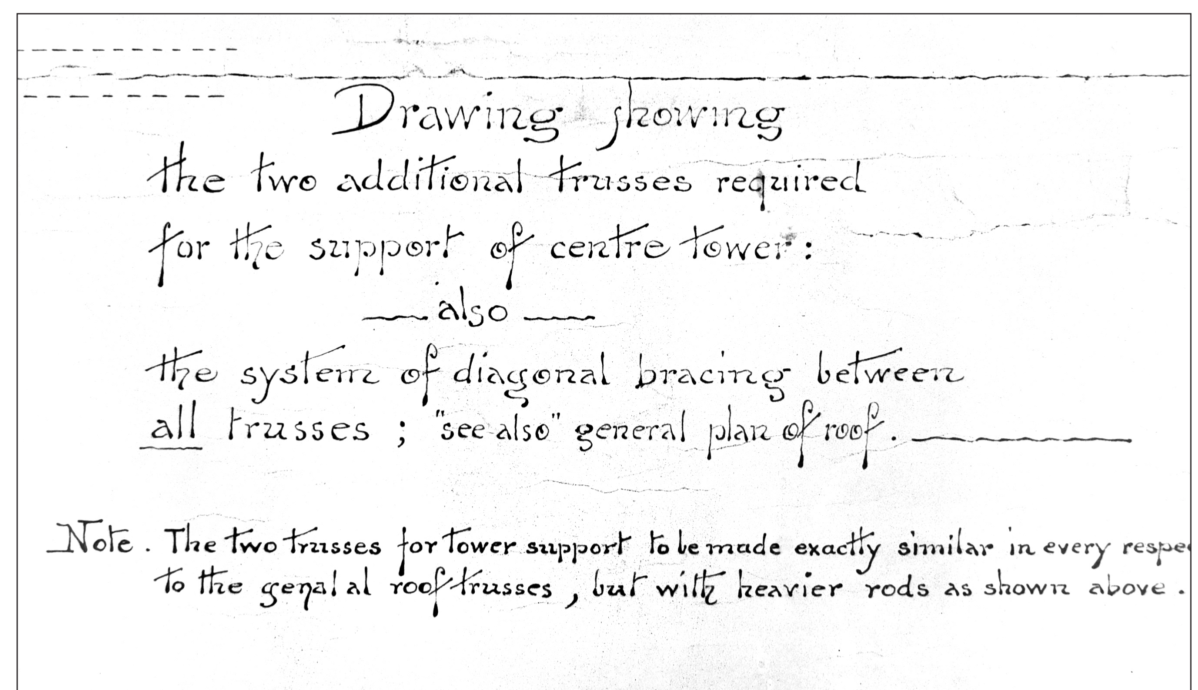
Plan of One half of main Roof construction, showing trussed purlins, ridges & valley rafters
Note. Purlins colored solid marked "A" to be trussed, with one strut in centre
Purlins " " " "B" as also the "Valley" rafters marked "C" to be trussed with two struts
The valley rafters "C" will be laid in the same plane as the purlins, and the purlins to be framed up against them and to be hung in bridle irons.
Scale 8 ft. = One Inch
Exercising Ring
Farm buildings
for
Dr. W. Seward Webb
Shelburne Vt.
R. H. Robertson Archt.
121 E 23rd St. N.Y.
centre line
24'-0"

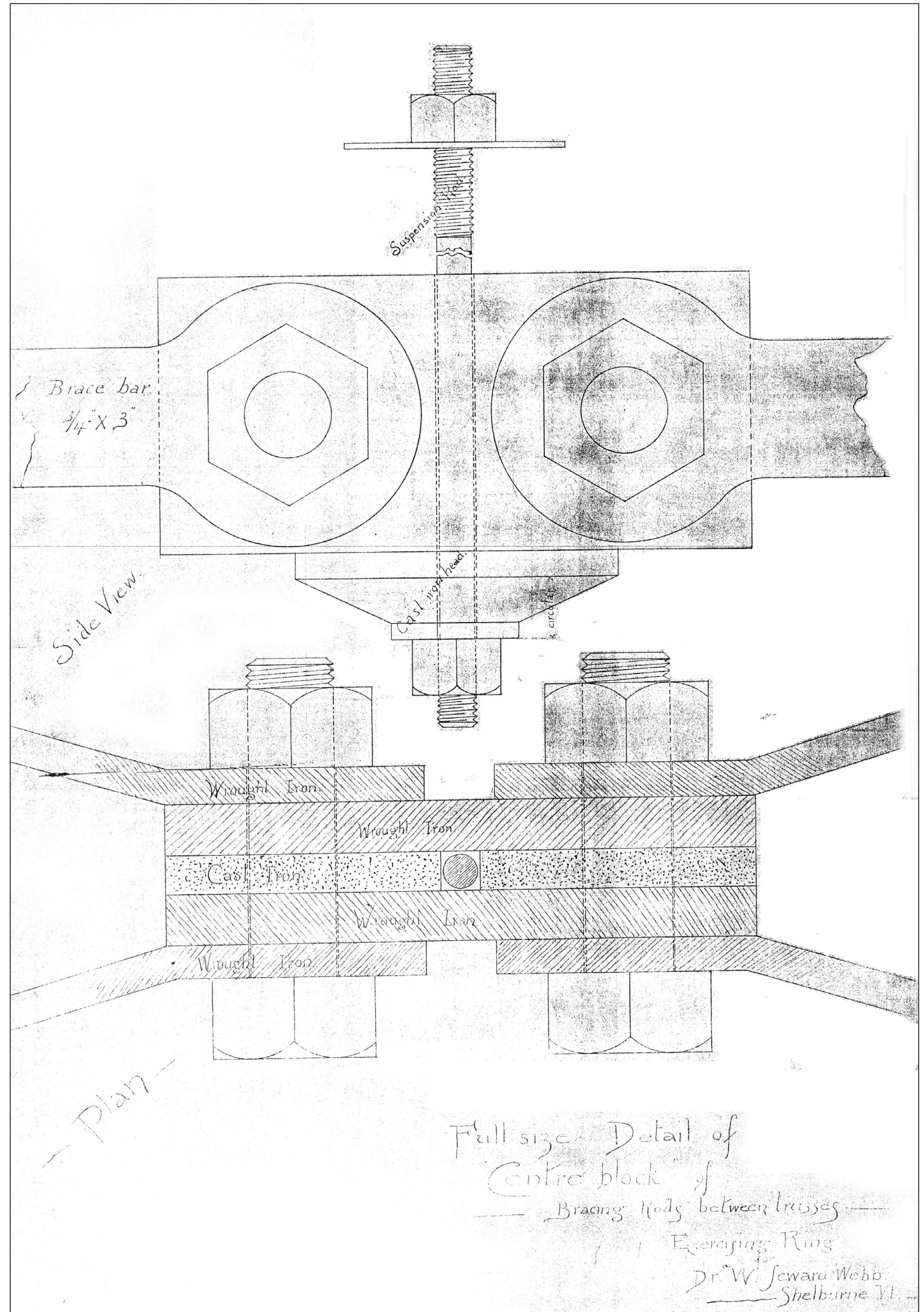

Plan of One half of main Roof construction
Showing "cross brace" Tie rods placed at foot of Main trusses, the centre ring buckle to be sustained from ridge purlin by vertical 3/4" rods.
Scale 8 ft. = One Inch
Exercising Ring
Farm buildings
for
Dr. W. Seward Webb
Shelburne Vt.
R. H. Robertson Archt.
121 E 23rd St N.Y.
Truss
Extra truss
Centre line
24'-0"

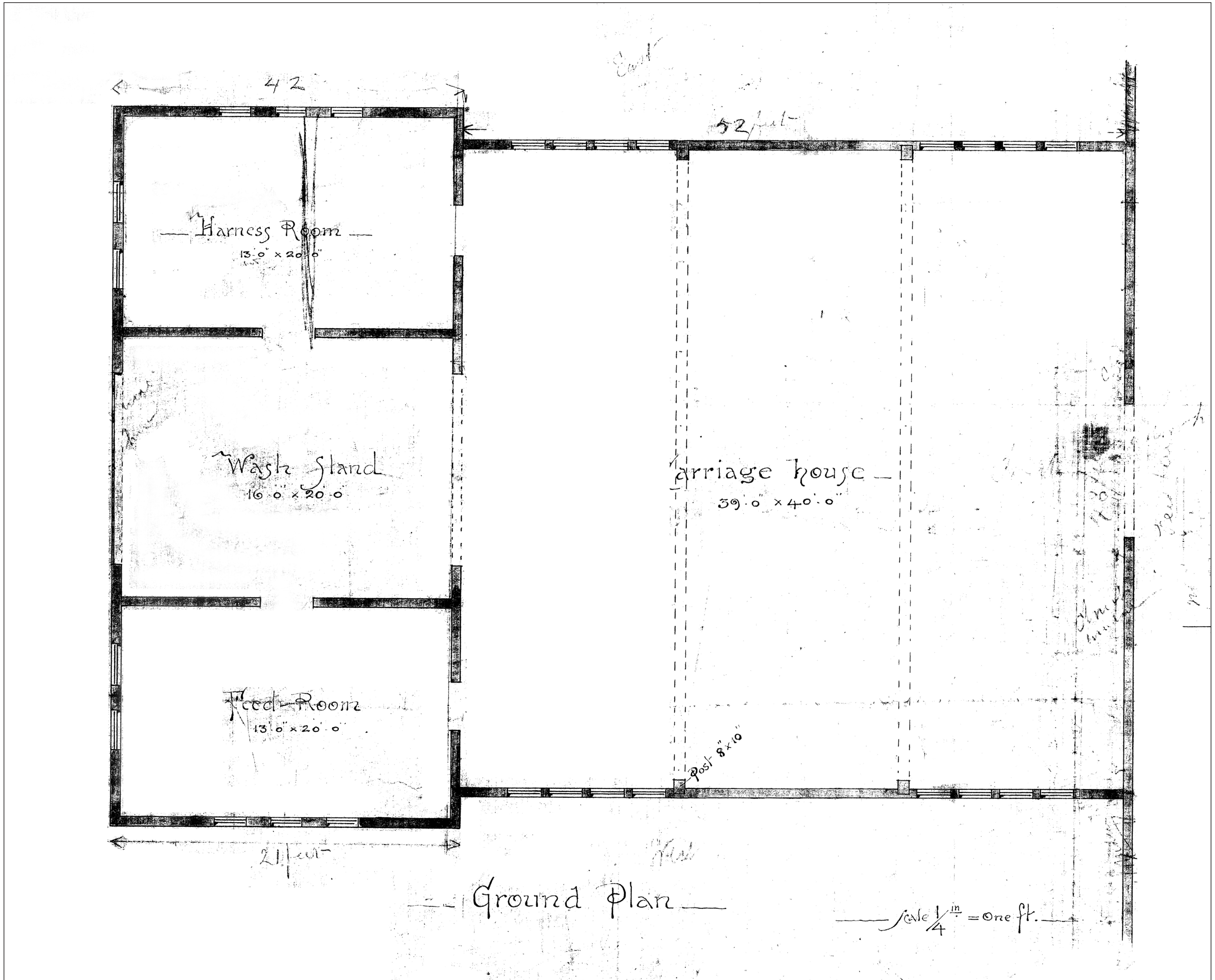

Centre Tower 24-10" frame"
One rod 1⅝" dia.
Two rods 1⅝" dia. each
"Diagonal bracing" Rod bar ¾" x 3"
56'-0"
Truss
Rod bar ¾" x 3"
Plan of diagonal bracing
24'-0"
Rod bar ¾" x 3"
Drawing showing
the two additional trusses required
for the support of centre tower:
also
the system of diagonal bracing between
all trusses; "see also" general plan of roof.
Note. The two trusses for tower support to be made exactly similar in every respect
to the general roof-trusses, but with heavier rods as shown above.
Transverse Section
of Exercising Ring
Scale ½ inch = One ft
Dr. W. Seward Webb
Shelburne Vt.
R. H. Robertson, Arch't

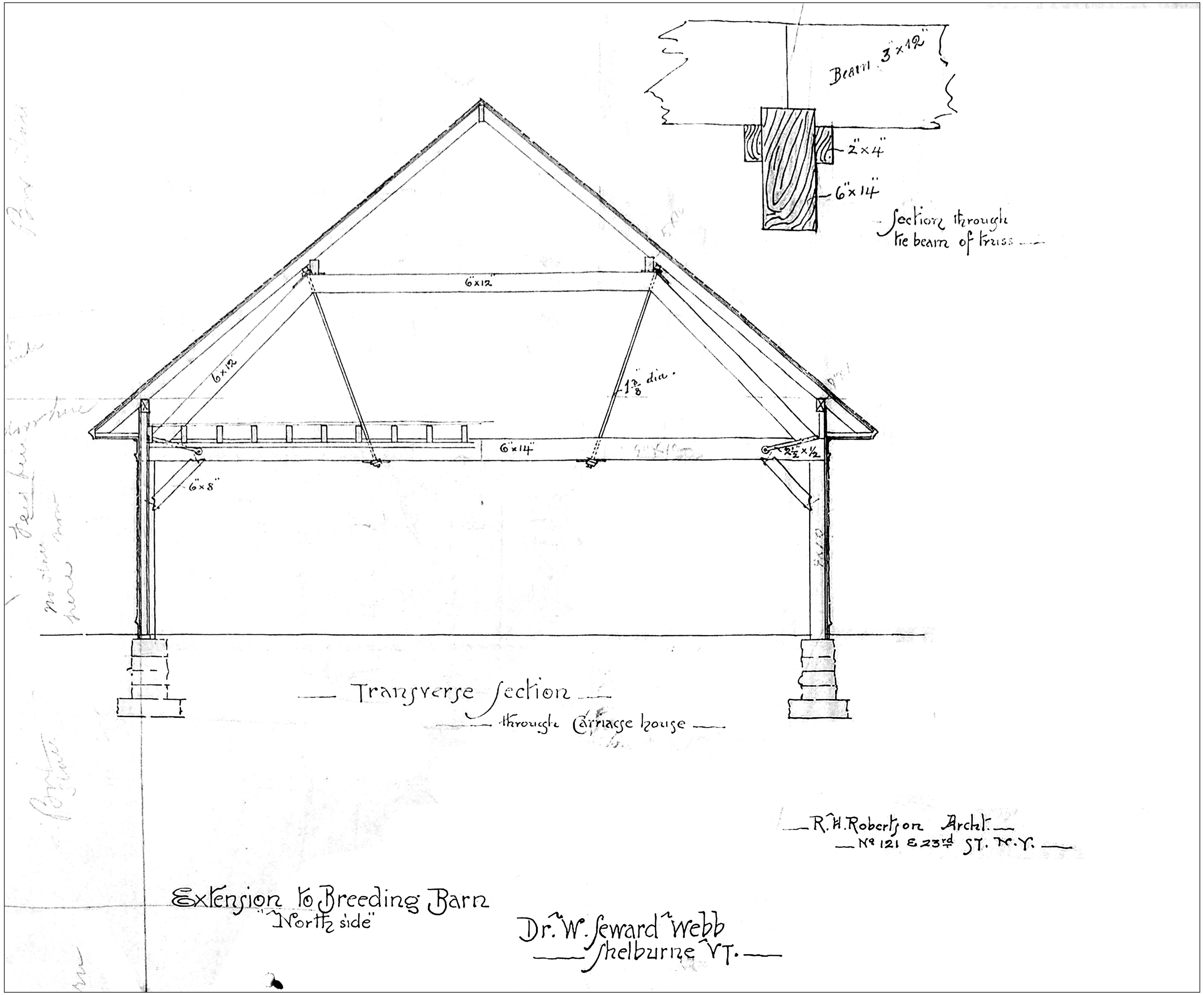

10"

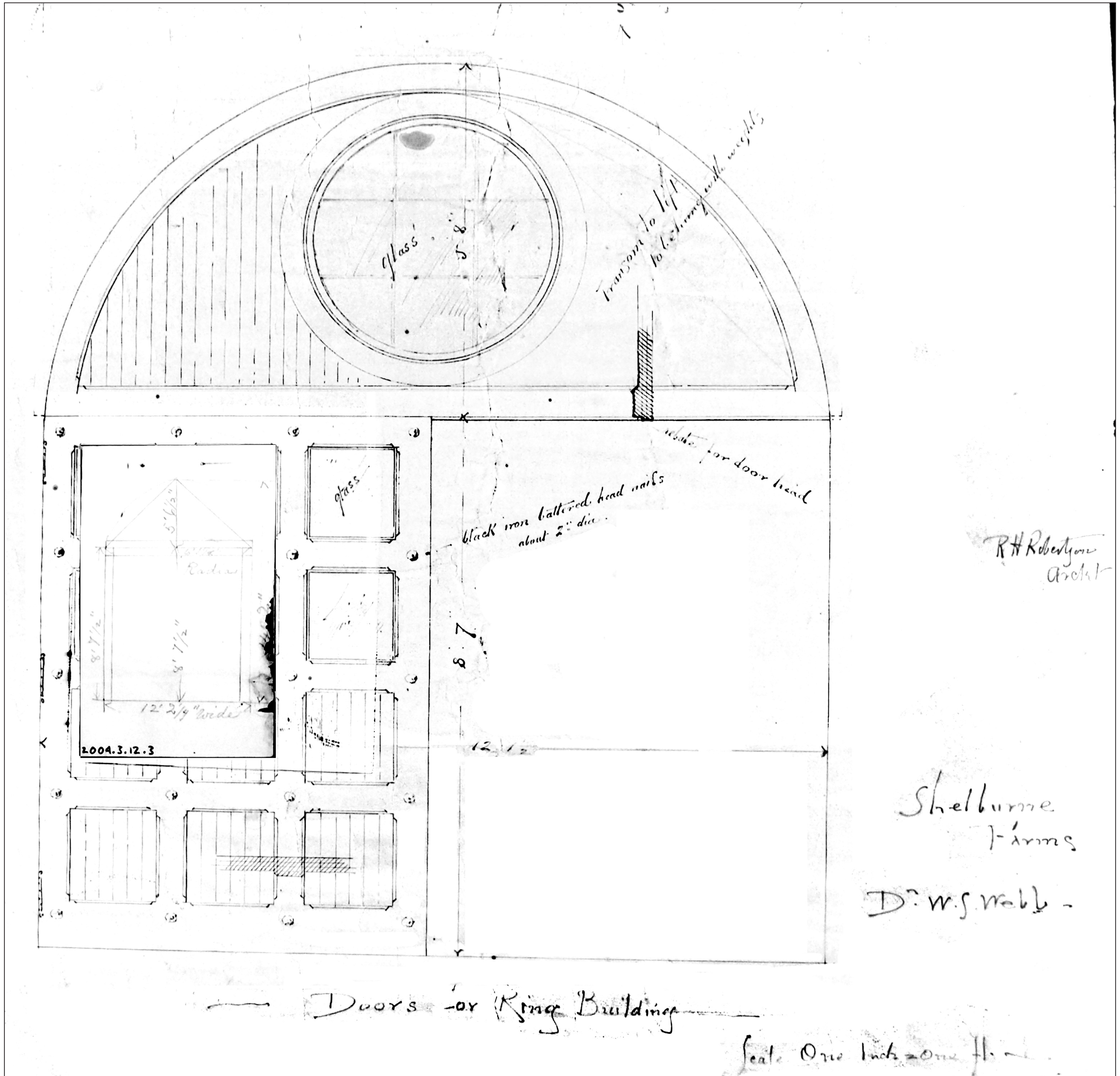


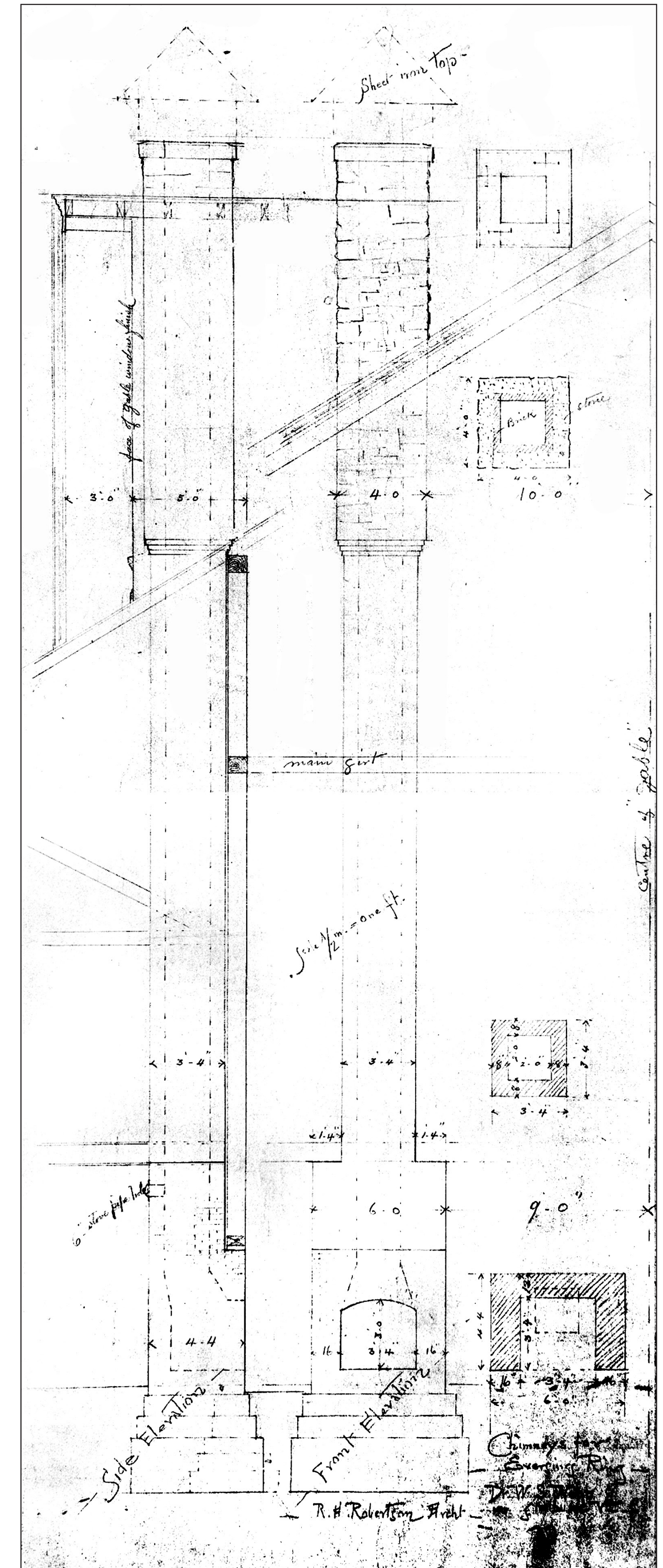

Drawing showing
the two additional trusses required
for the support of centre tower:
also
the system of diagonal bracing between
all trusses ; "see also" general plan of roof.
Note. The two trusses for tower support to be made exactly similar in every respe
to the genal al roof trusses, but with heavier rods as shown above.

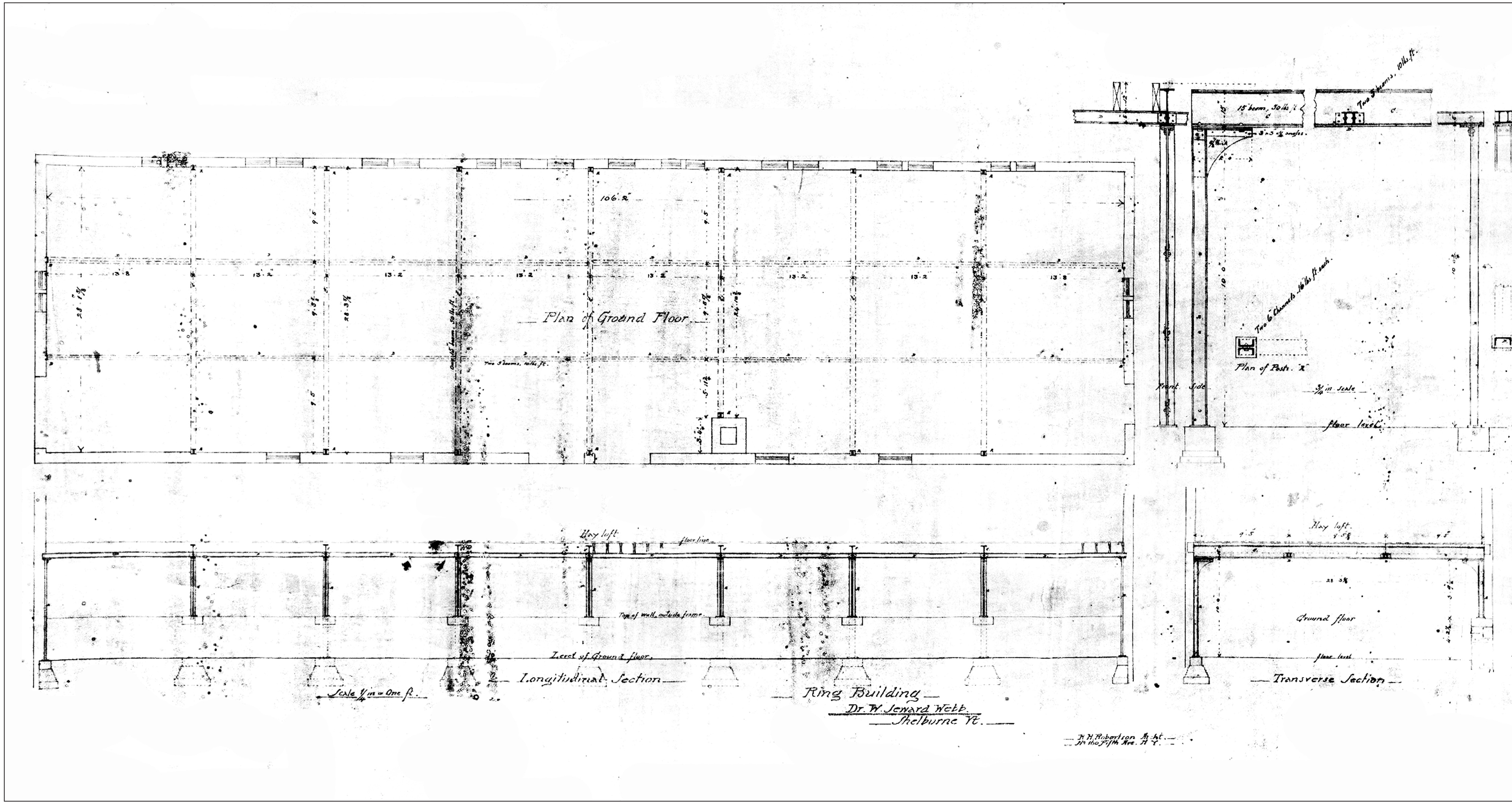

Suspension Rod
Brace bar
3/4"x 3"
Side View
Cast iron head
Wrought Iron
Wrought Iron
Cast Iron
Wrought Iron
Wrought Iron
Plan
Full size Detail of
Centre block of
Bracing Rods between trusses
Exercising Ring
Dr W. Seward Webb
Shelburne Vt.


42
East
52 feet
Harness Room
13'0" x 20'0"
Wash Stand
16'0" x 20'0"
Carriage house
39'0" x 40'0"
Feed Room
13'0" x 20'0"
Post 8"x10"
21 feet
Ground Plan
Scale 1/4 in = One ft.


Beam 3"x12"
2"x4"
6"x14"
Section through tie beam of truss
6"x12"
6"x12
1 3/8 dia.
6"x14"
6"x8"
Transverse Section
through Carriage house
R. H. Robertson Archt.
No 121 E 23rd ST. N.Y.
Extension to Breeding Barn
"North side"
Dr. W. Seward Webb
Shelburne VT.


glass 5'8"
black iron battered head nails about 2" dia.
8'7
12'1/2
12' 2/9" wide
2004.3.12.3
R H Robertson Archt
Shelburne Farms
Dr. W. S. Webb
Doors for Ring Building
Scale One Inch = one ft.


main girt
centre of gable
10.0
9.0
Side Elevation
Front Elevation
R. H. Robertson Archt


Plan of Ground Floor
106'-2"
Plan of Posts "X"
Front
Side
Floor level
Hay loft
Floor line
Top of wall outside frame
Level of Ground floor
Longitudinal Section
Scale 1/4 in = One ft.
Ground floor
Transverse Section
Ring Building
Dr. W. Seward Webb.
Shelburne Vt.
R. H. Robertson Archt.
160 Fifth Ave. N.Y.

Appendix C: HABS Drawings

The following appendix includes two-dimensional HABS-level drawings produced from laser scan data collected by the College of Architecture at Texas Tech University. The project design team felt it important to produce drawings that accurately record the building geometry (including displacements), and differences between the building “as-built” and the original drawings prepared by architect R.H. Robertson. Because of the wide-ranging applicability of the project’s investigation methods and repair strategies to the preservation of historic timber structures, and in order to make the building accessible to the widest possible audience, the drawings have been archived in the Library of Congress, Prints and Photographs Division.

Shelburne Farms

Breeding Barn

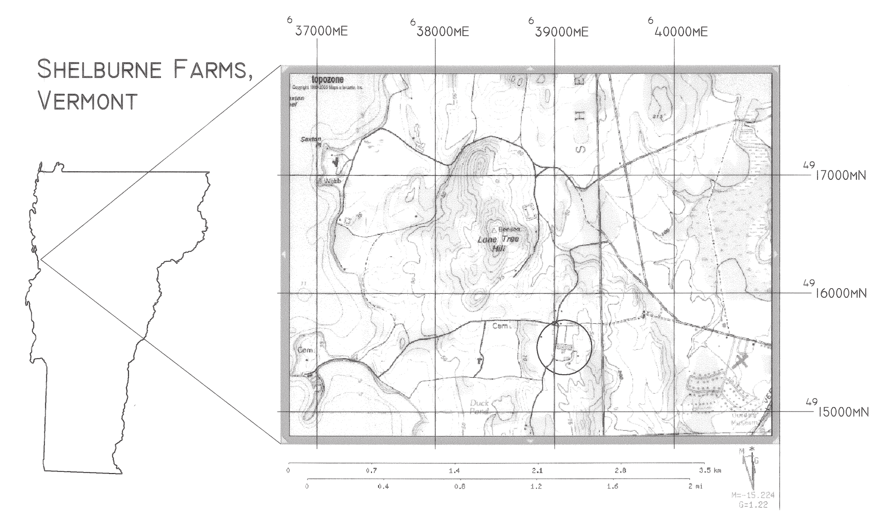
SHELBURNE FARMS, ORIGINALLY THE AGRICULTURAL ESTATE OF WILLIAM AND LILA WEBB, IS A 1400-ACRE NATIONAL HISTORIC LANDMARK DISTRICT LOCATED ON THE EASTERN EDGE OF LAKE CHAMPLAIN IN VERMONT. THE PROPERTY IS OWNED AND OPERATED BY A NONPROFIT ORGANIZATION DEVOTED TO THE CULTIVATION OF A CONSERVATION ETHIC THROUGH EDUCATION AND THE STEWARDSHIP OF NATURAL AND AGRICULTURAL RESOURCES.

THE PROPERTY IS SIGNIFICANT FOR ITS SCENIC LANDSCAPE AND FOR THE LANDMARK BUILDINGS SITUATED ON IT. THE LANDSCAPE DESIGN, CONTRIBUTED BY CELEBRATED LANDSCAPE ARCHITECT FREDERICK LAW OLMSTED, SR. (1822-1903), COMBINES THE PASTORAL AND PICTURESQUE IN THE TRADITION OF THE GREAT "ORNAMENTAL FARMS" OF NINETEENTH-CENTURY EUROPE. THE ESTATE ARCHITECTURE WAS DESIGNED BY NEW YORK ARCHITECT ROBERT HENDERSON ROBERTSON (1849-1919), A PROMINENT NINETEENTH-CENTURY DESIGNER OF MONUMENTAL ARCHITECTURE. TODAY, ROBERTSON IS BEST KNOWN FOR HIS PARK ROW BUILDING (1896-1899), WHICH WAS THE TALLEST BUILDINGS IN THE WORLD UNTIL 1908.

THE BUILDINGS AT SHELBURNE FARMS REPRESENT ROBERTSON'S MOST SIGNIFICANT ESTATE COMMISSION. SHELBURNE FARMS IS DOMINATED BY FOUR ENORMOUS BUILDINGS THAT COMBINE QUEEN ANNE AND SHINGLE STYLE FEATURES, AND WERE THE CENTERS OF LIFE ON THE MODEL ESTATE. THEY INCLUDE SHELBURNE HOUSE (1888, WITH SIGNIFICANT RENOVATIONS BY 1900), A TUDOR REVIVAL MANSION WHICH SERVED AS THE WEBB'S COUNTRY RESIDENCE; THE FARM BARN (1888-1890), WHICH WAS THE AGRICULTURAL HEADQUARTERS OF THE ESTATE; THE COACH BARN (1902), THE TRANSPORTATION CENTER OF THE ESTATE AND ONE OF ROBERTSON'S LAST MAJOR EFFORTS; AND THE BREEDING BARN (1891), WHICH SERVED AS THE CENTER OF DR. WEBB'S HORSE-BREEDING EFFORTS.

THE BREEDING BARN CONSISTS OF THE ORIGINAL MAIN BLOCK 108' - 8 5/8" WIDE BY 418' - 8 1/2" LONG, WITH A TWO-STORY ANNEX. THE BUILDING IS TIMBER-FRAMED, SUPPORTED ON A REDSTONE FOUNDATION, AND CLAD IN WOODEN SHINGLES. AT THE CENTER OF THE BUILDING, AN UNBROKEN CATHEDRAL-LIKE SPACE MEASURING 72' - 7 1/2" WIDE AND 359' - 1 1/2" FEET LONG ONCE HOUSED THE RIDING RING. SURROUNDED BY STABLES, THE RING WAS LIT BY SIX LARGE GLAZED DORMERS AND AN ENORMOUS CENTER LANTERN SUPPORTED 53' - 7/8" FEET ABOVE THE FLOOR. EXPOSED COMPOSITE QUEEN POST TRUSSES AND TRUSSED PURLINS SUPPORT THE ROOF EXPANSE, COMPRISING A BEAUTIFUL AND HIGHLY EFFICIENT ROOF STRUCTURE OF TIMBER, WROUGHT IRON, AND STEEL.

THE SHELBURNE FARMS BREEDING BARN DOCUMENTATION PROJECT WAS PART OF AN ONGOING EFFORT BY SHELBURNE FARMS TO PRESERVE THESE OUTSTANDING REPRESENTATIVES OF TURN OF THE CENTURY ESTATE BUILDINGS AND LANDSCAPE. THE PROJECT WAS INITIATED THROUGH THE GETTY GRANT PROGRAM AND ADDITIONAL SUPPORT WAS PROVIDED BY SHELBURNE FARMS' PRIVATE DONORS, THE OAKLAND FOUNDATION, AND THE CYNTHIA WOODS MITCHELL FUND. THE PROJECT WAS IMPLEMENTED UNDER THE DIRECTION OF DOUGLAS PORTER (UNIVERSITY OF VERMONT) AND PROFESSOR ELIZABETH LOUDEN (TEXAS TECH UNIVERSITY), WITH RESEARCH ASSISTANCE FROM STUDENTS HANNAH AIKIN, LAUREN CORTINAZ, OLIVER COX, BRANDON HAY, AND FELICIA SANTIAGO, AND THE COLLEGE OF ARCHITECTURE AT TEXAS TECH UNIVERSITY. KAREN HUGHES, PRESERVATION SPECIALIST AT HHM, INC. ALSO CONTRIBUTED TIME AND EXPERTISE. THREE DIMENSIONAL DATA FOR THE DRAWINGS WAS COLLECTED BY THE LEICA HDS3000 THAT SERVED AS A BASIS FOR DRAWINGS. RESEARCH AND HISTORIC INFORMATION WAS PROVIDED BY DOUGLAS PORTER.



UTM MAP
UTM REFERENCE - WILLSBORO BAY, VT, USGS
ZONE 18
63 9135ME 49 155274MN (NAD27)

SCALE: 0 1056 2112 4224 FEET

2008 CHARLES E. PETERSON PRIZE ENTRY

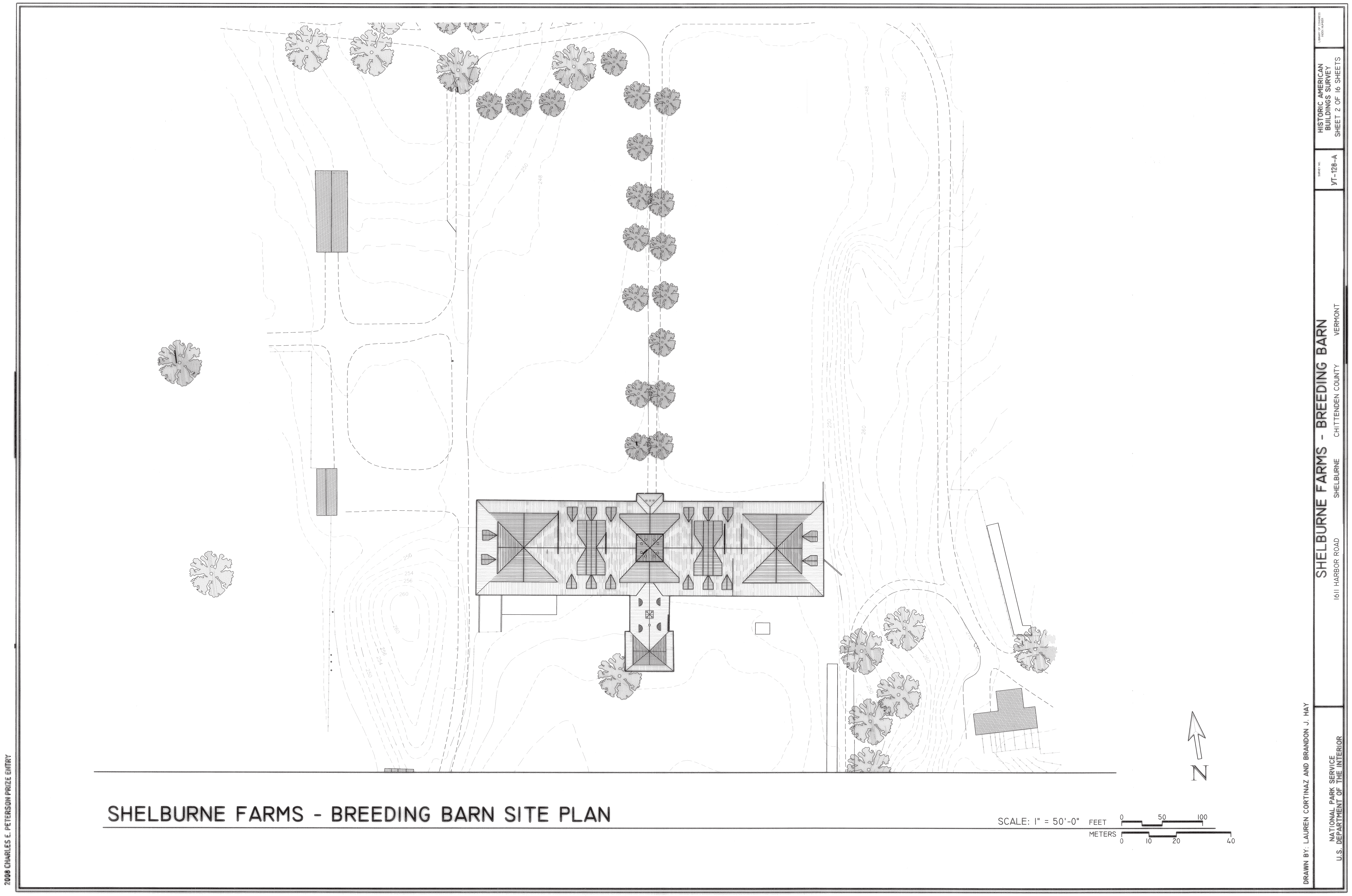
DRAWN BY: FELICIA SANTIAGO

NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR

SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT

VT-128-A

HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 1 OF 16 SHEETS



SHELBURNE FARMS - BREEDING BARN SITE PLAN
SCALE: 1" = 50'-0"
FEET 0 50 100
METERS 0 10 20 40
N
DRAWN BY: LAUREN CORTINAZ AND BRANDON J. HAY
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD
SHELBURNE
CHITTENDEN COUNTY
VERMONT
SURVEY NO.
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 2 OF 16 SHEETS
LIBRARY OF CONGRESS INDEX NUMBER
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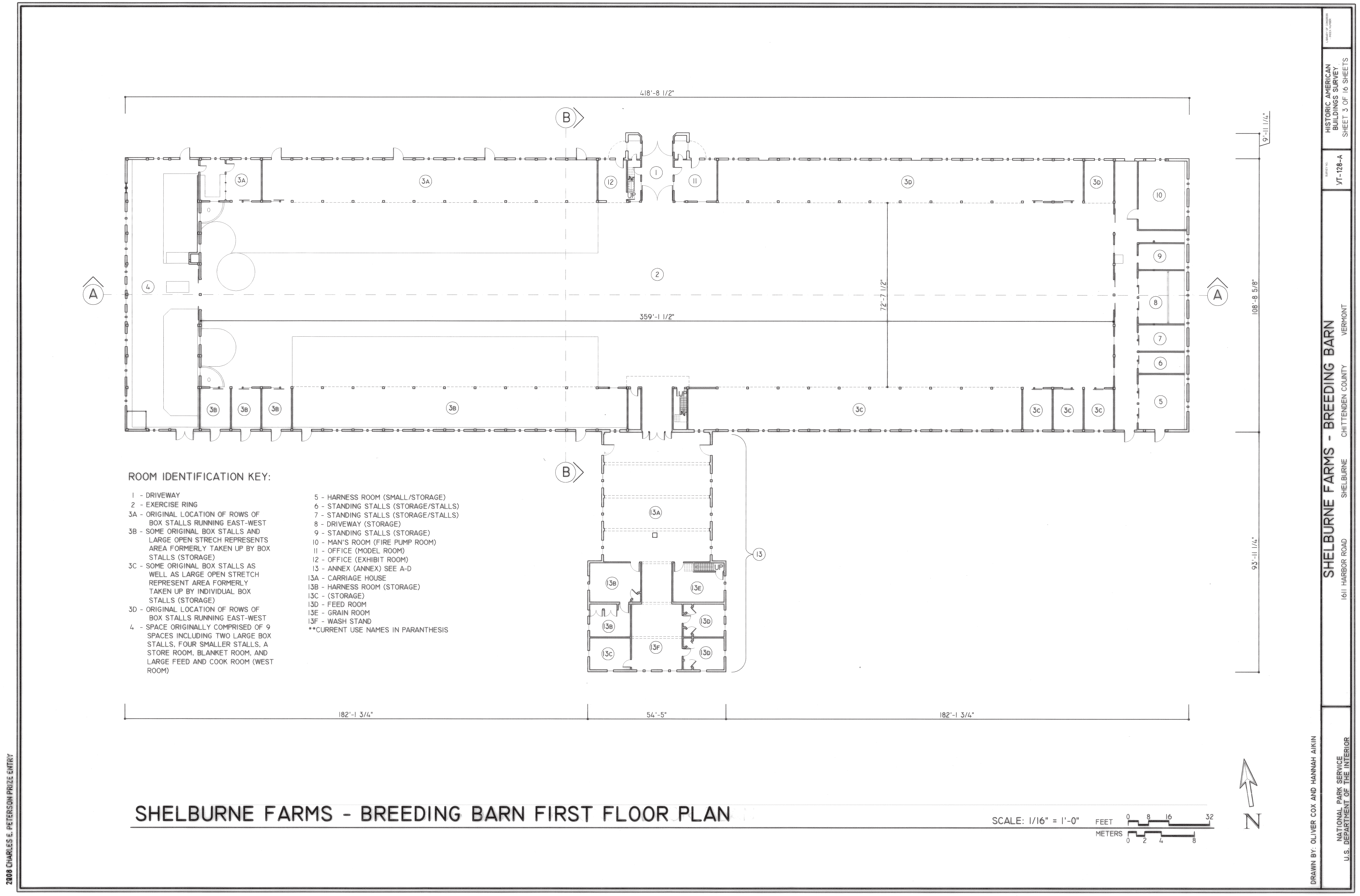
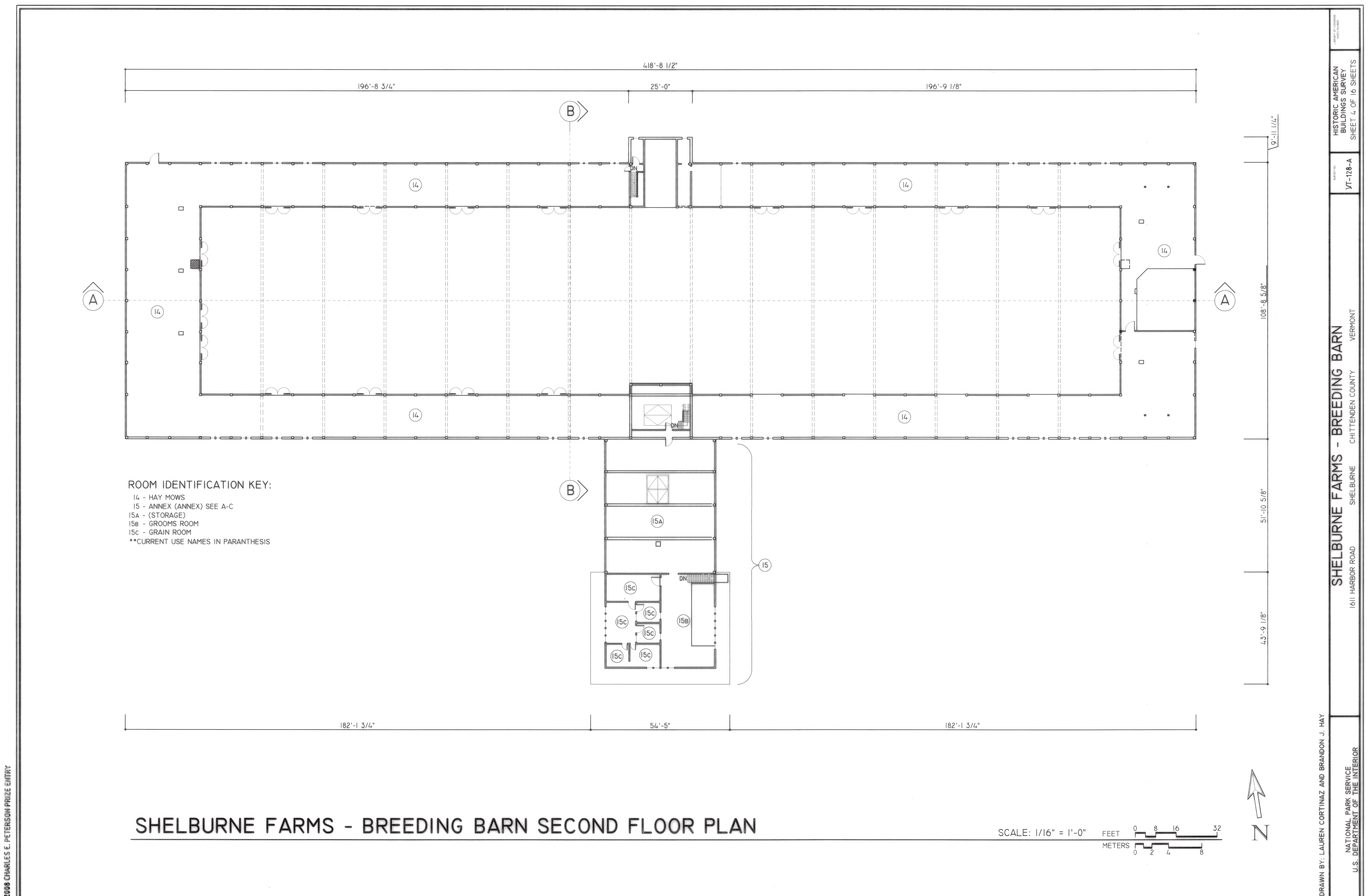

418'-8 1/2"
359'-1 1/2"
72'-7 1/2"
108'-8 5/8"
9'-11 1/4"
93'-11 1/4"
182'-1 3/4"
54'-5"
182'-1 3/4"
ROOM IDENTIFICATION KEY:
1 - DRIVEWAY
2 - EXERCISE RING
3A - ORIGINAL LOCATION OF ROWS OF BOX STALLS RUNNING EAST-WEST
3B - SOME ORIGINAL BOX STALLS AND LARGE OPEN STRECH REPRESENTS AREA FORMERLY TAKEN UP BY BOX STALLS (STORAGE)
3C - SOME ORIGINAL BOX STALLS AS WELL AS LARGE OPEN STRETCH REPRESENT AREA FORMERLY TAKEN UP BY INDIVIDUAL BOX STALLS (STORAGE)
3D - ORIGINAL LOCATION OF ROWS OF BOX STALLS RUNNING EAST-WEST
4 - SPACE ORIGINALLY COMPRISED OF 9 SPACES INCLUDING TWO LARGE BOX STALLS, FOUR SMALLER STALLS, A STORE ROOM, BLANKET ROOM, AND LARGE FEED AND COOK ROOM (WEST ROOM)
5 - HARNESS ROOM (SMALL/STORAGE)
6 - STANDING STALLS (STORAGE/STALLS)
7 - STANDING STALLS (STORAGE/STALLS)
8 - DRIVEWAY (STORAGE)
9 - STANDING STALLS (STORAGE)
10 - MAN'S ROOM (FIRE PUMP ROOM)
11 - OFFICE (MODEL ROOM)
12 - OFFICE (EXHIBIT ROOM)
13 - ANNEX (ANNEX) SEE A-D
13A - CARRIAGE HOUSE
13B - HARNESS ROOM (STORAGE)
13C - (STORAGE)
13D - FEED ROOM
13E - GRAIN ROOM
13F - WASH STAND
**CURRENT USE NAMES IN PARANTHESIS
SHELBURNE FARMS - BREEDING BARN FIRST FLOOR PLAN
SCALE: 1/16" = 1'-0"
FEET 0 8 16 32
METERS 0 2 4 8
N
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: OLIVER COX AND HANNAH AIKIN
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
SURVEY NO. VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 3 OF 16 SHEETS
LIBRARY OF CONGRESS INDEX NUMBER
IF REPRODUCED, PLEASE CREDIT THE HISTORIC AMERICAN ENGINEERING RECORD, NATIONAL PARK SERVICE, NAME OF DELINEATOR, DATE OF DRAWING


418'-8 1/2"
196'-8 3/4"
25'-0"
196'-9 1/8"
9'-11 1/4"
108'-8 5/8"
51'-10 5/8"
43'-9 1/8"
182'-1 3/4"
54'-5"
182'-1 3/4"
ROOM IDENTIFICATION KEY:
14 - HAY MOWS
15 - ANNEX (ANNEX) SEE A-C
15A - (STORAGE)
15B - GROOMS ROOM
15C - GRAIN ROOM
**CURRENT USE NAMES IN PARANTHESIS
SHELBURNE FARMS - BREEDING BARN SECOND FLOOR PLAN
SCALE: 1/16" = 1'-0"
FEET
METERS
N
DRAWN BY: LAUREN CORTINAZ AND BRANDON J. HAY
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
SURVEY NO.
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 4 OF 16 SHEETS
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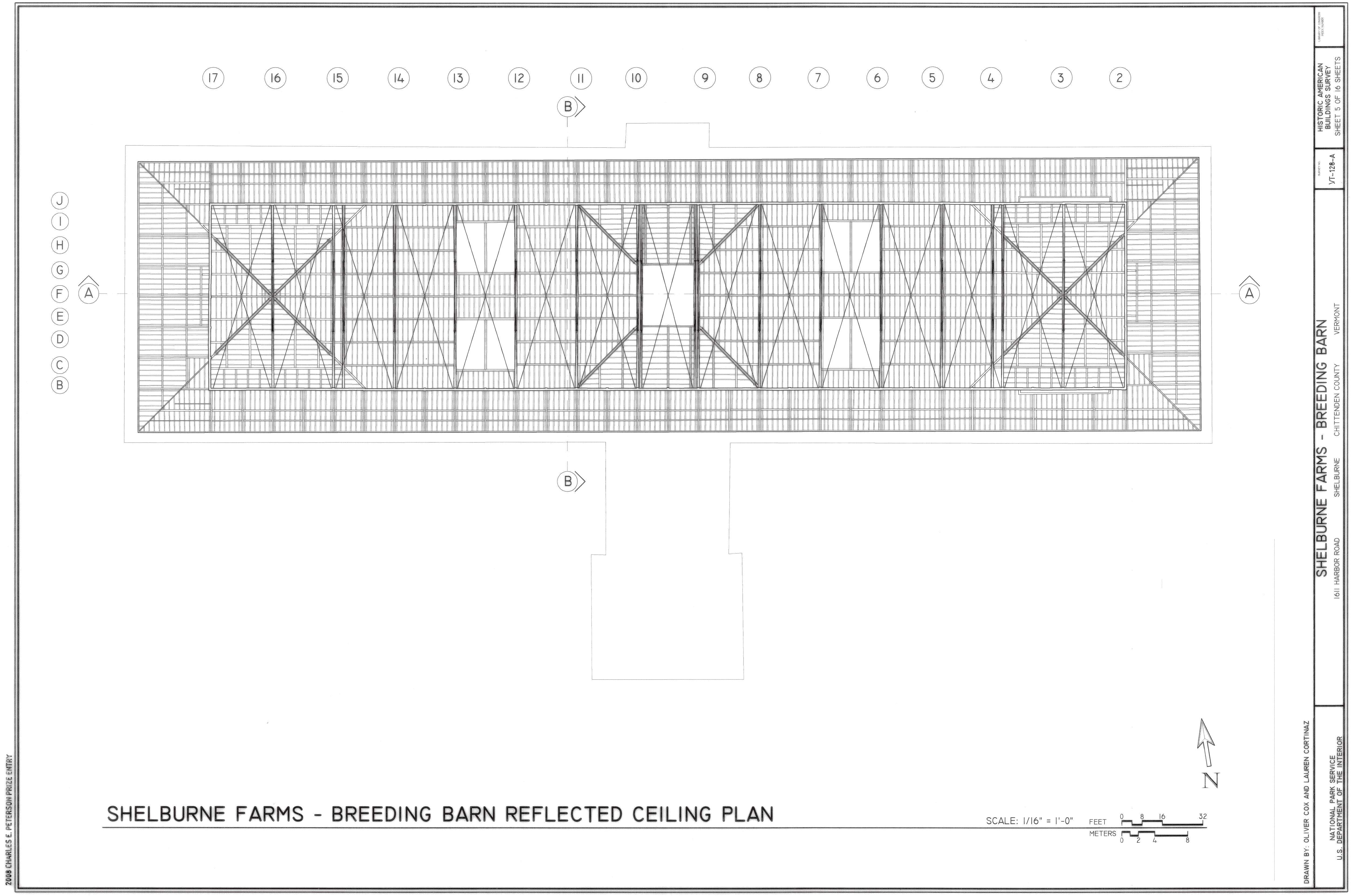
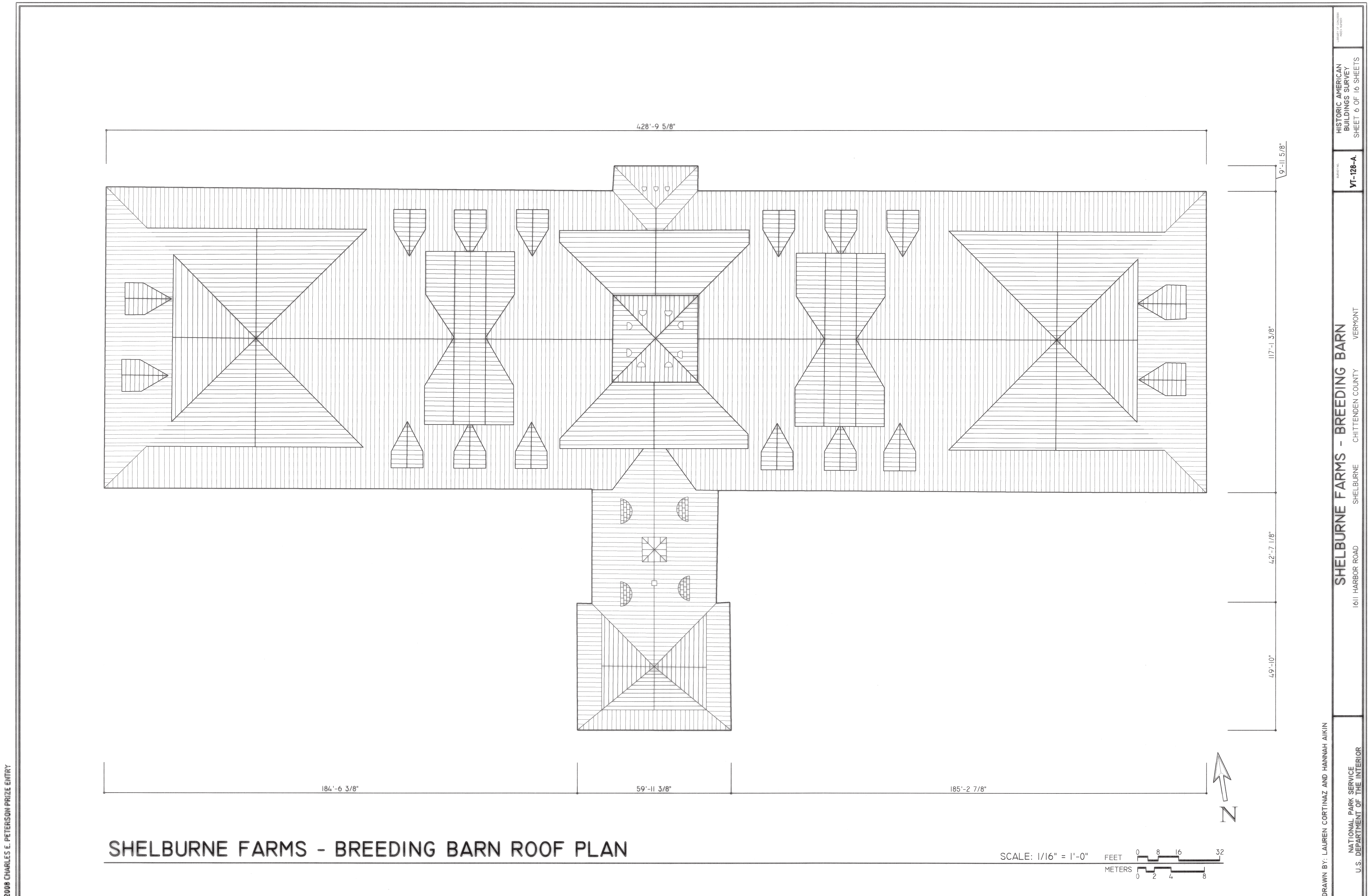
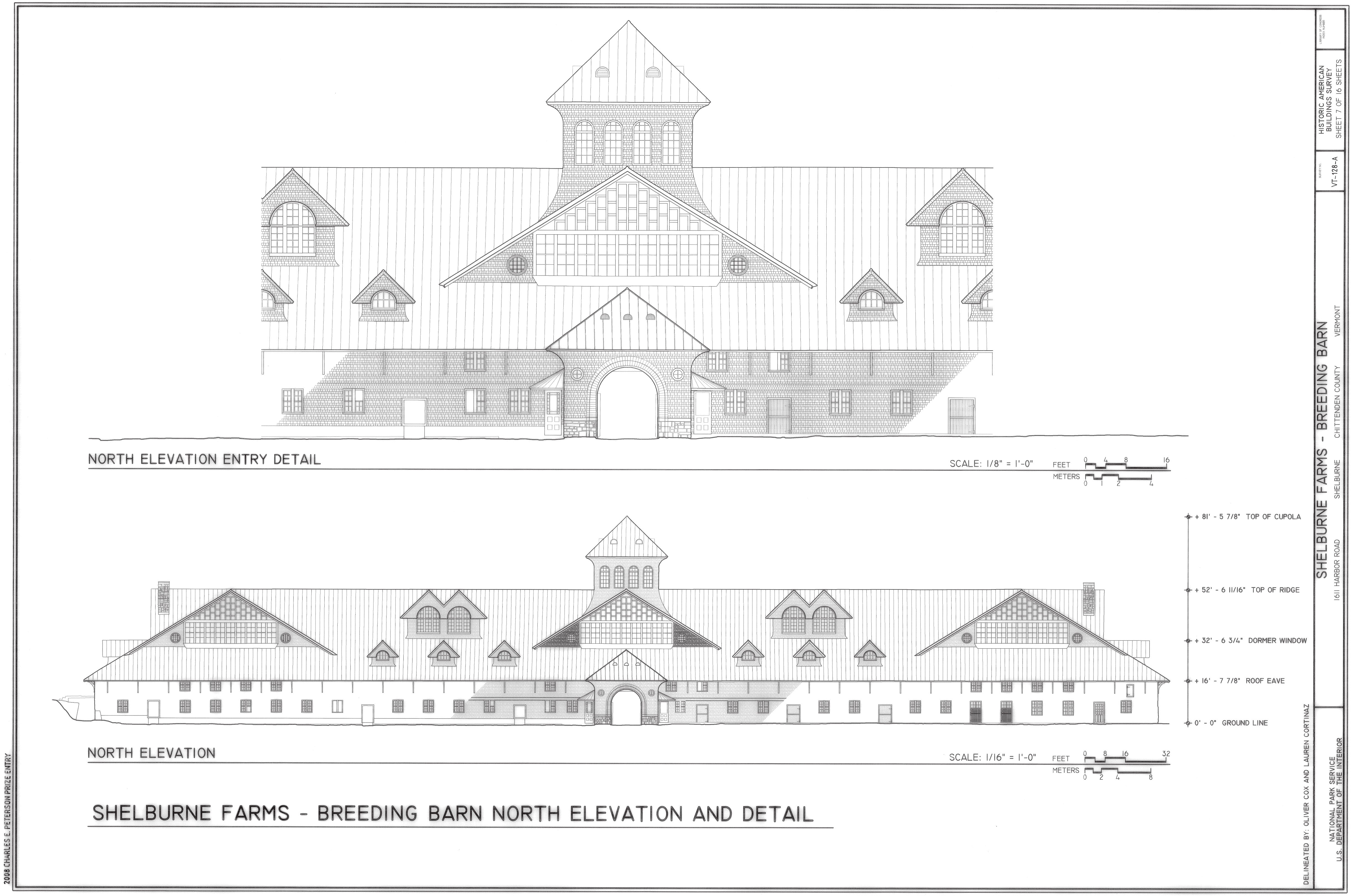
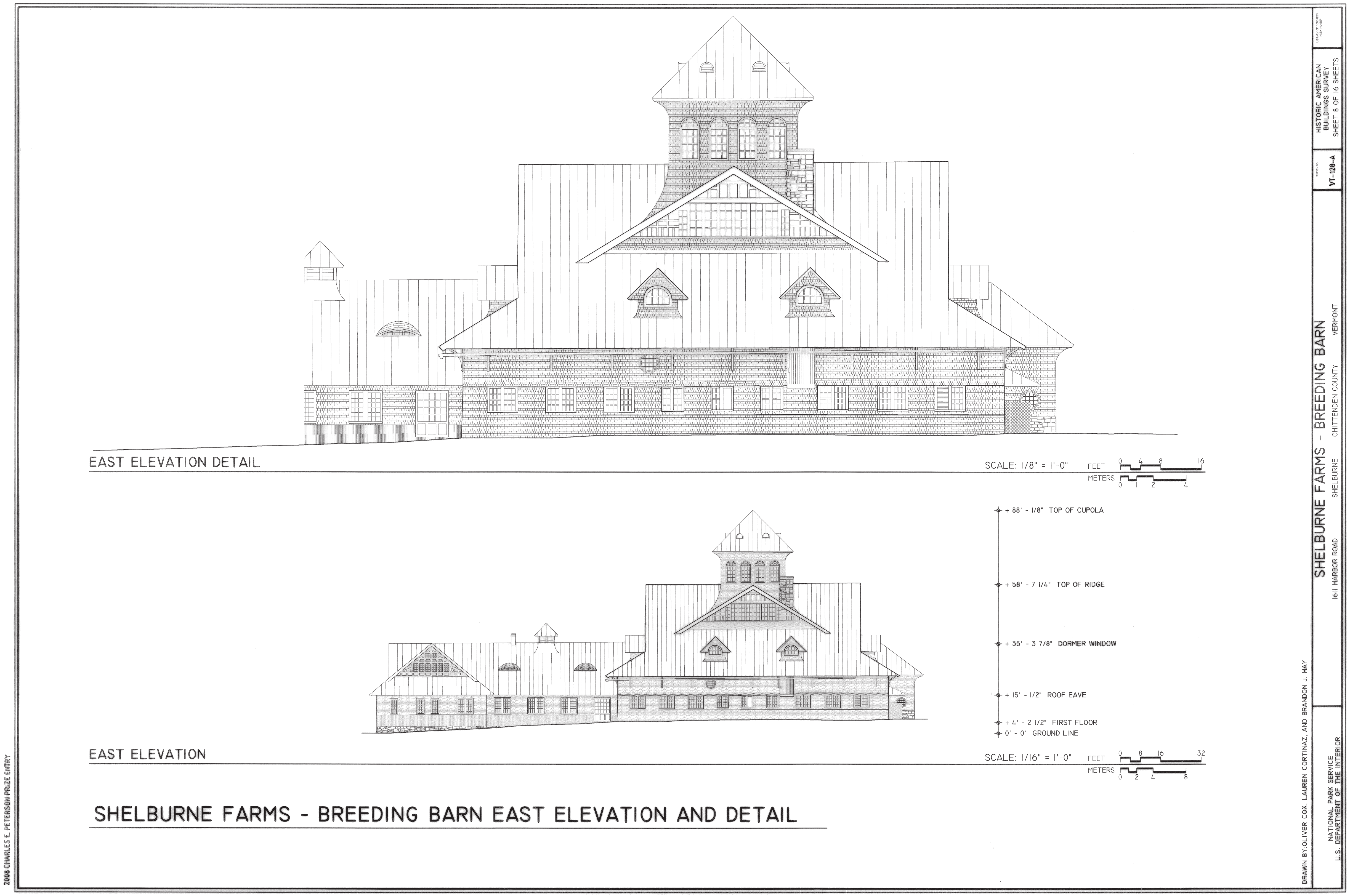
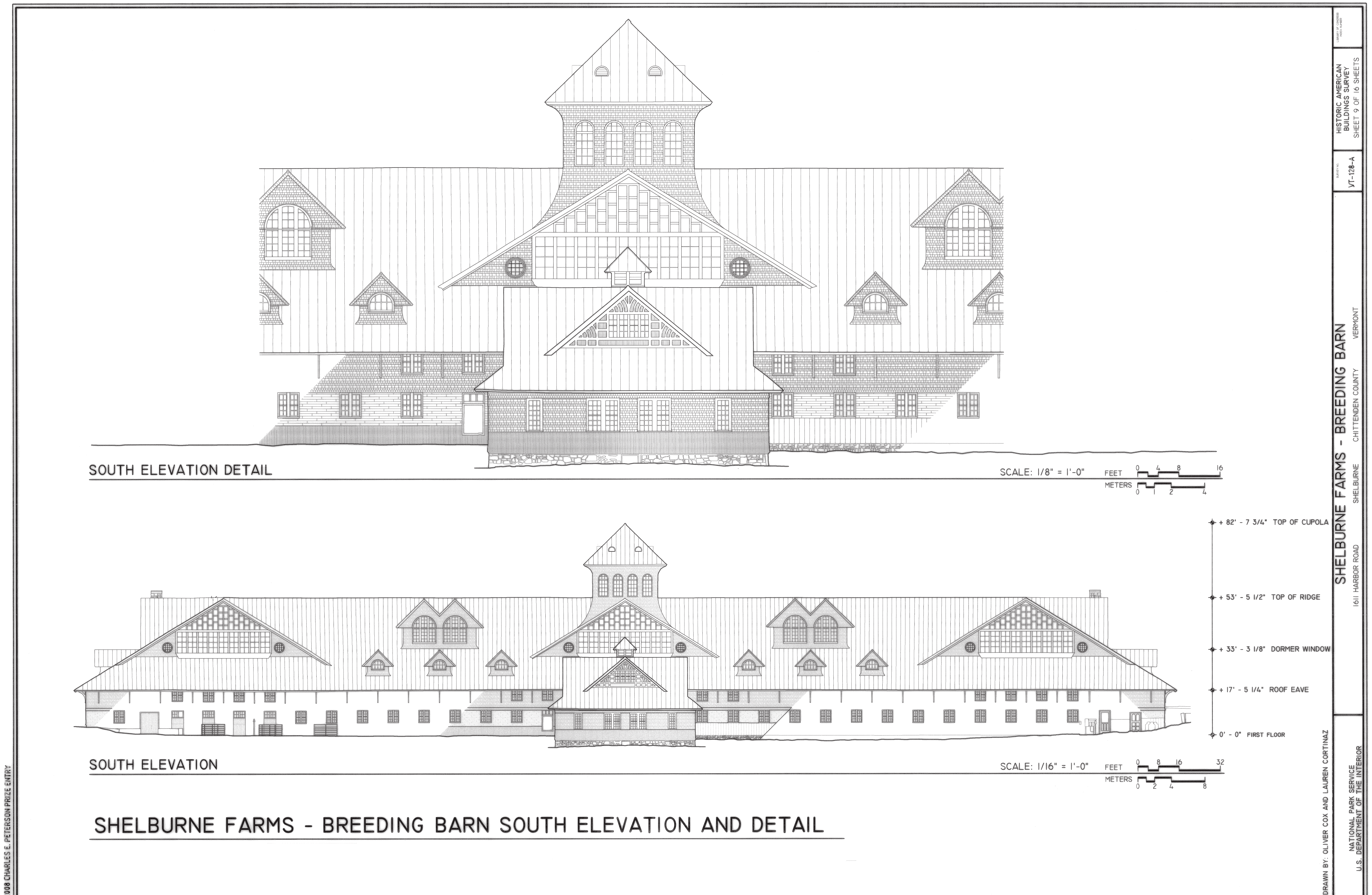

17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2
J I H G F E D C B
A
B
SHELBURNE FARMS - BREEDING BARN REFLECTED CEILING PLAN
SCALE: 1/16" = 1'-0"
FEET 0 8 16 32
METERS 0 2 4 8
N
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: OLIVER COX AND LAUREN CORTINAZ
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
SURVEY NO.
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 5 OF 16 SHEETS
LIBRARY OF CONGRESS INDEX NUMBER


428'-9 5/8"
9'-11 5/8"
117'-1 3/8"
42'-7 1/8"
49'-10"
184'-6 3/8"
59'-11 3/8"
185'-2 7/8"
N
SHELBURNE FARMS - BREEDING BARN ROOF PLAN
SCALE: 1/16" = 1'-0"
FEET 0 8 16 32
METERS 0 2 4 8
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: LAUREN CORTINAZ AND HANNAH AIKIN
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 6 OF 16 SHEETS


NORTH ELEVATION ENTRY DETAIL
SCALE: 1/8" = 1'-0"
FEET 0 4 8 16
METERS 0 1 2 4
+ 81' - 5 7/8" TOP OF CUPOLA
+ 52' - 6 11/16" TOP OF RIDGE
+ 32' - 6 3/4" DORMER WINDOW
+ 16' - 7 7/8" ROOF EAVE
0' - 0" GROUND LINE
NORTH ELEVATION
SCALE: 1/16" = 1'-0"
FEET 0 8 16 32
METERS 0 2 4 8
SHELBURNE FARMS - BREEDING BARN NORTH ELEVATION AND DETAIL
2008 CHARLES E. PETERSON PRIZE ENTRY
DELINEATED BY: OLIVER COX AND LAUREN CORTINAZ
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 7 OF 16 SHEETS


EAST ELEVATION DETAIL
SCALE: 1/8" = 1'-0"
FEET 0 4 8 16
METERS 0 1 2 4
+ 88' - 1/8" TOP OF CUPOLA
+ 58' - 7 1/4" TOP OF RIDGE
+ 35' - 3 7/8" DORMER WINDOW
+ 15' - 1/2" ROOF EAVE
+ 4' - 2 1/2" FIRST FLOOR
0' - 0" GROUND LINE
EAST ELEVATION
SCALE: 1/16" = 1'-0"
FEET 0 8 16 32
METERS 0 2 4 8
SHELBURNE FARMS - BREEDING BARN EAST ELEVATION AND DETAIL
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: OLIVER COX, LAUREN CORTINAZ, AND BRANDON J. HAY
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD
SHELBURNE
CHITTENDEN COUNTY
VERMONT
SURVEY NO.
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 8 OF 16 SHEETS
LIBRARY OF CONGRESS INDEX NUMBER


SOUTH ELEVATION DETAIL
SCALE: 1/8" = 1'-0"
FEET 0 4 8 16
METERS 0 1 2 4
+ 82' - 7 3/4" TOP OF CUPOLA
+ 53' - 5 1/2" TOP OF RIDGE
+ 33' - 3 1/8" DORMER WINDOW
+ 17' - 5 1/4" ROOF EAVE
0' - 0" FIRST FLOOR
SOUTH ELEVATION
SCALE: 1/16" = 1'-0"
FEET 0 8 16 32
METERS 0 2 4 8
SHELBURNE FARMS - BREEDING BARN SOUTH ELEVATION AND DETAIL
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: OLIVER COX AND LAUREN CORTINAZ
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
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VT-128-A
HISTORIC AMERICAN
BUILDINGS SURVEY
SHEET 9 OF 16 SHEETS
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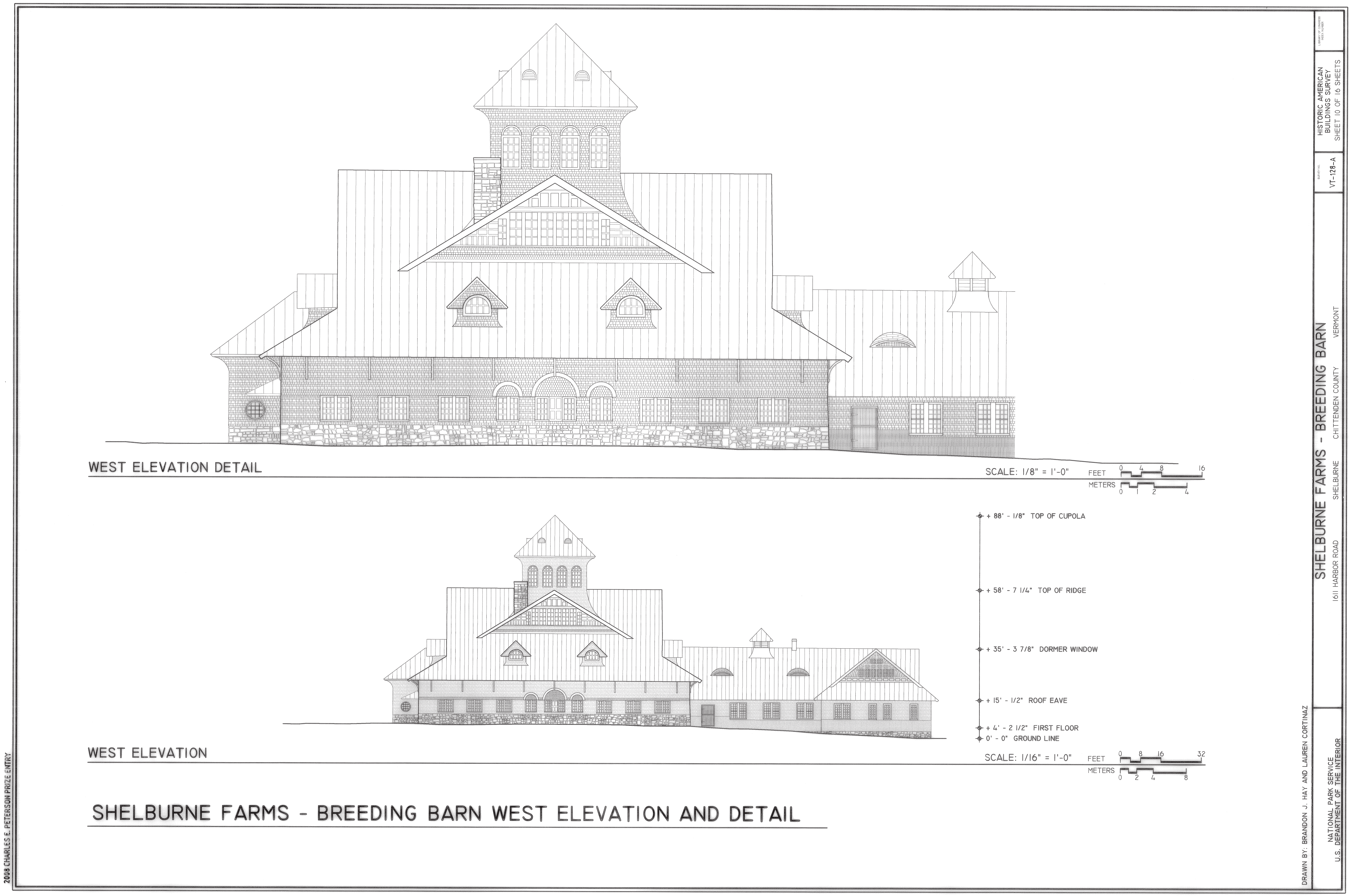
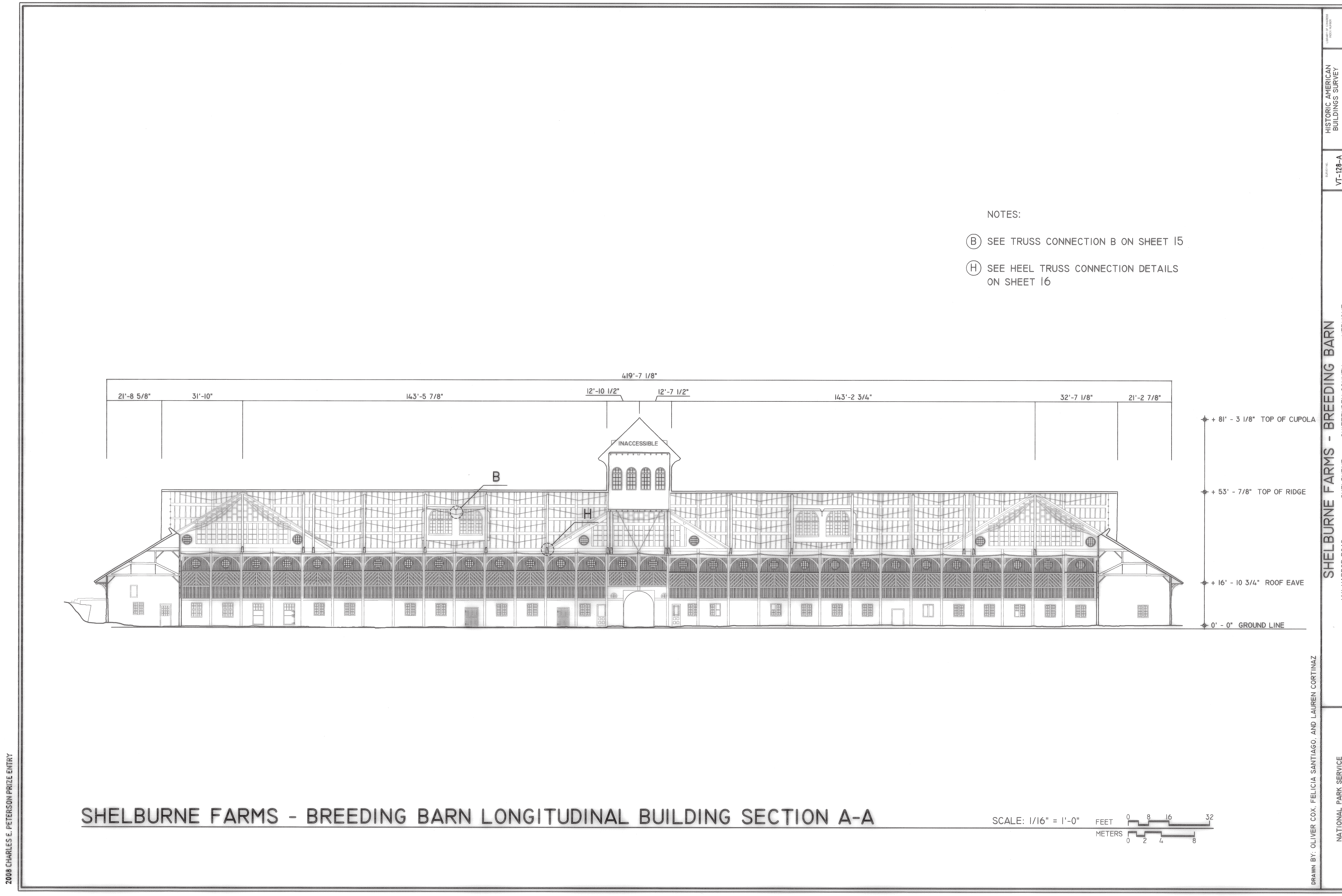
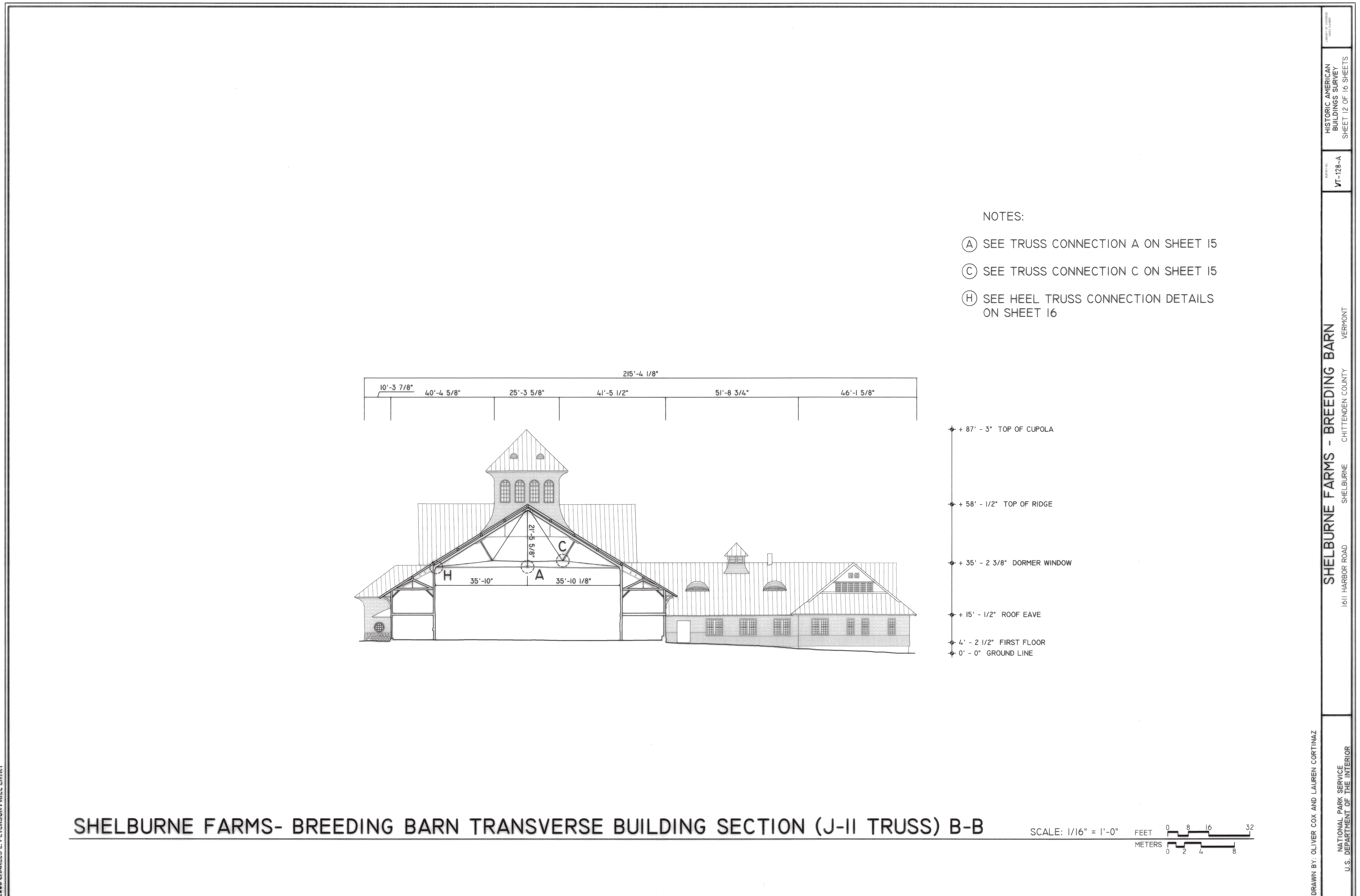
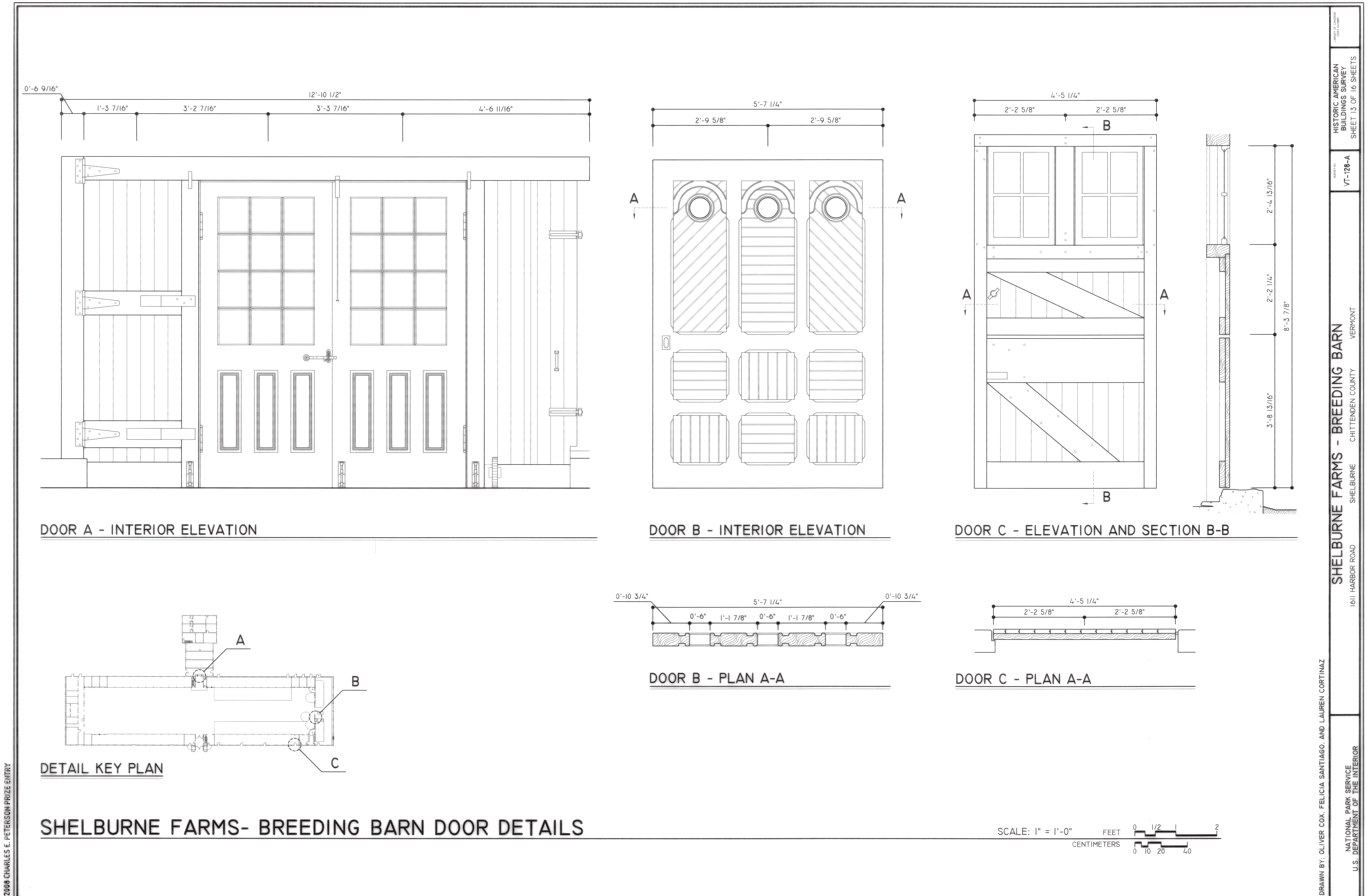
INDEX NUMBER

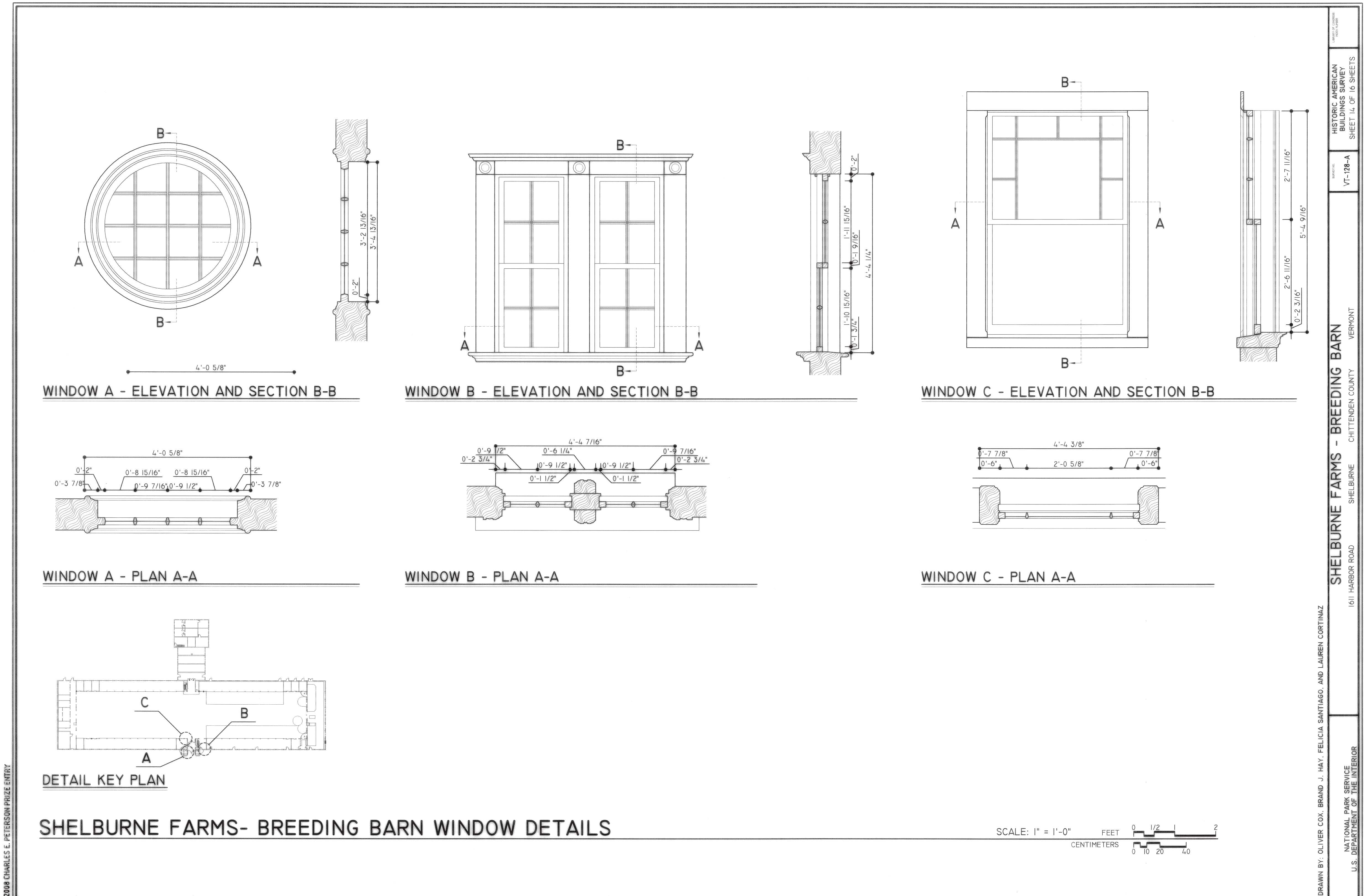

WEST ELEVATION DETAIL
SCALE: 1/8" = 1'-0"
FEET 0 4 8 16
METERS 0 1 2 4
+ 88' - 1/8" TOP OF CUPOLA
+ 58' - 7 1/4" TOP OF RIDGE
+ 35' - 3 7/8" DORMER WINDOW
+ 15' - 1/2" ROOF EAVE
+ 4' - 2 1/2" FIRST FLOOR
0' - 0" GROUND LINE
WEST ELEVATION
SCALE: 1/16" = 1'-0"
FEET 0 8 16 32
METERS 0 2 4 8
SHELBURNE FARMS - BREEDING BARN WEST ELEVATION AND DETAIL
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: BRANDON J. HAY AND LAUREN CORTINAZ
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 10 OF 16 SHEETS

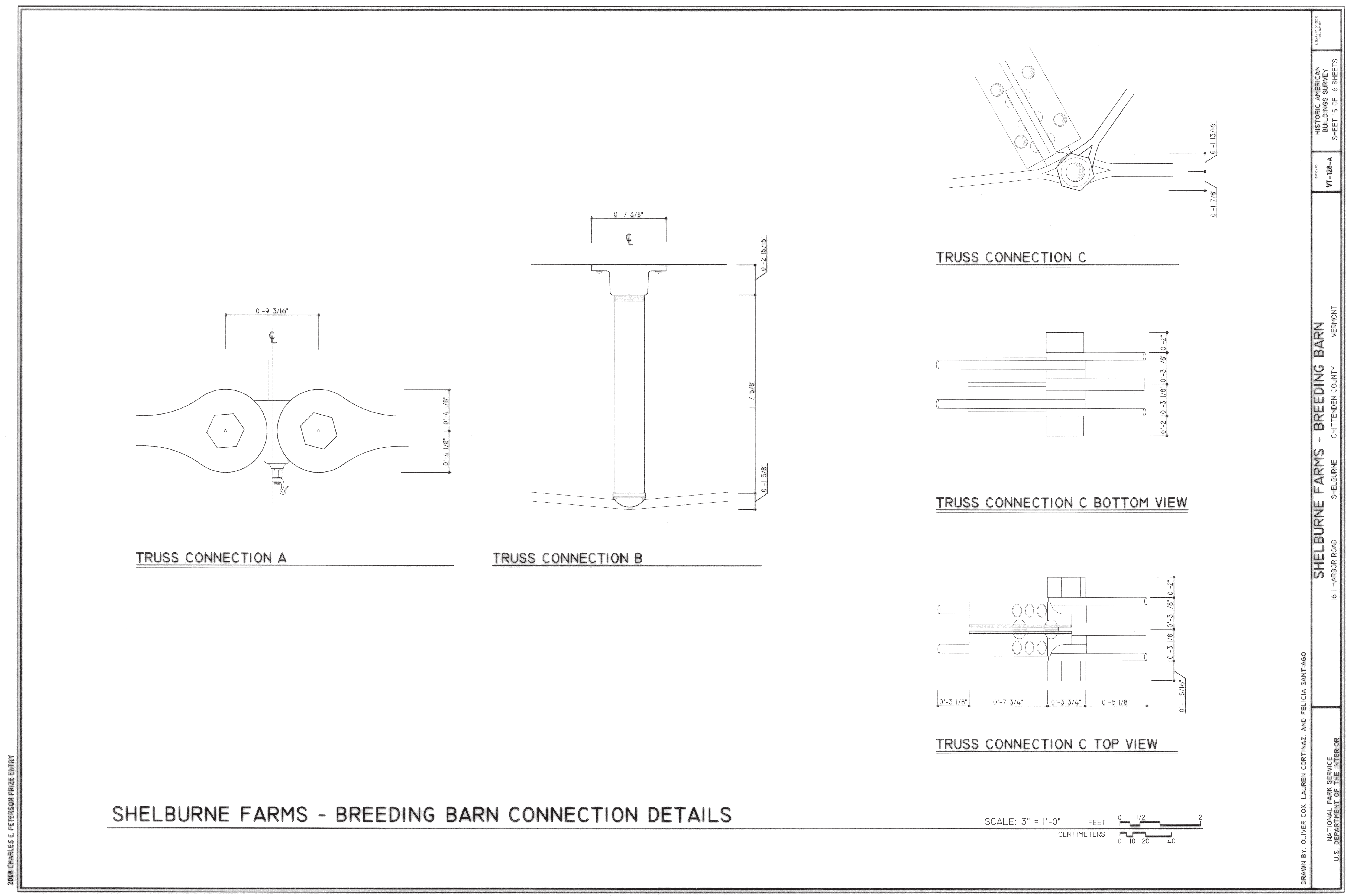

NOTES:
(B) SEE TRUSS CONNECTION B ON SHEET 15
(H) SEE HEEL TRUSS CONNECTION DETAILS ON SHEET 16
419'-7 1/8"
21'-8 5/8"
31'-10"
143'-5 7/8"
12'-10 1/2"
12'-7 1/2"
143'-2 3/4"
32'-7 1/8"
21'-2 7/8"
INACCESSIBLE
B
H
+ 81' - 3 1/8" TOP OF CUPOLA
+ 53' - 7/8" TOP OF RIDGE
+ 16' - 10 3/4" ROOF EAVE
0' - 0" GROUND LINE
SHELBURNE FARMS - BREEDING BARN LONGITUDINAL BUILDING SECTION A-A
SCALE: 1/16" = 1'-0"
FEET 0 8 16 32
METERS 0 2 4 8
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: OLIVER COX, FELICIA SANTIAGO, AND LAUREN CORTINAZ
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 11 OF 16 SHEETS

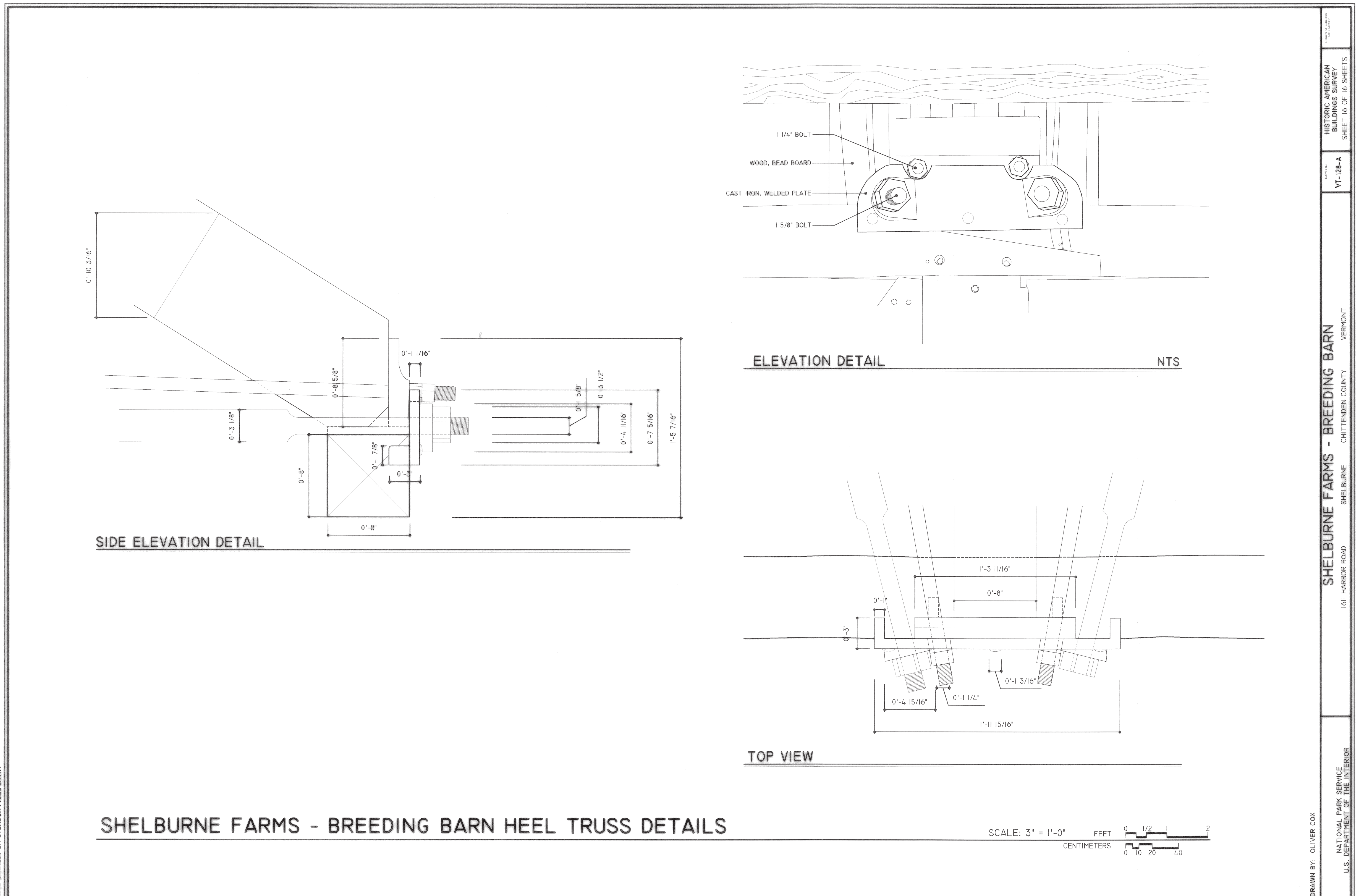


NOTES:
(A) SEE TRUSS CONNECTION A ON SHEET 15
(C) SEE TRUSS CONNECTION C ON SHEET 15
(H) SEE HEEL TRUSS CONNECTION DETAILS ON SHEET 16
215'-4 1/8"
10'-3 7/8"
40'-4 5/8"
25'-3 5/8"
41'-5 1/2"
51'-8 3/4"
46'-1 5/8"
21'-5 5/8"
35'-10"
35'-10 1/8"
+ 87' - 3" TOP OF CUPOLA
+ 58' - 1/2" TOP OF RIDGE
+ 35' - 2 3/8" DORMER WINDOW
+ 15' - 1/2" ROOF EAVE
4' - 2 1/2" FIRST FLOOR
0' - 0" GROUND LINE
SHELBURNE FARMS- BREEDING BARN TRANSVERSE BUILDING SECTION (J-11 TRUSS) B-B
SCALE: 1/16" = 1'-0"
FEET 0 8 16 32
METERS 0 2 4 8
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: OLIVER COX AND LAUREN CORTINAZ
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 12 OF 16 SHEETS


DOOR A - INTERIOR ELEVATION
DOOR B - INTERIOR ELEVATION
DOOR C - ELEVATION AND SECTION B-B
DETAIL KEY PLAN
DOOR B - PLAN A-A
DOOR C - PLAN A-A
SHELBURNE FARMS- BREEDING BARN DOOR DETAILS
SCALE: 1" = 1'-0"
FEET
CENTIMETERS
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: OLIVER COX, FELICIA SANTIAGO, AND LAUREN CORTINAZ
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD SHELBURNE CHITTENDEN COUNTY VERMONT
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 13 OF 16 SHEETS


WINDOW A - ELEVATION AND SECTION B-B
WINDOW B - ELEVATION AND SECTION B-B
WINDOW C - ELEVATION AND SECTION B-B
WINDOW A - PLAN A-A
WINDOW B - PLAN A-A
WINDOW C - PLAN A-A
DETAIL KEY PLAN
SHELBURNE FARMS- BREEDING BARN WINDOW DETAILS
SCALE: 1" = 1'-0"
FEET
CENTIMETERS
DRAWN BY: OLIVER COX, BRAND J. HAY, FELICIA SANTIAGO, AND LAUREN CORTINAZ
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD
SHELBURNE
CHITTENDEN COUNTY
VERMONT
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 14 OF 16 SHEETS
2008 CHARLES E. PETERSON PRIZE ENTRY


TRUSS CONNECTION A
0'-9 3/16"
0'-4 1/8"
0'-4 1/8"
TRUSS CONNECTION B
0'-7 3/8"
0'-2 15/16"
1'-7 5/8"
0'-1 5/8"
TRUSS CONNECTION C
0'-1 13/16"
0'-1 7/8"
TRUSS CONNECTION C BOTTOM VIEW
0'-2"
0'-3 1/8"
0'-3 1/8"
0'-2"
TRUSS CONNECTION C TOP VIEW
0'-3 1/8"
0'-7 3/4"
0'-3 3/4"
0'-6 1/8"
0'-1 15/16"
SHELBURNE FARMS - BREEDING BARN CONNECTION DETAILS
SCALE: 3" = 1'-0"
FEET
CENTIMETERS
2008 CHARLES E. PETERSON PRIZE ENTRY
DRAWN BY: OLIVER COX, LAUREN CORTINAZ, AND FELICIA SANTIAGO
NATIONAL PARK SERVICE
U.S. DEPARTMENT OF THE INTERIOR
SHELBURNE FARMS - BREEDING BARN
1611 HARBOR ROAD
SHELBURNE
CHITTENDEN COUNTY
VERMONT
VT-128-A
HISTORIC AMERICAN BUILDINGS SURVEY
SHEET 15 OF 16 SHEETS



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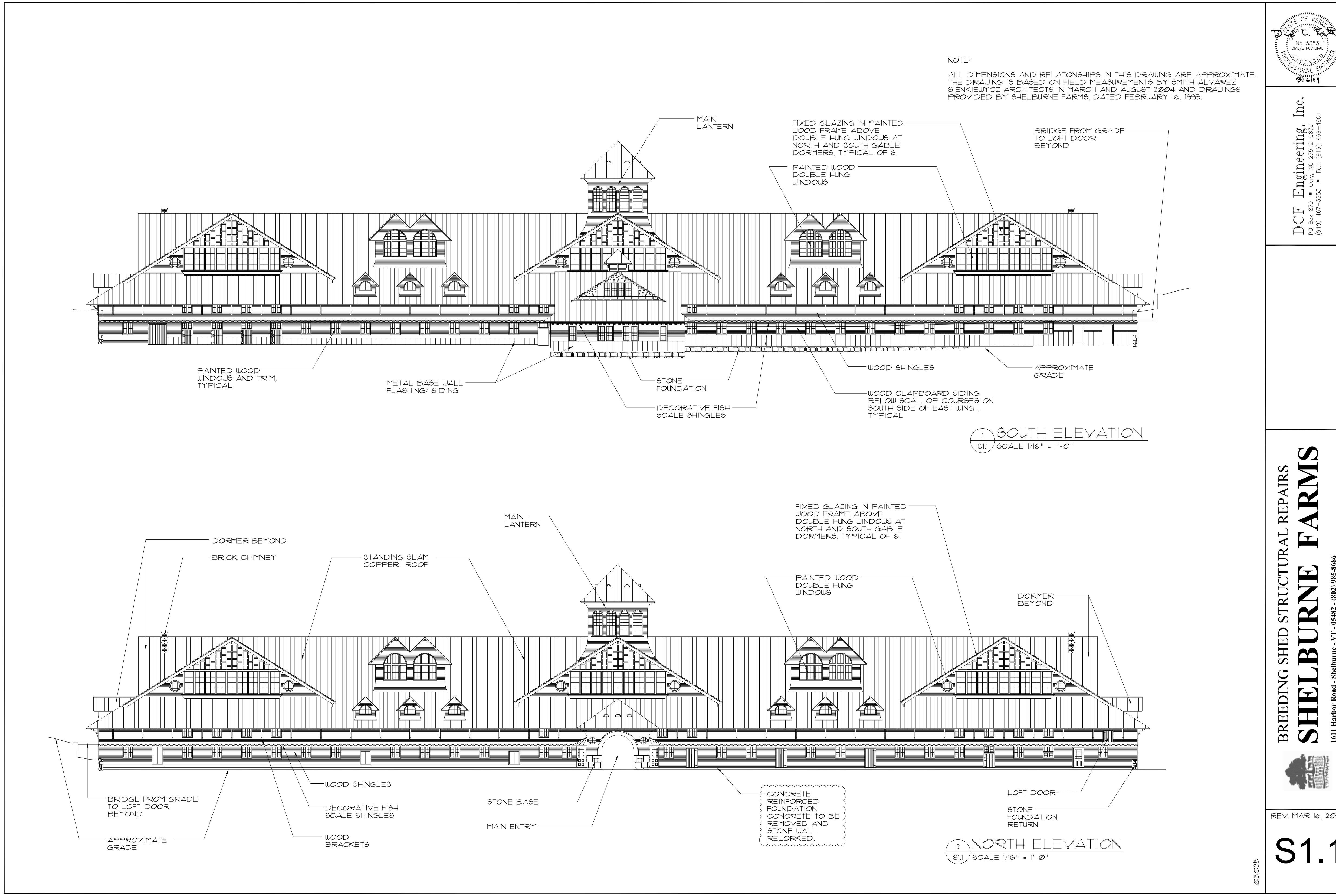
Appendix D: Design Drawings

The following appendix includes the design drawings developed by Project Engineer, David Fischetti (DCF Engineering) to guide structural repair of the Breeding Barn. They are based on three years of detailed examination, building investigation and analysis. Because of the quality of the information yielded by the investigation, it was possible to anticipate the type and extent of repairs needed in detail. This provided the designer with the lead-time necessary to develop repairs that were conservative of original material while meeting public safety requirements, and helped to prevent expensive delays in construction.


NOTE:
ALL DIMENSIONS AND RELATONSHIPS IN THIS DRAWING ARE APPROXIMATE. THE DRAWING IS BASED ON FIELD MEASUREMENTS BY SMITH ALVAREZ SIENKIEWYCZ ARCHITECTS IN MARCH AND AUGUST 2004 AND DRAWINGS PROVIDED BY SHELBURNE FARMS, DATED FEBRUARY 16, 1995.
MAIN LANTERN
FIXED GLAZING IN PAINTED WOOD FRAME ABOVE DOUBLE HUNG WINDOWS AT NORTH AND SOUTH GABLE DORMERS, TYPICAL OF 6.
PAINTED WOOD DOUBLE HUNG WINDOWS
BRIDGE FROM GRADE TO LOFT DOOR BEYOND
PAINTED WOOD WINDOWS AND TRIM, TYPICAL
METAL BASE WALL FLASHING/ SIDING
STONE FOUNDATION
DECORATIVE FISH SCALE SHINGLES
WOOD SHINGLES
APPROXIMATE GRADE
WOOD CLAPBOARD SIDING BELOW SCALLOP COURSES ON SOUTH SIDE OF EAST WING , TYPICAL
1 S1.1 SOUTH ELEVATION
SCALE 1/16" = 1'-0"
MAIN LANTERN
DORMER BEYOND
BRICK CHIMNEY
STANDING SEAM COPPER ROOF
FIXED GLAZING IN PAINTED WOOD FRAME ABOVE DOUBLE HUNG WINDOWS AT NORTH AND SOUTH GABLE DORMERS, TYPICAL OF 6.
PAINTED WOOD DOUBLE HUNG WINDOWS
DORMER BEYOND
WOOD SHINGLES
DECORATIVE FISH SCALE SHINGLES
BRIDGE FROM GRADE TO LOFT DOOR BEYOND
APPROXIMATE GRADE
WOOD BRACKETS
STONE BASE
MAIN ENTRY
CONCRETE REINFORCED FOUNDATION. CONCRETE TO BE REMOVED AND STONE WALL REWORKED.
LOFT DOOR
STONE FOUNDATION RETURN
2 S1.1 NORTH ELEVATION
SCALE 1/16" = 1'-0"
05025
DCF Engineering, Inc.
PO Box 879 • Cary, NC 27512-0879
(919) 467-3853 • Fax: (919) 469-4901
BREEDING SHED STRUCTURAL REPAIRS
SHELBURNE FARMS
1611 Harbor Road - Shelburne - VT - 05482 - (802) 985-8686
REV. MAR 16, 2009
S1.1

NOTE:

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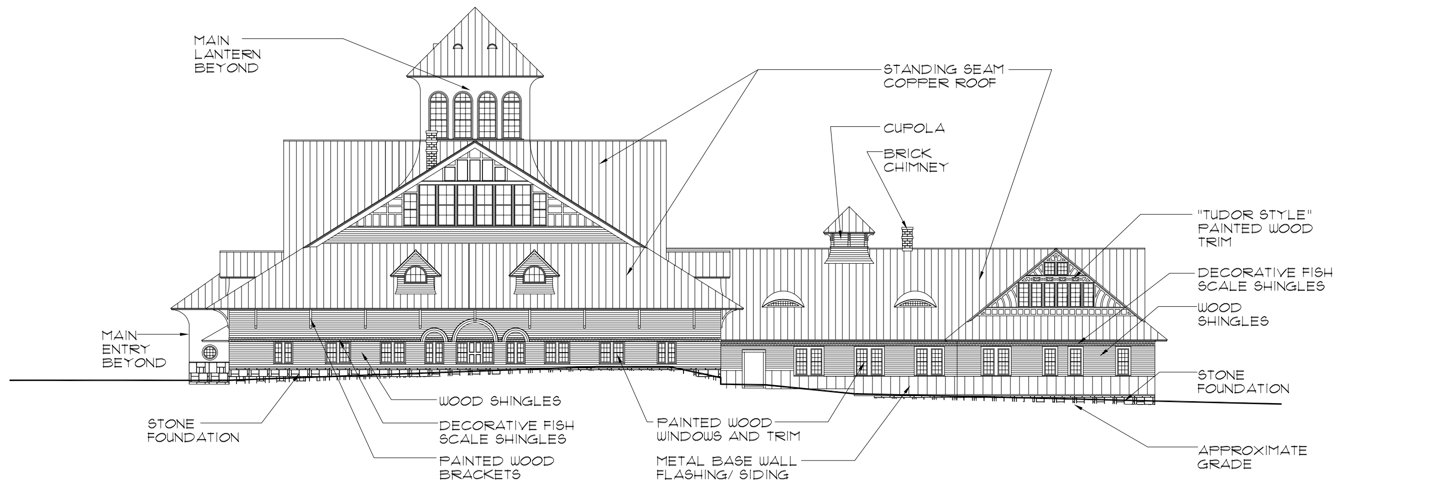
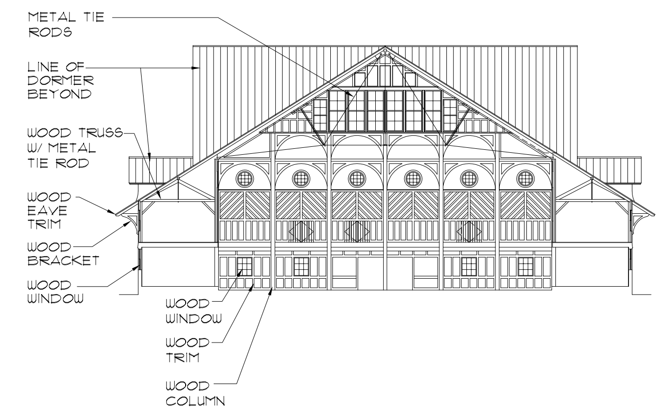
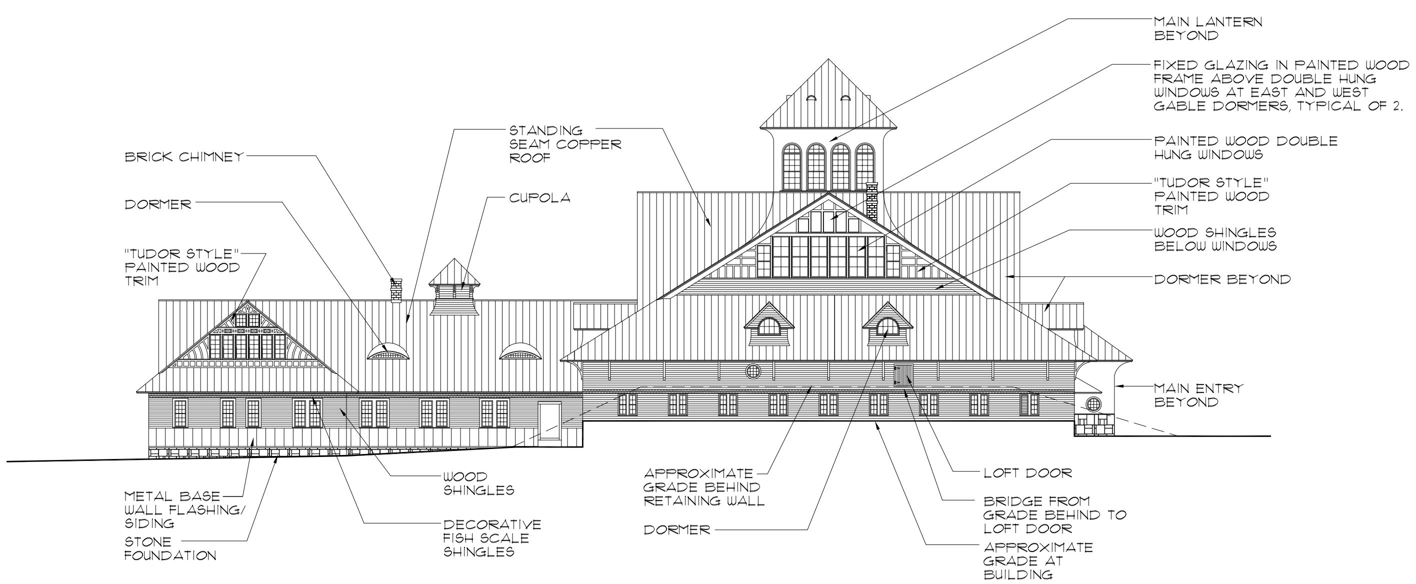


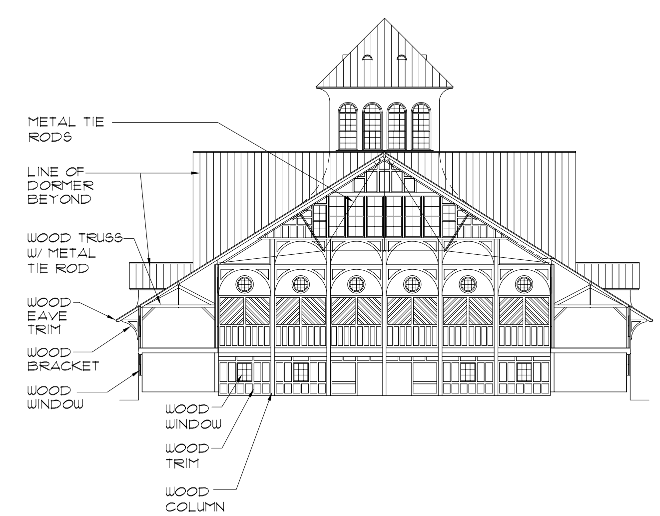
4 S1.2 WEST ELEVATION
SCALE 1/16" = 1'-0"



3 S1.2 CROSS SECTION THROUGH BUILDING AT THE RING LOOKING WEST
SCALE 1/16" = 1'-0"



2 S1.2 EAST ELEVATION
SCALE 1/16" = 1'-0"



1 S1.2 CROSS SECTION THROUGHT BUILDING AT THE RING
SCALE 1/16" = 1'-0"

STATE OF VERMONT DAVID C. FISCHETTI No 5353 CIVIL/STRUCTURAL LICENSED PROFESSIONAL ENGINEER
3/16/09

DCF Engineering, Inc.
PO Box 879 ▪ Cary, NC 27512-0879
(919) 467-3853 ▪ Fax: (919) 469-4901

BREEDING SHED STRUCTURAL REPAIRS

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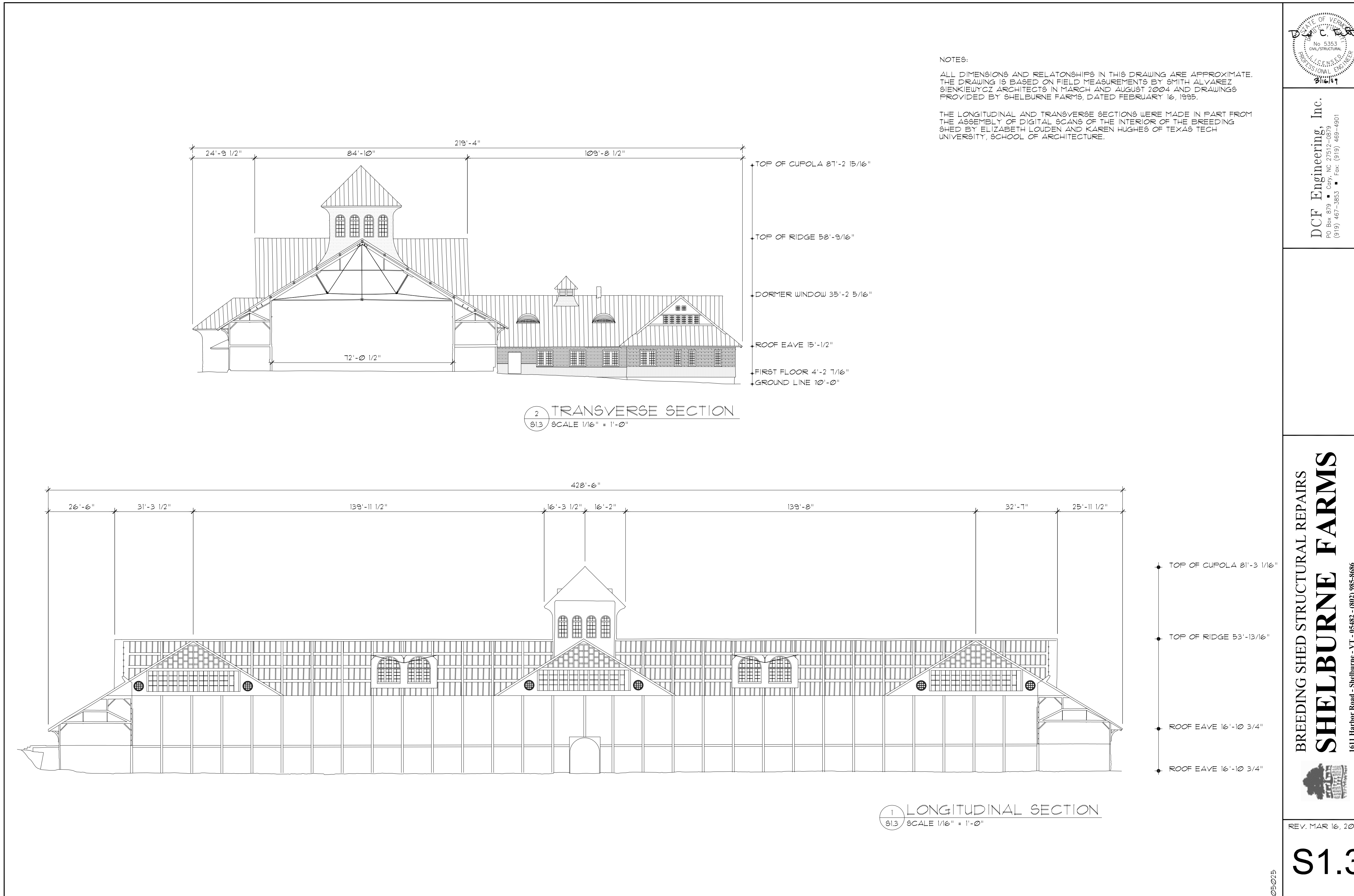
S1.2

05025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ALL DIMENSIONS AND RELATONSHIPS IN THIS DRAWING ARE APPROXIMATE. THE DRAWING IS BASED ON FIELD MEASUREMENTS BY SMITH ALVAREZ SIENKIEWYCZ ARCHITECTS IN MARCH AND AUGUST 2004 AND DRAWINGS PROVIDED BY SHELBURNE FARMS, DATED FEBRUARY 16, 1995.
THE LONGITUDINAL AND TRANSVERSE SECTIONS WERE MADE IN PART FROM THE ASSEMBLY OF DIGITAL SCANS OF THE INTERIOR OF THE BREEDING SHED BY ELIZABETH LOUDEN AND KAREN HUGHES OF TEXAS TECH UNIVERSITY, SCHOOL OF ARCHITECTURE.
219'-4"
24'-9 1/2"
84'-10"
109'-8 1/2"
TOP OF CUPOLA 87'-2 15/16"
TOP OF RIDGE 58'-9/16"
DORMER WINDOW 35'-2 5/16"
ROOF EAVE 15'-1/2"
72'-0 1/2"
FIRST FLOOR 4'-2 7/16"
GROUND LINE 10'-0"
2 S1.3 TRANSVERSE SECTION
SCALE 1/16" = 1'-0"
428'-6"
26'-6"
31'-3 1/2"
139'-11 1/2"
16'-3 1/2"
16'-2"
139'-8"
32'-7"
25'-11 1/2"
TOP OF CUPOLA 81'-3 1/16"
TOP OF RIDGE 53'-13/16"
ROOF EAVE 16'-10 3/4"
ROOF EAVE 16'-10 3/4"
1 S1.3 LONGITUDINAL SECTION
SCALE 1/16" = 1'-0"
05025
STATE OF VERMONT
No 5353
CIVIL/STRUCTURAL
LICENSED
PROFESSIONAL ENGINEER
3/16/09
DCF Engineering, Inc.
PO Box 879 ▪ Cary, NC 27512-0879
(919) 467-3853 ▪ Fax: (919) 469-4901
BREEDING SHED STRUCTURAL REPAIRS
SHELBURNE FARMS
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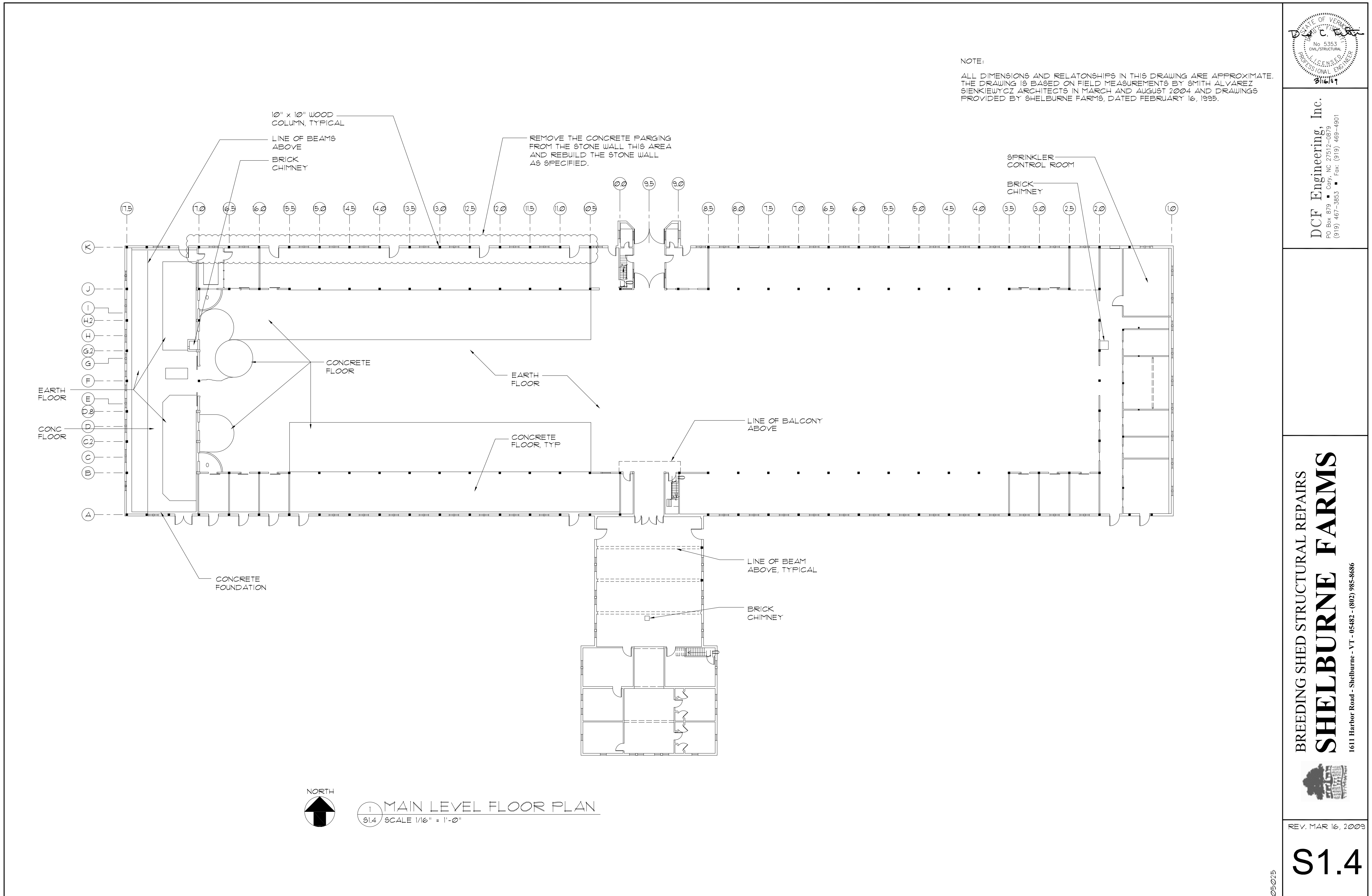
REV. MAR 16, 2009
S1.3



NOTE:
ALL DIMENSIONS AND RELATONSHIPS IN THIS DRAWING ARE APPROXIMATE. THE DRAWING IS BASED ON FIELD MEASUREMENTS BY SMITH ALVAREZ SIENKIEWYCZ ARCHITECTS IN MARCH AND AUGUST 2004 AND DRAWINGS PROVIDED BY SHELBURNE FARMS, DATED FEBRUARY 16, 1995.
10" x 10" WOOD COLUMN, TYPICAL
LINE OF BEAMS ABOVE
BRICK CHIMNEY
REMOVE THE CONCRETE PARGING FROM THE STONE WALL THIS AREA AND REBUILD THE STONE WALL AS SPECIFIED.
SPRINKLER CONTROL ROOM
BRICK CHIMNEY
CONCRETE FLOOR
EARTH FLOOR
EARTH FLOOR
CONC FLOOR
LINE OF BALCONY ABOVE
CONCRETE FLOOR, TYP
CONCRETE FOUNDATION
LINE OF BEAM ABOVE, TYPICAL
BRICK CHIMNEY
NORTH
MAIN LEVEL FLOOR PLAN
SCALE 1/16" = 1'-0"
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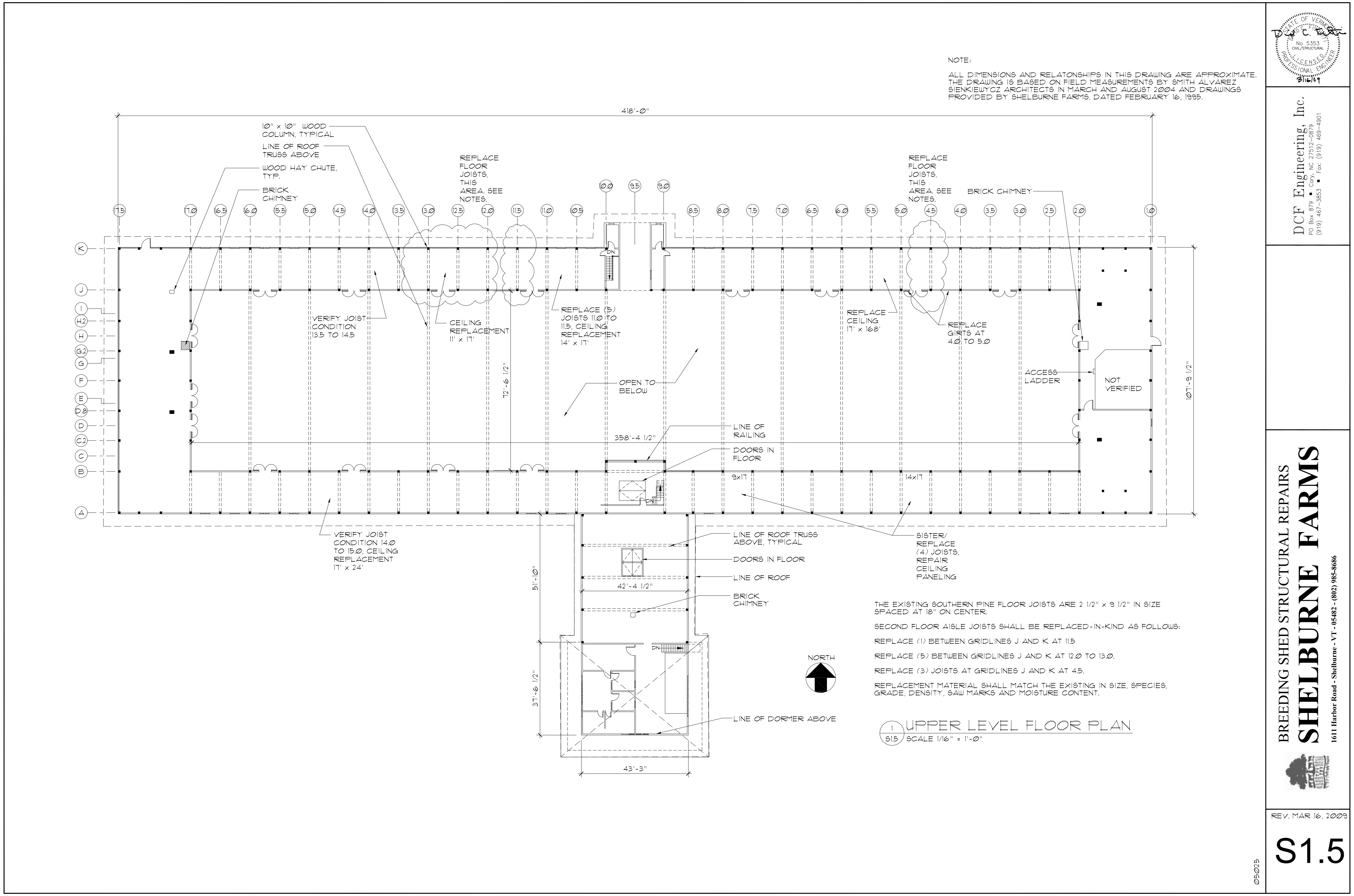
NOTE:
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418'-0"
10" x 10" WOOD COLUMN, TYPICAL
LINE OF ROOF TRUSS ABOVE
WOOD HAY CHUTE, TYP.
BRICK CHIMNEY
REPLACE FLOOR JOISTS, THIS AREA, SEE NOTES.
BRICK CHIMNEY
VERIFY JOIST CONDITION 13.5 TO 14.5
CEILING REPLACEMENT 11' x 17'
REPLACE (5) JOISTS 11.0 TO 11.5, CEILING REPLACEMENT 14' x 17'
REPLACE CEILING 17' x 16.8'
REPLACE GIRTS AT 4.0 TO 5.0
ACCESS LADDER
NOT VERIFIED
OPEN TO BELOW
72'-6 1/2"
107'-9 1/2"
358'-4 1/2"
LINE OF RAILING
DOORS IN FLOOR
9x17
14x17
VERIFY JOIST CONDITION 14.0 TO 15.0, CEILING REPLACEMENT 17' x 24'
LINE OF ROOF TRUSS ABOVE, TYPICAL
DOORS IN FLOOR
LINE OF ROOF
42'-4 1/2"
BRICK CHIMNEY
SISTER/ REPLACE (4) JOISTS, REPAIR CEILING PANELING
51'-10"
37'-6 1/2"
LINE OF DORMER ABOVE
43'-3"
NORTH
THE EXISTING SOUTHERN PINE FLOOR JOISTS ARE 2 1/2" x 9 1/2" IN SIZE SPACED AT 18" ON CENTER.
SECOND FLOOR AISLE JOISTS SHALL BE REPLACED-IN-KIND AS FOLLOWS:
REPLACE (1) BETWEEN GRIDLINES J AND K AT 11.5
REPLACE (5) BETWEEN GRIDLINES J AND K AT 12.0 TO 13.0.
REPLACE (3) JOISTS AT GRIDLINES J AND K AT 4.5.
REPLACEMENT MATERIAL SHALL MATCH THE EXISTING IN SIZE, SPECIES, GRADE, DENSITY, SAW MARKS AND MOISTURE CONTENT.
1 S1.5 UPPER LEVEL FLOOR PLAN
SCALE 1/16" = 1'-0"
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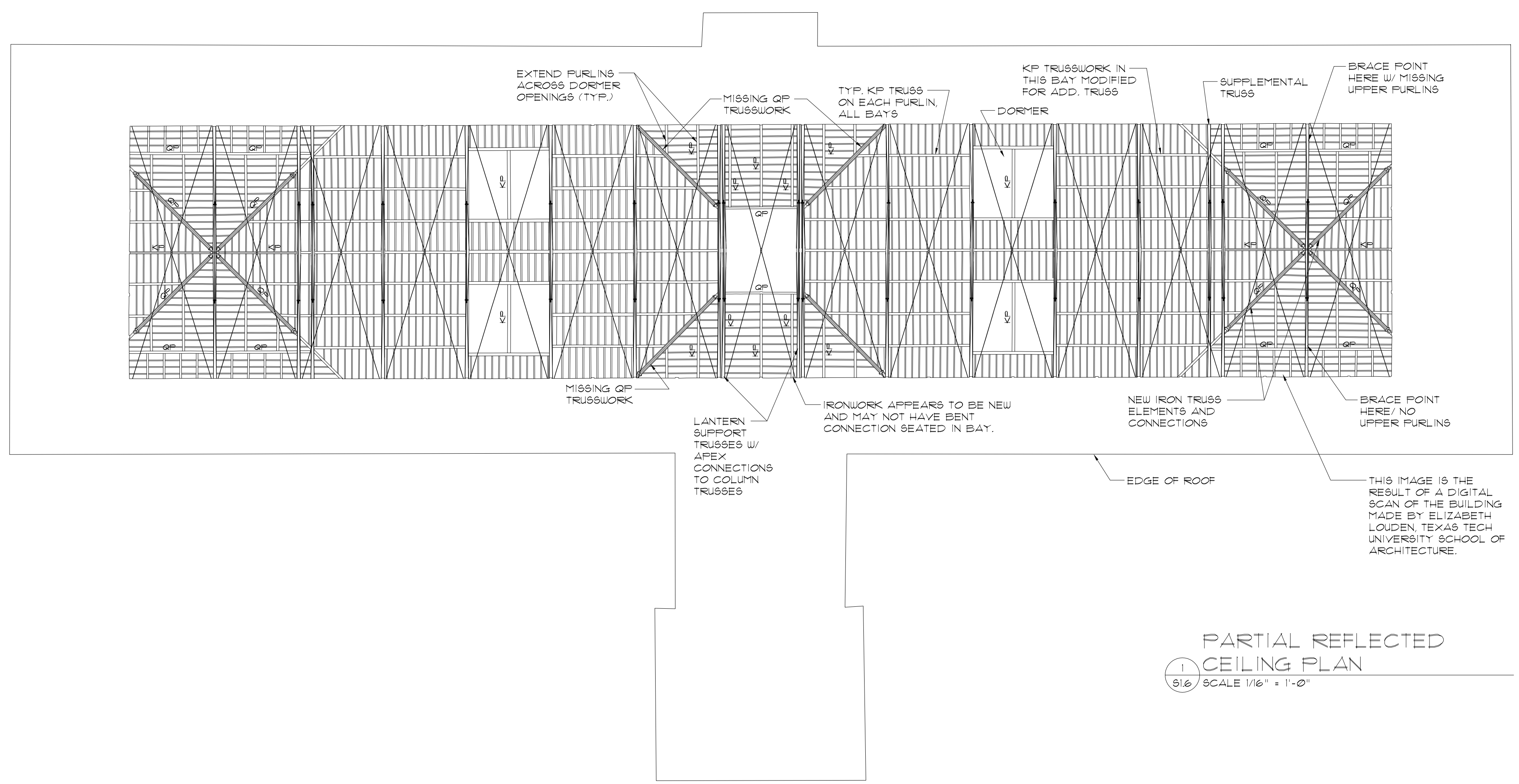
REV. MAR 16, 2009
S1.5
05025

NOTE:

THIS PARTIAL REFLECTED CEILING PLAN WAS MADE FROM THE ASSEMBLY OF DIGITAL SCANS OF THE INTERIOR OF THE BREEDING SHED BY ELIZABETH LOUDEN AND KAREN HUGHES OF TEXAS TECH UNIVERSITY, SCHOOL OF ARCHITECTURE.



PARTIAL REFLECTED CEILING PLAN

1 S1.6 SCALE 1/16" = 1'-0"

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S1.6

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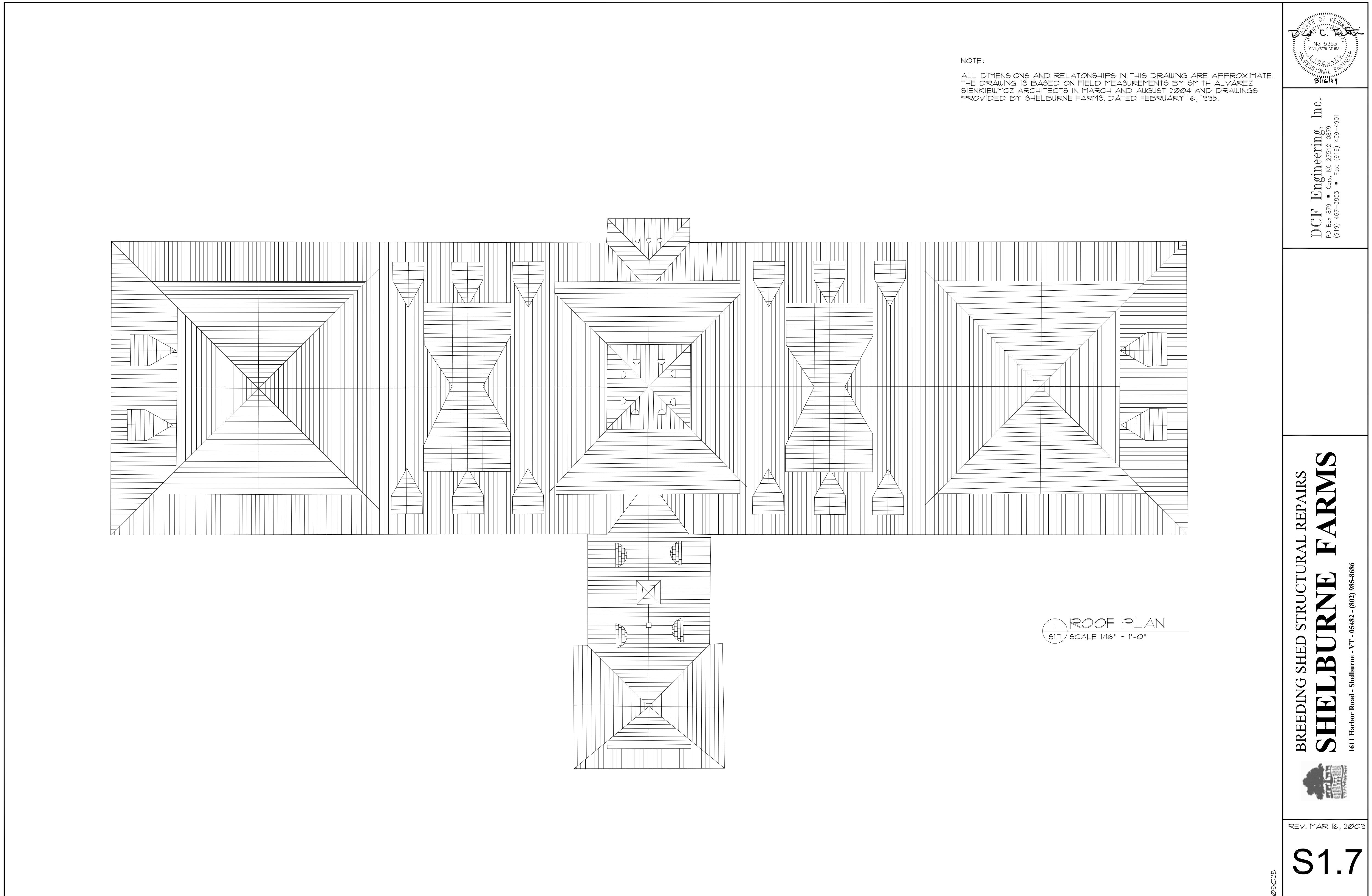
NOTE:
ALL DIMENSIONS AND RELATONSHIPS IN THIS DRAWING ARE APPROXIMATE. THE DRAWING IS BASED ON FIELD MEASUREMENTS BY SMITH ALVAREZ SIENKIEWYCZ ARCHITECTS IN MARCH AND AUGUST 2004 AND DRAWINGS PROVIDED BY SHELBURNE FARMS, DATED FEBRUARY 16, 1995.
ROOF PLAN
SCALE 1/16" = 1'-0"
1
S1.7
DCF Engineering, Inc.
BREEDING SHED STRUCTURAL REPAIRS
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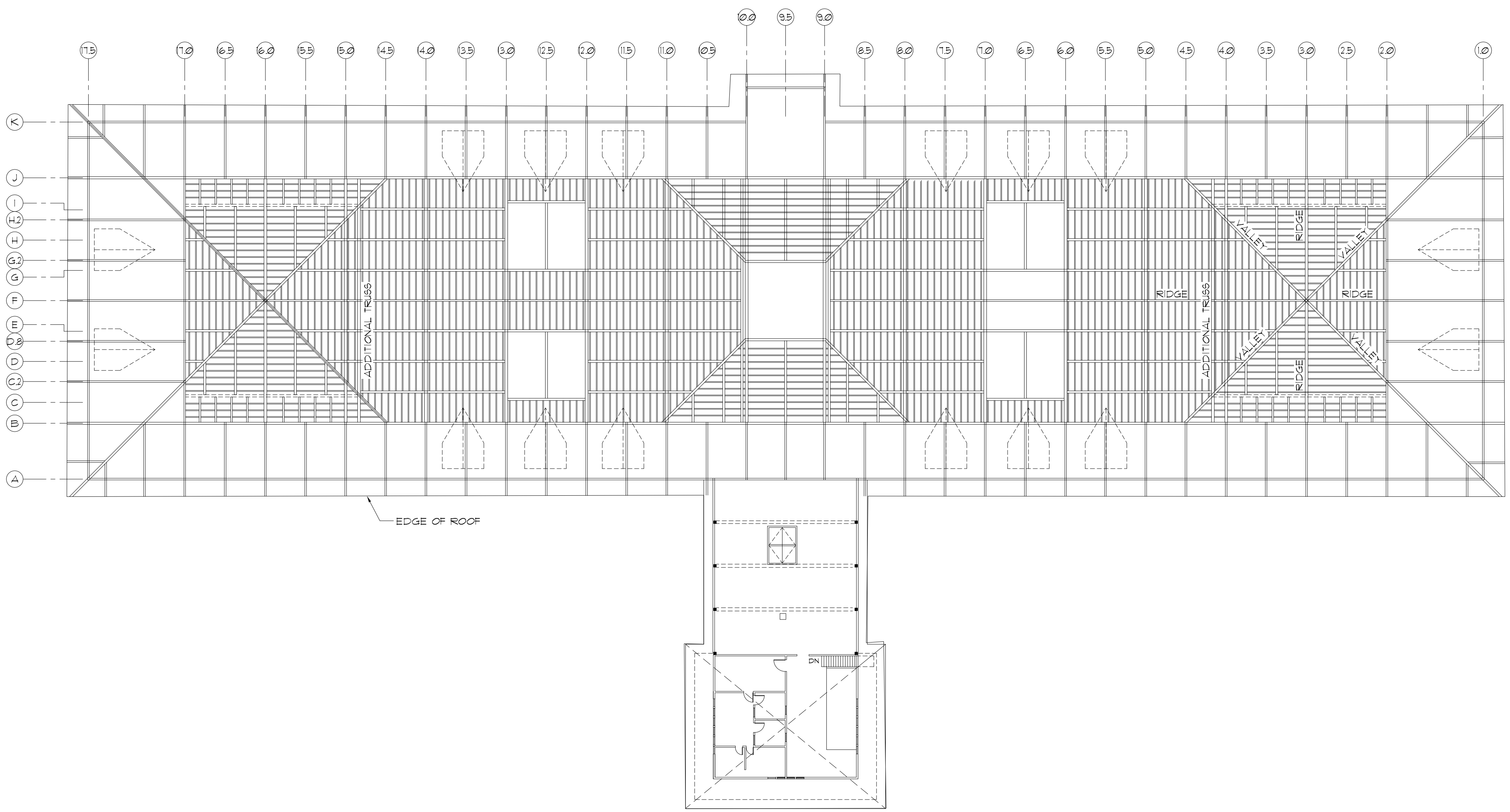
REV. MAR 16, 2008
S1.7
05025

NOTE:
ALL DIMENSIONS AND RELATONSHIPS IN THIS DRAWING ARE APPROXIMATE. THE DRAWING IS BASED ON FIELD MEASUREMENTS BY SMITH ALVAREZ SIENKIEWYCZ ARCHITECTS IN MARCH AND AUGUST 2004 AND DRAWINGS PROVIDED BY SHELBURNE FARMS, DATED FEBRUARY 16, 1995.



1 / S1.8 ROOF FRAMING PLAN
SCALE 1/16" = 1'-0"

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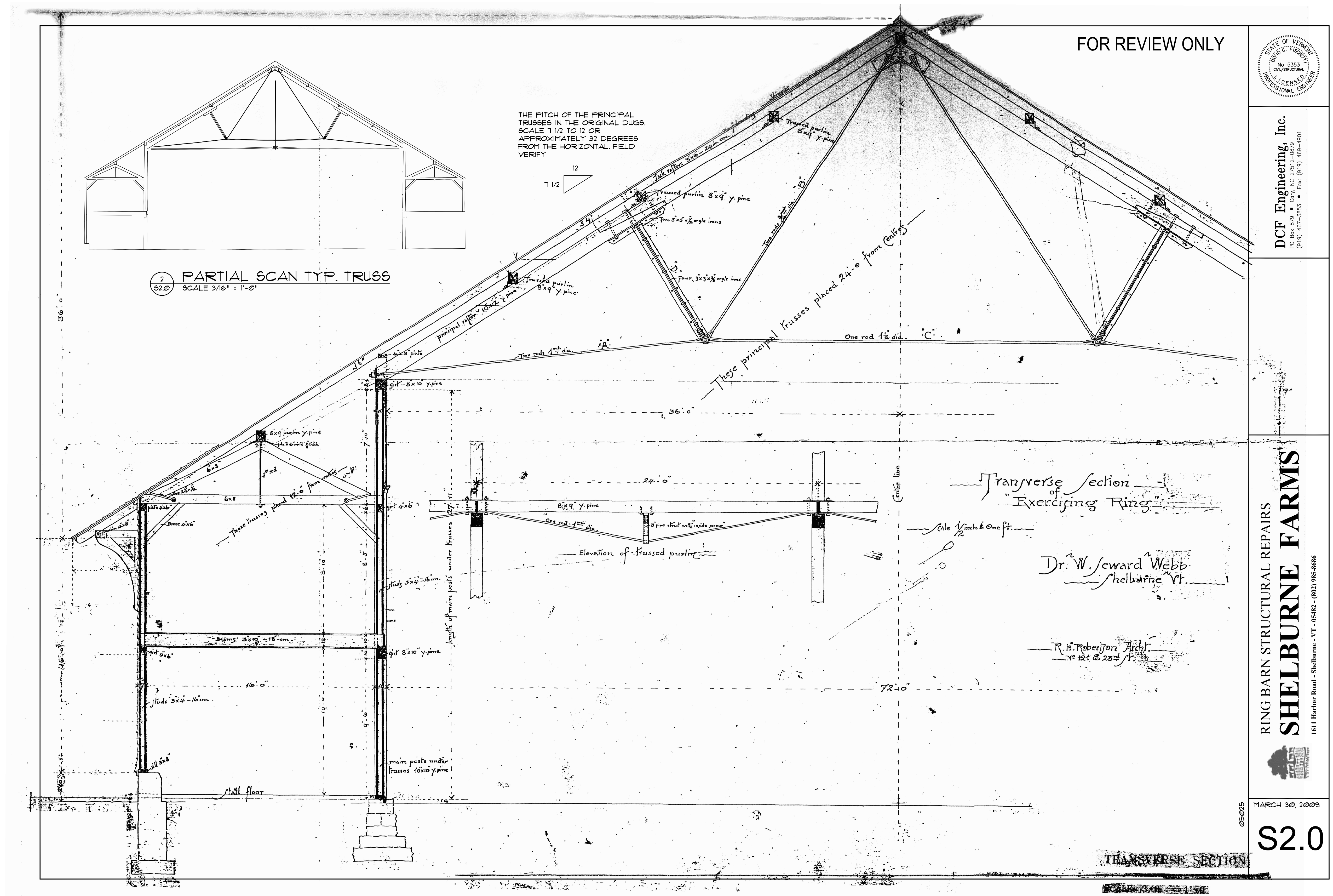

FOR REVIEW ONLY
THE PITCH OF THE PRINCIPAL TRUSSES IN THE ORIGINAL DWGS. SCALE 7 1/2 TO 12 OR APPROXIMATELY 32 DEGREES FROM THE HORIZONTAL. FIELD VERIFY
PARTIAL SCAN TYP. TRUSS
SCALE 3/16" = 1'-0"
Transverse Section of "Exercising Ring"
Scale 1/2 inch = One ft.
Dr. W. Seward Webb
Shelburne Vt.
R. H. Robertson Archt.
These principal trusses placed 24'-0 from centres
Elevation of trussed purlin
stall floor
main posts under trusses 10x10 y. pine
DCF Engineering, Inc.
PO Box 879 • Cary, NC 27512-0879
(919) 467-3853 • Fax: (919) 469-4901
RING BARN STRUCTURAL REPAIRS
SHELBURNE FARMS
1611 Harbor Road - Shelburne - VT - 05482 - (802) 985-8686
MARCH 30, 2009
S2.0
TRANSVERSE SECTION

GENERAL STRUCTURAL NOTES

FOR REVIEW ONLY

A. LIVE LOADS

1. ROOF LIVE LOAD 30 PSF
2. WIND LOAD (BASIC) 90 MPH
3. SECOND LEVEL 50 PSF
4. STAIRS, EXITS 100 PSF

2005 VERMONT FIRE & BUILDING SAFETY CODE
2003 EDITION INTERNATIONAL BUILDING CODE
SEISMIC HAZARD EXPOSURE GROUP I
STRUCTURAL SYSTEM: TIMBER FRAME

B. FOUNDATIONS

1. THE SOIL BEARING PRESSURE ASSUMED FOR DESIGN IS 2000 PSF.
2. ALL FILL SHALL BE PLACED IN AN 8 INCH MAXIMUM LOOSE LIFTS AND SHALL BE COMPACTED TO A MINIMUM OF 98 PER CENT MAXIMUM DRY DENSITY AS DETERMINED IN ACCORDANCE WITH ASTM D-698 (STANDARD PROCTOR METHOD).

C. CAST-IN-PLACE CONCRETE

1. CONCRETE WORK SHALL CONFORM TO ACI SPECIFICATIONS.
2. ALL CAST-IN-PLACE CONCRETE 28-DAY COMPRESSIVE STRENGTH SHALL BE 3000 PSI IN ACCORDANCE WITH ACI 318.
3. THE APPLICABLE STANDARD FOR FLOOR SLAB CONSTRUCTION SHALL BE ACI 302.1R "GUIDE FOR CONCRETE FLOOR AND SLAB CONSTRUCTION".
4. PROVIDE CONTROL AND ISOLATION JOINTS IN SLABS AT LOCATIONS AS SHOWN ON THE DRAWINGS.

D. REINFORCING STEEL

1. ALL REINFORCING STEEL SHALL BE ASTM A-615, GRADE 60.
2. PLACEMENT OF THE REINFORCING STEEL SHALL BE REVIEWED BY THE STRUCTURAL ENGINEER PRIOR TO PLACING CONCRETE.
3. DETAIL AND FABRICATE REINFORCING STEEL IN ACCORDANCE WITH ACI-315. REINFORCING STEEL SHALL BE PLACED IN ACCORDANCE WITH THE PROJECT DOCUMENTS.
4. FABRICATE IN ACCORDANCE WITH APPROVED SHOP DRAWINGS.
5. DO NOT HEAT BEND REINFORCING BARS.

E. MASONRY

1. STONE SHALL BE SPECIAL OVERSIZED (NON-STANDARD MODULAR) STONE MATCHING THE ORIGINAL IN SIZE, COLOR, SHAPE, FINISH.
2. MORTAR FOR UNIT MASONRY SHALL BE A SPECIAL LIME BASED MIX.

F. MISCELLANEOUS STEEL

1. APPLICABLE STANDARDS:
   A. "SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS" OF THE AMERICAN INSTITUTE OF STEEL CONSTRUCTION.
   B. "CODE OF STANDARD PRACTICE" OF THE AMERICAN INSTITUTE OF STEEL CONSTRUCTION.
   C. "APPLICATION OF THE EXTRACTS FROM CODE FOR ARC AND GAS WELDING IN BUILDING CONSTRUCTION" BY THE AMERICAN WELDING SOCIETY.
2. STEEL FOR PLATES AND OTHER MEMBERS SHALL CONFORM TO ASTM A-36.
3. WELDING PROCESSES, TECHNIQUES, AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH AWS STRUCTURAL WELDING CODE D1.1. WHERE EXCEPTIONS TAKEN IN THE AISC SPECIFICATION TO THE AWS CODE, THE AISC SPECIFICATION SHALL GOVERN.
4. FIELD WELDING SHALL NOT BE ALLOWED INSIDE THE BUILDING.
5. WELDING IN SHOP SHALL BE EXECUTED BY OPERATORS WHO HAVE BEEN QUALIFIED PREVIOUSLY BY TESTS IN ACCORDANCE WITH THE AMERICAN WELDING SOCIETY "STANDARD QUALIFICATION PROCEDURE" TO PERFORM THE TYPE OF WORK REQUIRED.
6. SHOP PAINT SHALL BE A SUPERIOR QUALITY ZINC CHROMATE-IRON OXIDE RED PRIMER OR SHOP STANDARD.
7. HOLES IN BASE AND TEMPLATE PLATES SHALL BE DRILLED OR PUNCHED, BURNING OF HOLES SHALL NOT BE PERMITTED.
8. ERECTION SHALL FOLLOW AISC SPECIFICATION "THE DESIGN, FABRICATION, AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS."
9. ANCHOR BOLTS MUST BE SET IN ACCORDANCE WITH THE "APPROVED" OR "APPROVED AS NOTED" ANCHOR BOLT SETTING PLAN PROVIDED BY THE STEEL FABRICATOR AS PART OF THE MISCELLANEOUS STEEL SHOP DRAWINGS.

G. TIMBER REPAIR

1. APPLY A 2 PER CENT METALLIC CONTENT OIL SOLUTION OF ZINC NAPTHENATE TO THE CUT WOOD SURFACES BY BRUSHING TWO COATS.
2. A PLYWOOD TEMPLATE SHALL BE USED TO CHECK THAT THE DIMENSIONS FOR THE REPAIR OF TIMBER MEMBERS, SHOWN ON THE DRAWINGS, WILL FIT THE CONDITIONS IN THE FIELD AT EVERY LOCATION.
3. DRILLING FOR BOLTS IN THE FIELD MUST BE VERY ACCURATE. ALL BOLT HOLES IN STEEL AND TIMBER SHALL BE 1/16 INCH LARGER THAN THE BOLT. FIELD DRILLING OF HOLES IN THE TIMBER MUST BE SQUARE TO THE FACE OF THE MEMBER. A MAGNETIC DRILL OR OTHER RELIABLE METHOD SHALL BE EMPLOYED. A TIGHT FIT REQUIRING FORCIBLE DRIVING OF BOLTS IS NOT RECOMMENDED. REAMING OF HOLES WILL NOT BE ALLOWED.
4. BOLTS SHALL BE ASTM STANDARD SQUARE HEADED A-307, HOT DIPPED GALVANIZED AND PAINTED.
5. MATERIALS AND EXECUTION SHALL FOLLOW THE REQUIREMENTS OF THE 2001 EDITION OF THE NATIONAL DESIGN SPECIFICATION FOR WOOD CONSTRUCTION, PUBLISHED BY THE AMERICAN FOREST & PAPER ASSOCIATION.
6. ALL BOLTS SHALL HAVE WASHERS AT TIMBER BEARING LOCATIONS. WASHERS SHALL BE ROUND HOT DIPPED GALVANIZED CAST IRON OGEE WASHERS PAINTED BLACK.
7. ALL LAG SCREWS, BOLTS, NUTS, NAILS AND WASHERS SHALL BE HOT DIPPED GALVANIZED. BOLTS, WASHERS AND LAG SCREWS SHALL BE PAINTED BLACK.
8. TIMBER REPLACEMENT MATERIALS IN CONTACT WITH MASONRY SHALL BE TREATED BY THE PENTA-WR PROCESS IN ACCORDANCE WITH AWPA STANDARDS P8, P9 (TYPE C) AND C1 TO A RETENTION OF 0.40 PCF, UNLESS THE WOOD IS A NATURALLY DECAY RESISTANT SPECIES.

H. HEAVY TIMBER CONSTRUCTION

1. SOUTHERN PINE REPLACEMENT OR REINFORCING MEMBERS SHALL BE DENSE SELECT STRUCTURAL, GRADED IN ACCORDANCE WITH SPIB GRADING RULES.
2. LUMBER SHALL BE FULL DIMENSION, ROUGH SAWN, CONFORMING TO NO. 1 SOUTHERN PINE GRADED IN ACCORDANCE TO SPIB GRADING RULES.
3. FABRICATION AND CONNECTIONS SHALL CONFORM TO THE AITC TIMBER CONSTRUCTION MANUAL AND THE 2001 EDITION OF THE NATIONAL DESIGN SPECIFICATION.

J. MISCELLANEOUS

1. THE STRUCTURAL DRAWINGS SHALL BE USED IN CONJUNCTION WITH AND COORDINATED WITH THE ARCHITECTURAL DRAWINGS AND OTHER CONTRACT DOCUMENTS.
2. THE CONTRACTOR SHALL VERIFY IN THE FIELD ALL CONDITIONS AT THE SITE INCLUDING DIMENSIONS AND ELEVATIONS WHICH MAY AFFECT THE FABRICATION OF MISCELLANEOUS STEEL OR THE INSTALLATION OF THE FABRICATED ITEMS AS DETAILED.
3. FABRICATOR'S SHOP DRAWINGS SHALL SHOW AND NOTE ALL MATERIAL REQUIRED IN SUFFICIENT DETAIL FOR PROPER FABRICATION AND ERECTION IN ACCORDANCE WITH THE CONTRACT DRAWINGS AND DOCUMENTS.
4. ANCHOR BOLTS SHALL BE SET IN ACCORDANCE WITH THE APPROVED SHOP DRAWING ANCHOR BOLT SETTING PLAN FOR MISCELLANEOUS STEEL MATERIALS.

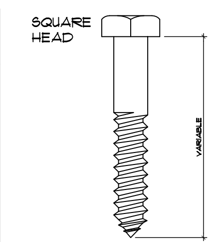
NOTES:

1. FIELD VERIFY DIMENSIONS, SIZES, QUANTITIES
2. TIMBER SIZES ARE APPROXIMATE. MATCH EXISTING.
3. MEMBERS WHICH ARE TO REPLACED IN KIND MAY BE OBTAINED FROM THE SOUND PORTIONS OF SALVAGED PIECES UPON THE PRIOR APPROVAL OF THE ENGINEER. IT IS THE CONTRACTOR'S RESPONSIBILITY TO OBTAIN SUFFICIENT REPLACEMENT MATERIAL TO COMPLETE THE WORK WITHIN THE PROJECT SCHEDULE.
4. REPLACEMENT TIMBER MATERIAL SHALL MATCH THE EXISTING IN SIZE.
5. THE CONTRACTOR SHALL VERIFY SIZES AND CONDITIONS PRIOR TO ORDERING MATERIAL.

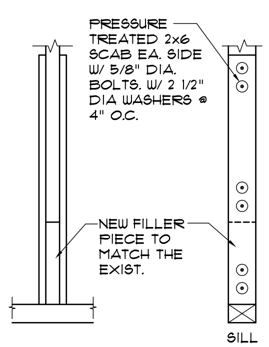
OGEE WASHER		
BOLT SIZE	OVERALL DIAMETER	THICKNESS AT CENTER
1/2"	2"	1/2"
5/8"	2 3/4"	5/8"
3/4"	3"	3/4"

1 / S3.0 OGEE WASHER
NOT TO SCALE

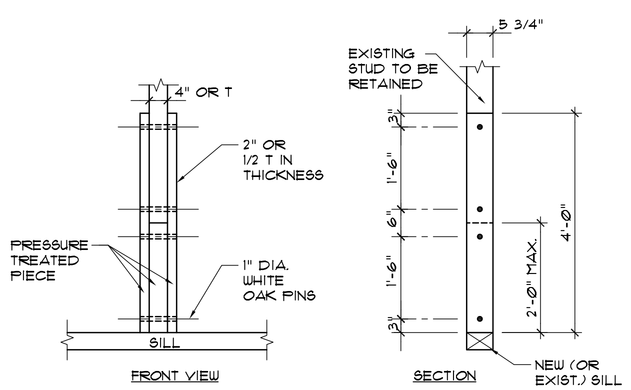
LAG SCREW LEAD HOLES		
NOMINAL DIAMETER OF LAG BOLT	DIAMETER OF LEAD HOLE. IN.	
	SHANK (UNTHREADED) PORTION, IN.	THREADED PORTION GROUP II SPECIES
1/2"	1/2"	5/16"
5/8"	5/8"	13/32"
3/4"	3/4"	1/2"



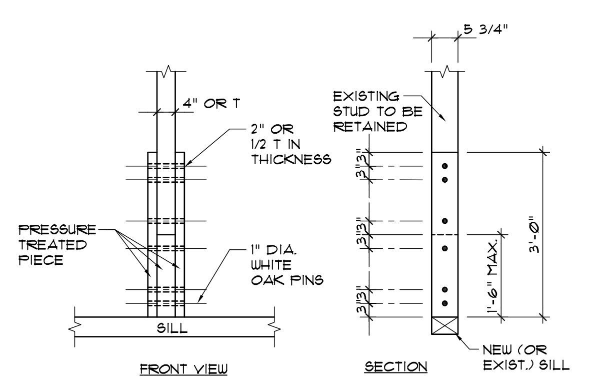
2 / S3.0 LAG SCREW
NOT TO SCALE



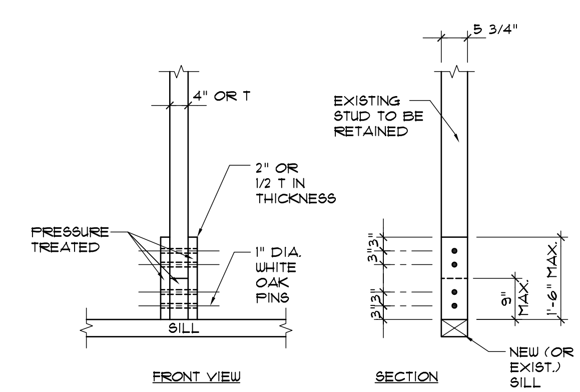
4 / S3.0 STUD SCAB DET.
SCALE 3/4"=1'-0"



3 / S3.0 STUD REPAIR "C"
SCALE 3/4"=1'-0"



2 / S3.0 STUD REPAIR "B"
SCALE 3/4"=1'-0"



1 / S3.0 STUD REPAIR "A"
SCALE 3/4"=1'-0"

05025

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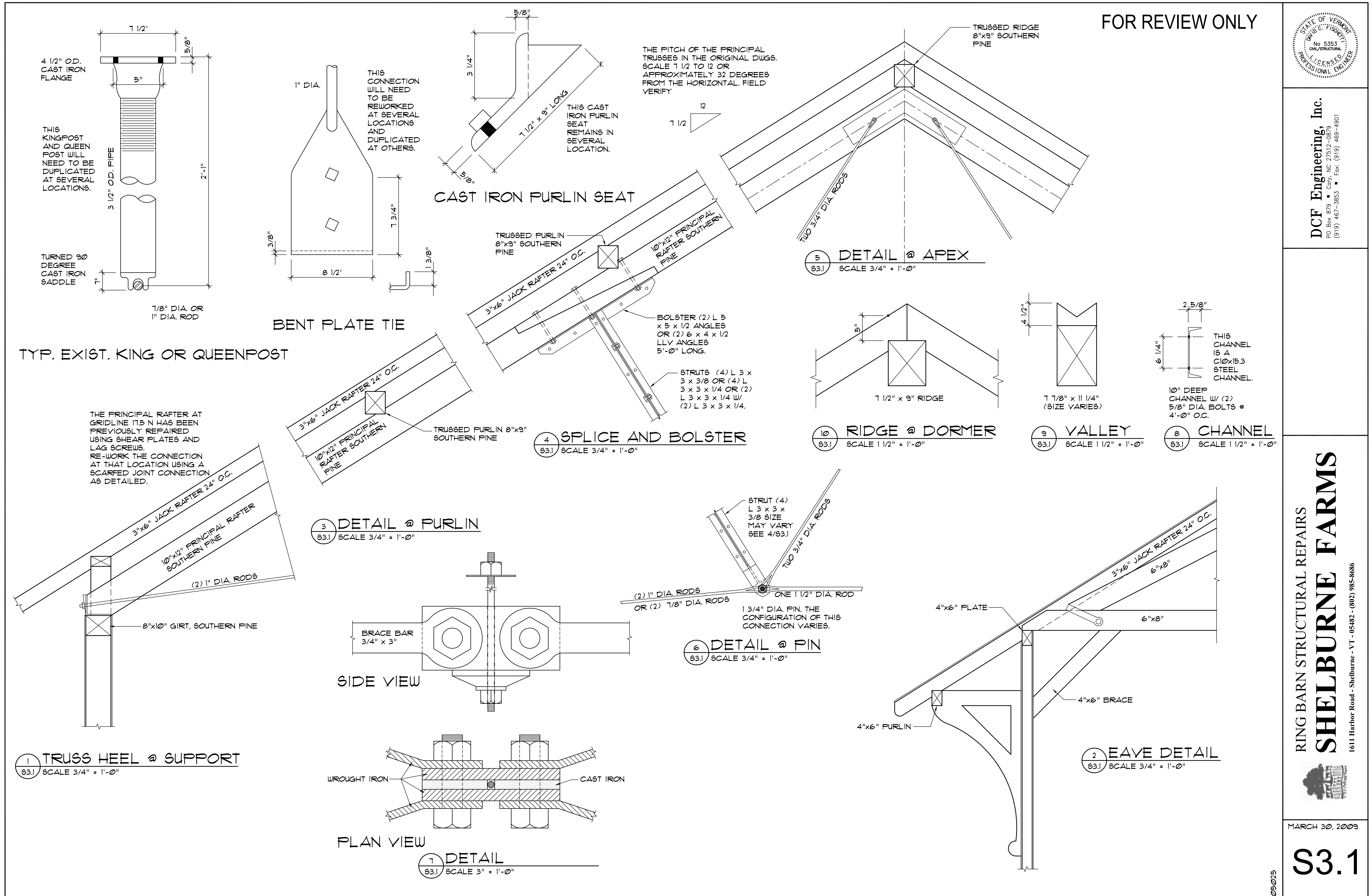
MARCH 30, 2009

S3.0


FOR REVIEW ONLY
7 1/2'
5/8"
4 1/2" O.D. CAST IRON FLANGE
5"
THIS KINGPOST AND QUEEN POST WILL NEED TO BE DUPLICATED AT SEVERAL LOCATIONS.
3 1/2" O.D. PIPE
2'-1"
TURNED 90 DEGREE CAST IRON SADDLE
7/8" DIA. OR 1" DIA. ROD
TYP. EXIST. KING OR QUEENPOST
1" DIA.
THIS CONNECTION WILL NEED TO BE REWORKED AT SEVERAL LOCATIONS AND DUPLICATED AT OTHERS.
7 3/4"
3/8"
8 1/2'
3/8"
BENT PLATE TIE
5/8"
3 1/4"
5/8"
1 1/2" x 9" LONG
THIS CAST IRON PURLIN SEAT REMAINS IN SEVERAL LOCATION.
CAST IRON PURLIN SEAT
THE PITCH OF THE PRINCIPAL TRUSSES IN THE ORIGINAL DWGS. SCALE 7 1/2 TO 12 OR APPROXIMATELY 32 DEGREES FROM THE HORIZONTAL. FIELD VERIFY
12
7 1/2
TRUSSED RIDGE 8"x9" SOUTHERN PINE
TWO 3/4" DIA. RODS
5 DETAIL @ APEX
S3.1 SCALE 3/4" = 1'-0"
TRUSSED PURLIN 8"x9" SOUTHERN PINE
10"x12" PRINCIPAL RAFTER SOUTHERN PINE
3"x6" JACK RAFTER 24" O.C.
BOLSTER (2) L 5 x 5 x 1/2 ANGLES OR (2) 6 x 4 x 1/2 LLV ANGLES 5'-0" LONG.
STRUTS (4) L 3 x 3 x 3/8 OR (4) L 3 x 3 x 1/4 OR (2) L 3 x 3 x 1/4 W/ (2) L 3 x 3 x 1/4.
4 SPLICE AND BOLSTER
S3.1 SCALE 3/4" = 1'-0"
5"
7 1/2" x 9" RIDGE
10 RIDGE @ DORMER
S3.1 SCALE 1 1/2" = 1'-0"
4 1/2"
7 7/8" x 11 1/4" (SIZE VARIES)
9 VALLEY
S3.1 SCALE 1 1/2" = 1'-0"
2 5/8"
6 1/4"
THIS CHANNEL IS A C10x15.3 STEEL CHANNEL.
10" DEEP CHANNEL W/ (2) 5/8" DIA. BOLTS @ 4'-0" O.C.
8 CHANNEL
S3.1 SCALE 1 1/2" = 1'-0"
THE PRINCIPAL RAFTER AT GRIDLINE 17.5 N HAS BEEN PREVIOUSLY REPAIRED USING SHEAR PLATES AND LAG SCREWS. RE-WORK THE CONNECTION AT THAT LOCATION USING A SCARFED JOINT CONNECTION AS DETAILED.
3"x6" JACK RAFTER 24" O.C.
10"x12" PRINCIPAL RAFTER SOUTHERN PINE
TRUSSED PURLIN 8"x9" SOUTHERN PINE
3 DETAIL @ PURLIN
S3.1 SCALE 3/4" = 1'-0"
3"x6" JACK RAFTER 24" O.C.
10"x12" PRINCIPAL RAFTER SOUTHERN PINE
(2) 1" DIA. RODS
8"x10" GIRT, SOUTHERN PINE
1 TRUSS HEEL @ SUPPORT
S3.1 SCALE 3/4" = 1'-0"
BRACE BAR 3/4" x 3"
SIDE VIEW
WROUGHT IRON
CAST IRON
PLAN VIEW
7 DETAIL
S3.1 SCALE 3" = 1'-0"
STRUT (4) L 3 x 3 x 3/8 SIZE MAY VARY SEE 4/S3.1
TWO 3/4" DIA. RODS
(2) 1" DIA. RODS
OR (2) 7/8" DIA. RODS
ONE 1 1/2" DIA. ROD
1 3/4" DIA. PIN. THE CONFIGURATION OF THIS CONNECTION VARIES.
6 DETAIL @ PIN
S3.1 SCALE 3/4" = 1'-0"
3"x6" JACK RAFTER 24" O.C.
6"x8"
4"x6" PLATE
6"x8"
4"x6" BRACE
4"x6" PURLIN
2 EAVE DETAIL
S3.1 SCALE 3/4" = 1'-0"
STATE OF VERMONT
DAVID C. FISCHETTI
No. 5353
CIVIL/STRUCTURAL
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PROFESSIONAL ENGINEER
DCF Engineering, Inc.
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MARCH 30, 2009
S3.1
05025

FOR REVIEW ONLY

EPOXY REPAIR

REPAIR BY SEGMENTAL INFILL

EPOXY SYSTEMS USED FOR TIMBER REPAIR ARE DIFFERENT THAN THOSE USED FOR CONCRETE.

A PIECE SIMILAR TO A DUTCHMAN INSERT CAN BE USED AS A FILLER. THE ADHESIVE USED CAN BE A GAP FILLING EPOXY TO SECURE A FILLER PIECE MADE FROM WOOD SALVAGED FROM ELSEWHERE IN THE BUILDING.

SHARPLY CUT AWAY ALL DETERIORATED WOOD PLUS A MINIMUM OF ONE QUARTER INCH OF SOUND WOOD. MAKE CUTS WITH A RECIPROCATING SAW OR ROUTER AND NOT A CIRCULAR SAW TO AVOID OVERCUTTING. LEAVE FACES OF CUT-OUT AREAS SQUARE AND PARALLEL. CUT THE REPLACEMENT BLOCK TO FIT SNUGLY BUT NOT TIGHT, WITH NOT MORE THAN 1/8" GAP AT ANY MEETING SURFACE. ENSURE THAT THE MOISTURE CONTENT OF BOTH THE MEMBER TO BE REPAIRED AND THE REPLACEMENT WOOD ARE SIMILAR. AVOID BUTT SPLICES. SLOPE OR SCARF THE ENDS OF PIECES JOINED.

MIX EPOXY ACCORDING TO MANUFACTURER'S INSTRUCTIONS AND PRECAUTIONS. NOTE THAT ONCE PART A AND PART B ARE COMBINED THE CHEMICAL PROCESS BY WHICH SETTING OCCURS CANNOT BE STOPPED. THEREFORE, MIX ONLY SUFFICIENT EPOXY TO REPAIR AREAS THAT HAVE BEEN PREPARED.

FULLY COAT ALL MATING SURFACES OF WOOD MEMBER AND REPAIR BLOCK. PLACE BLOCK INTO CUT-OUT AREA, ENSURING THAT ALL COATED FACES ARE IN FULL CONTACT, WITH NO VOIDS OR GAPS. TAP BLOCK INTO PLACE AND CLAMP FOR 48 HOURS. REMOVE EXCESS ADHESIVE FROM GLUE LINE. ENSURE THAT THE NEW AREA IS NOT DISTURBED DURING SETTING PERIOD.

IN EXPOSED AREAS NEATLY TOOL ADHESIVE AT GLUE LINE, BEING CAREFUL TO SCRAPE IT FROM ADJACENT SURFACES. EXCESS GLUE MAY HAVE TO BE SANDED AWAY AFTER CURING.

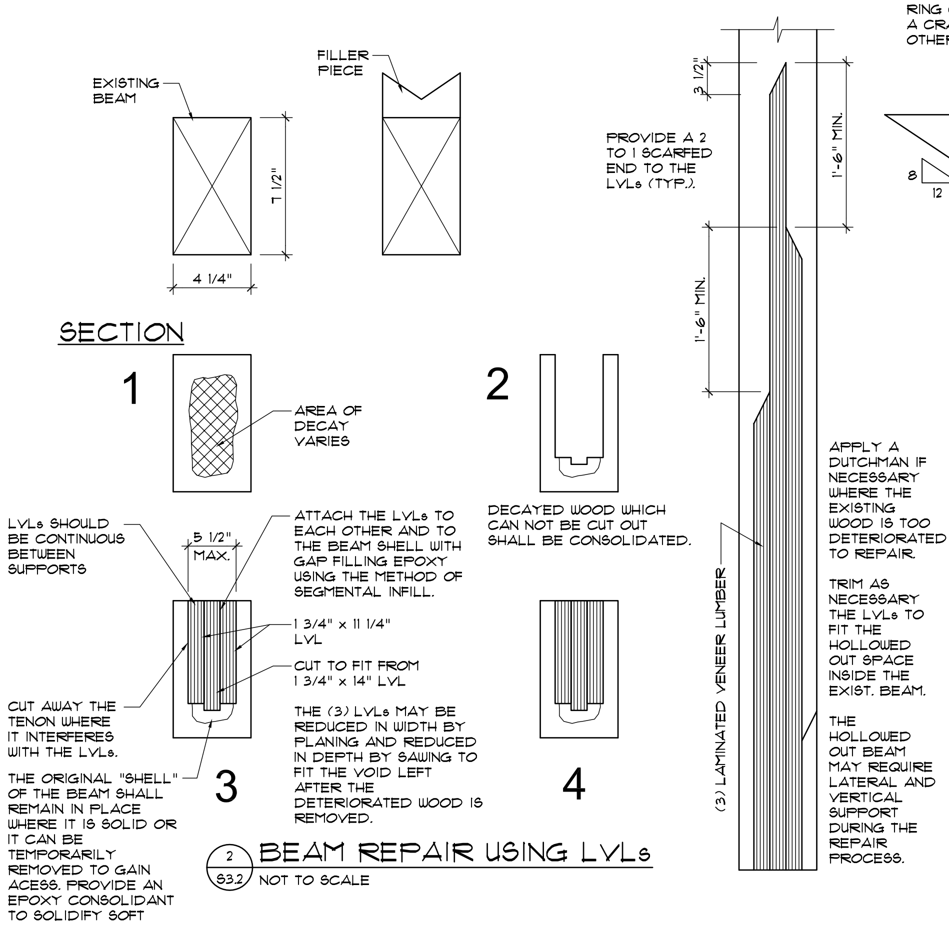
CRACK REPAIR

TO SUCCESSFULLY COMPLETE AN EPOXY CRACK REPAIR WILL REQUIRE COMPONENTS OF A SUITABLE VISCOSITY TO REACH ALL AREAS OF THE FRACTURE WITHOUT SPILLING OUT.

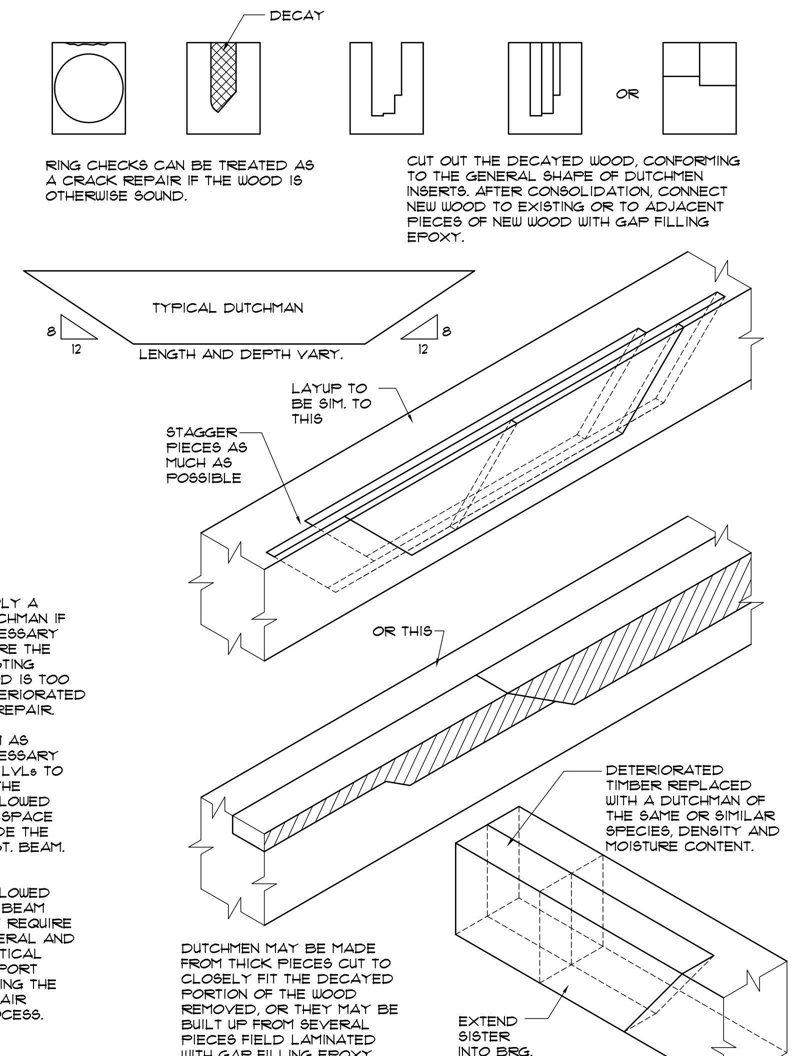
THE LOOSE WOOD CAN BE MADE SECURE BY USING EPOXY AS A CRACK REPAIR ADHESIVE. CONTAINING THE EPOXY WHILE ALSO ENSURING PENETRATION FULLY INTO EACH CRACK SHALL BE THE FOCUS OF THIS WORK.

SEASON CRACKS (CHECKS) DO NOT REQUIRE REPAIR.

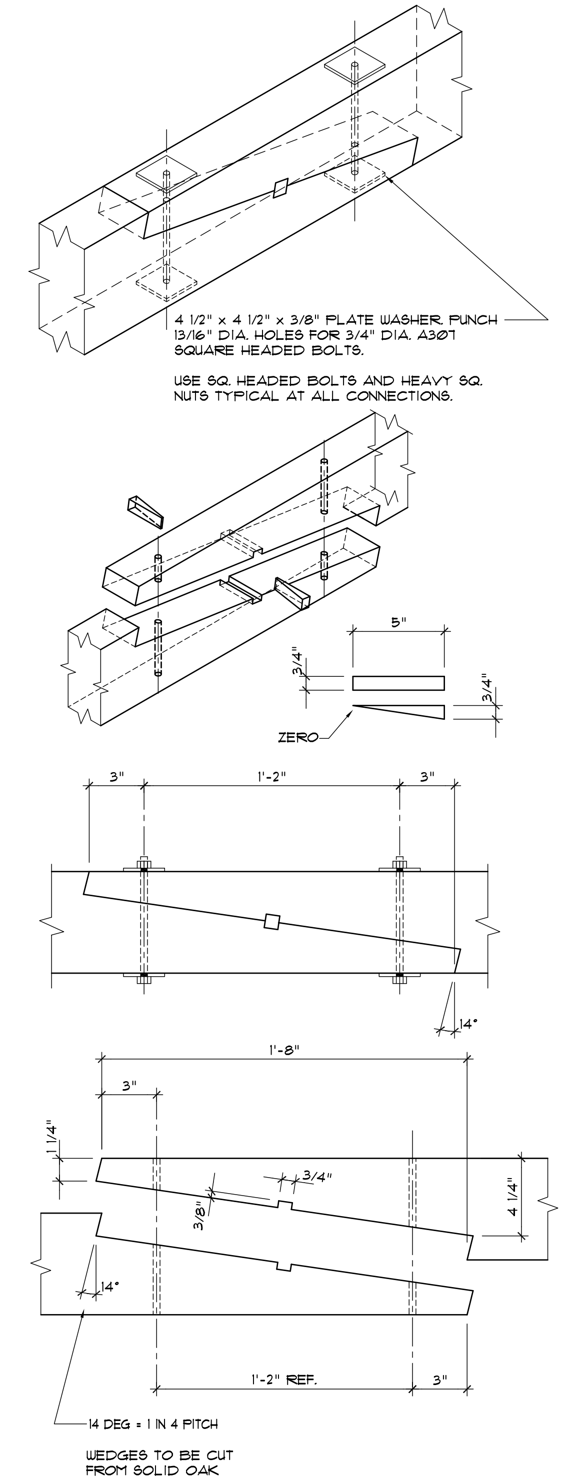
MIX EPOXY CONSOLIDANT ACCORDING TO MANUFACTURER'S INSTRUCTIONS, PRECAUTIONS, AND ENVIRONMENTAL REQUIREMENTS. INSTALL CONSOLIDANT IN SPONGY WOOD AND IN DRILLED HOLES USING APPLICATOR SUPPLIED BY MANUFACTURER OR PLASTIC SQUEEZE BOTTLES WITH SMALL DIAMETER NOZZLES. DO NOT OVERFILL HOLES. IF CONSOLIDANT RUNS FROM LOWER DRILLED HOLES, CHECKS, OR OPENINGS, FILL THESE WITH MODELING CLAY.



2 / S3.2 BEAM REPAIR USING LVLs
NOT TO SCALE



1 / S3.2 EPOXY REPAIRS
NOT TO SCALE



3 / S3.2 SPLICE DETAILS
SCALE VARIES

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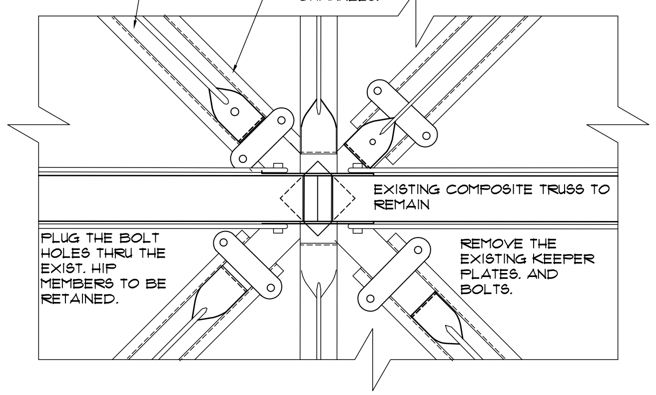
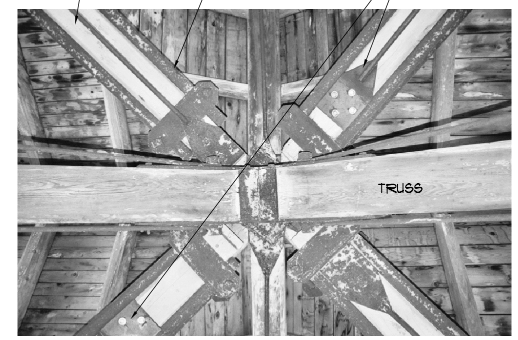


6 S3.3 REFLECTED PLAN VIEW DET. @ APEX
SCALE 3/4" = 1'-0"



SEE 8/S3/3 FOR DETAIL @ APEX

7 S3.3 PLAN VIEW @ APEX
NOT TO SCALE

3 S3.3 STONE WALL WITH CONCRETE
NOT TO SCALE

IN THE NEXT PHASE OF THE PROJECT THE EXISTING CONCRETE PARGING FROM THE EXTERIOR FACE OF THE STONE WALL WILL BE REMOVED AND THE WALL WILL BE RE-BUILT. THE EXTENT OF THIS WORK IS INDICATED ON SHEET S1.4

NOTE:
REPAIRS TO THE STONE WALL ARE NOT INCLUDED IN THIS PHASE OF THE PROJECT.

REPLACE THE FLAT PLATE AND (4) SMALL LAGS SCREWS WITH A CONNECTION SIMILAR TO THE ORIGINAL. PROVIDE A BENT PLATE CONNECTION KERFED INTO THE UNDERSIDE OF THE PRINCIPAL RAFTER OR VALLEY RAFTER.

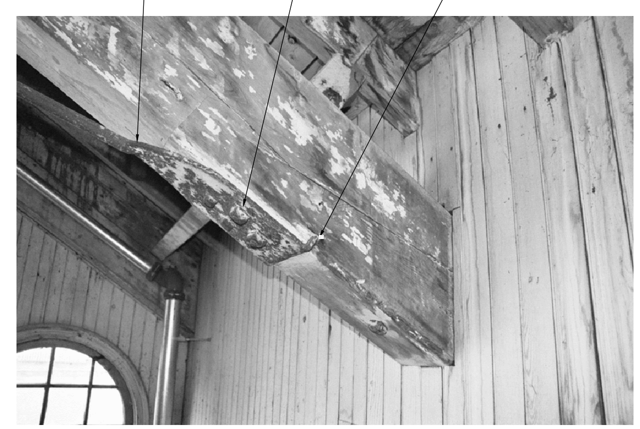
SEE 4/S3.3 FOR THE MORE APPROPRIATE CONNECTION.

THIS OCCURS AT 1N, 4.5S, AND AT PURLINS CONNECTED TO 8S AND 12.5N.

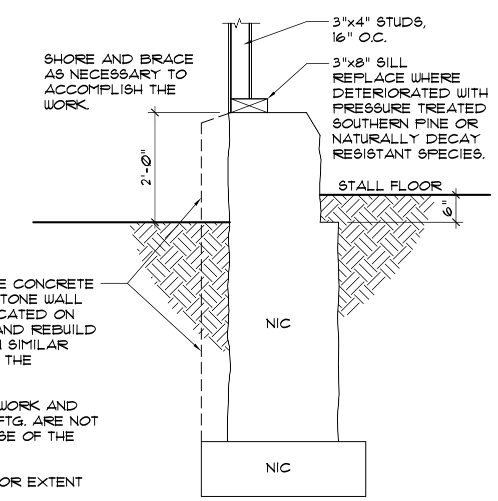
2 S3.3 DETAIL @ STONE WALL
NOT TO SCALE

NOTE:
REPAIRS TO THE STONE WALL ARE NOT INCLUDED IN THIS PHASE OF THE PROJECT

5 S3.3 MODIFIED QUEEN POST ROD CONNECTION
NOT TO SCALE



4 S3.3 ORIGINAL QUEEN POST ROD CONNECTION
NOT TO SCALE



1 S3.3 DETAIL @ EXT. WALL
SCALE 3/4" = 1'-0"

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POST REPAIR NOTES

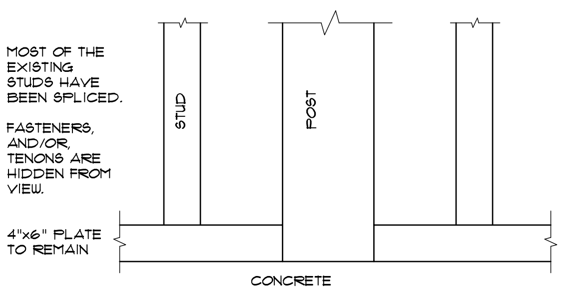
POST NO.	DESCRIPTION	NOTE
B 2.0	ASSUME NO REPAIR	
B 2.5	NO REPAIR	
B 3.0	NO REPAIR	
B 3.5	NORTH 12", WEST 16"	
B 4.0	NORTH 12", EAST 12", WEST 12"	
B 4.5	NEW COLUMN TO ELEVATION 11'-5" ABOVE FIN. FLOOR	●
B 5.0	NORTH 12", EAST 12"	
B 5.5	NORTH 12", EAST 12"	
B 6.0	NORTH 12" TO 16", EAST 12" TO 16", SOUTH 12" TO 16", WEST 12: TO 16"	
B 6.5	NORTH, SOUTH, EAST, WEST 12" TO 16", FILL SEASON CHECK	
B 7.0	NO REPAIR	
B 7.5	NORTH, SOUTH, EAST, WEST 12" TO 16"	
B 8.0	NORTH, EAST, WEST 12"	
B 8.5	EAST, SOUTH 12", PARTIAL NORTH POSSIBLE	
B 9.0	NO REPAIR	
B 9.5	NO REPAIR	
B 10.0	NO REPAIR	
B 10.5	WEST, SOUTH 12", SW CORNER 84"	
B 11.0	NORTH, EAST, SOUTH, WEST 12"	
B 11.5	LAP NEW COLUMN BOTTOM 5'-0" ON SOUTH	
B 12.0	NORTH, EAST 8"	
B 12.5	NORTH, EAST, SOUTH, WEST 8" TO 12", FILL SEASON CHECK ON WEST	
B 13.0	SOUTH, EAST 8"	
B 13.5	NORTH, EAST, SOUTH, WEST 12" TO 16"	
B 14.0	NORTH, EAST, SOUTH, WEST 8" TO 16"	
B 14.5	NEW COLUMN TO 12'-2"	●
B 15.0	NORTH, EAST, SOUTH, WEST 12" TO 16"	
B 15.5	NORTH, EAST 24"	
B 16.0	NO REPAIR	
B 16.5	NO REPAIR	
B 17.0	NO REPAIR	
G2 17.0	NEW BOTTOM @ 30"	●
H2 17.0	EAST 12", FILL SEASON CHECK	

● DENOTES TIMBER REPAIR PER DETAIL 2/S3.4.

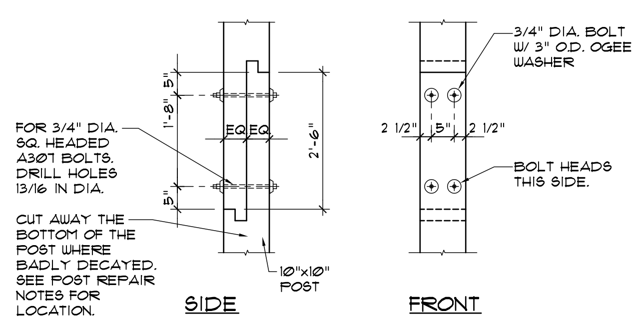
POST REPAIR NOTES

POST NO.	DESCRIPTION	NOTE
J 2.0	NO REPAIR	
J 2.5	NO REPAIR	
J 3.0	NO REPAIR	
J 3.5	REPLACE BOTTOM UP TO 2'-0"	
J 4.0	NO REPAIR	
J 4.5	REPLACE TIMBER POST UP TO 11'-6"	
J 5.0	NO REPAIR	●
J 5.5	NORTH, EAST, WEST 6" TO 12"	
J 6.0	SOUTH, EAST, WEST 8" TO 12", REMOVE CORNER ANGLE	
J 6.5	SOUTH, EAST, WEST 8" TO 12"	
J 7.0	NO REPAIR	
J 7.5	NORTH, SOUTH, EAST, WEST 12" TO 16", EPOXY REPAIR OF STONE	
J 8.0	SOUTH, EAST, WEST 6" TO 12"	
J 8.5	EAST 12" HIGH	
J 9.0	NO REPAIR	
J 9.5	NO REPAIR	
J 10.0	NO REPAIR	
J 10.5	FILL CHECK (E)	
J 11.0	REPLACE TIMBER TO 3 FT. TO 6 FT.	●
J 11.5	SOUTH, EAST 8" TO 12"	
J 12.0	NO REPAIR	
J 12.5	REPLACE TIMBER TO 11'-1" ABOVE FIN. FLOOR.	●
J 13.0	NORTH, EAST, SOUTH, WEST 8"	
J 13.5	NORTH, EAST, SOUTH, WEST 12" TO 16", COSMETIC REPAIR OF NE CORNER	
J 14.0	SOUTH, WEST 12" TO 16", REPLACE SW CORNER TO 7'-0"	
J 14.5	SOUTH, EAST, WEST 12" TO 16", REPLACE SE TO 6'-0"	●
J 15.0	NO REPAIR	
J 15.5	NORTH, EAST 12" TO 16"	
J 16.0	NO REPAIR	
J 16.5	FILL CHECK	
J 17.0	NO REPAIR	
F 2.0	NORTH, EAST, WEST 12" TO 16"	
E 2.0	FILL CHECK (W)	
C 2.0	EXAMPLE OF EXISTING SPLICE	

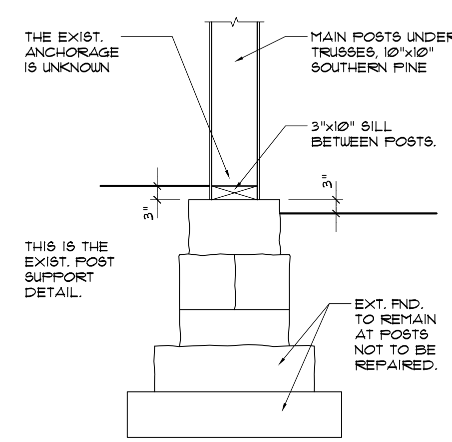
● DENOTES TIMBER REPAIR PER DETAIL 2/S3.4.



3/S3.4 POST DETAIL
SCALE 1 1/2" = 1'-0"



2/S3.4 TYP. POST SPLICE DETAIL
SCALE 3/4" = 1'-0"



1/S3.4 EXIST. DET. @ POSTS
SCALE 3/4" = 1'-0"

NOTES:

THE TIMBER FRAME CONTRACTOR/ SUBCONTRACTOR SHALL VERIFY THE CONDITION OF THE EXIST. POSTS AND THE EXTENT OF DETERIORATION USING RESISTANCE DRILLING OR OTHER MEANS PRIOR TO MAKING REPAIRS. THE RESULTS OF THIS INVESTIGATION SHALL BE SUBMITTED TO THE PROJECT MANAGER AND CONSULTING ENGINEER.

THE EXTENT OF THE PROPOSED REPAIRS SHALL REVIEWED BY THE PROJECT MANAGER AND THE CONSULTING ENGINEER PRIOR TO COMMENCING WORK.

THE DESIGN FOR TEMPORARY SHORING AND BRACING SHALL BE SUBMITTED TO THE PROJECT MANAGER AND CONSULTING ENGINEER PRIOR TO STARTING WORK.

05025

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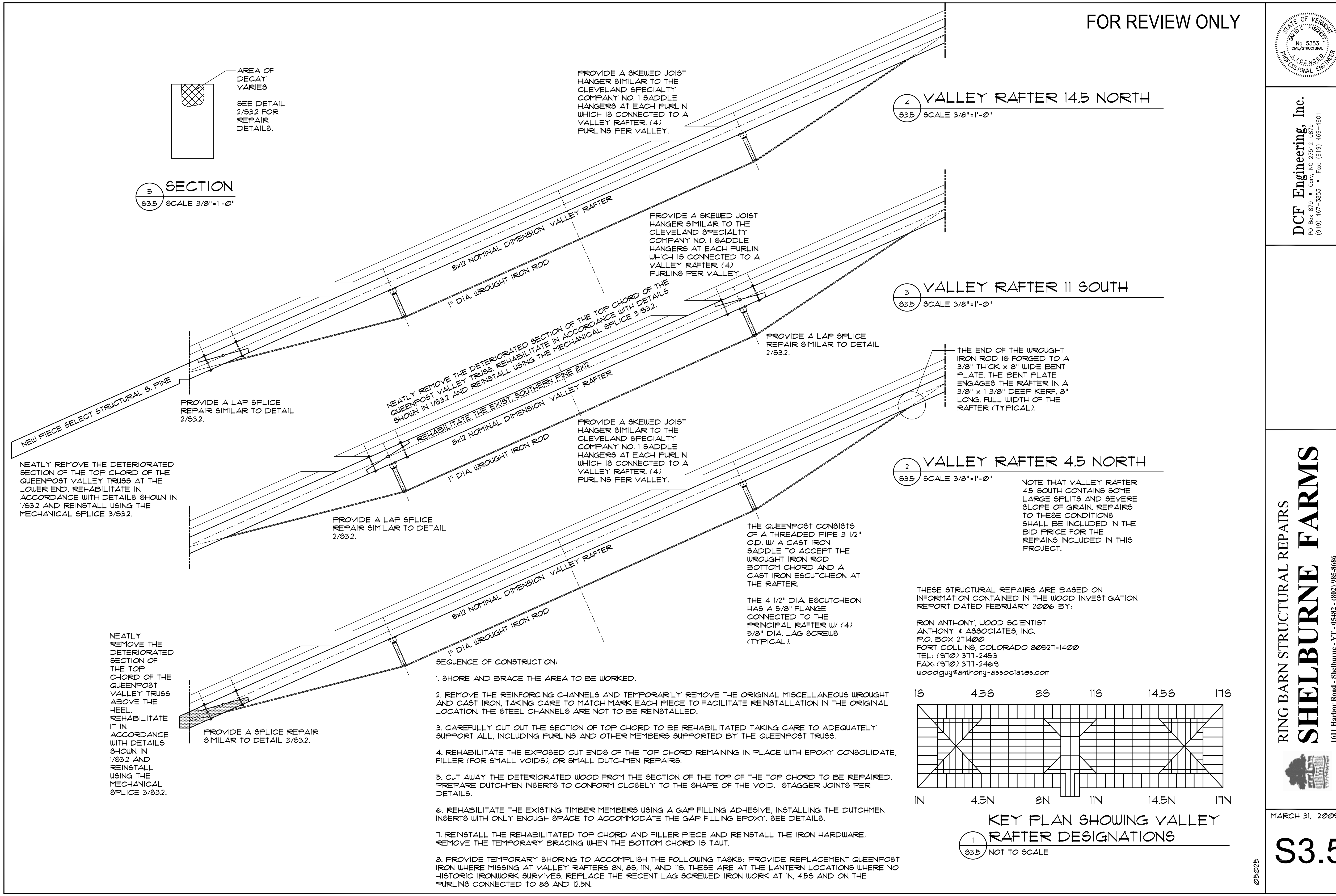
FOR REVIEW ONLY
AREA OF DECAY VARIES
SEE DETAIL 2/S3.2 FOR REPAIR DETAILS.
5 S3.5 SECTION
SCALE 3/8"=1'-0"
PROVIDE A SKEWED JOIST HANGER SIMILAR TO THE CLEVELAND SPECIALTY COMPANY NO. 1 SADDLE HANGERS AT EACH PURLIN WHICH IS CONNECTED TO A VALLEY RAFTER. (4) PURLINS PER VALLEY.
4 S3.5 VALLEY RAFTER 14.5 NORTH
SCALE 3/8"=1'-0"
8x12 NOMINAL DIMENSION VALLEY RAFTER
1" DIA. WROUGHT IRON ROD
NEW PIECE SELECT STRUCTURAL S. PINE
PROVIDE A LAP SPLICE REPAIR SIMILAR TO DETAIL 2/S3.2.
NEATLY REMOVE THE DETERIORATED SECTION OF THE TOP CHORD OF THE QUEENPOST VALLEY TRUSS AT THE LOWER END. REHABILITATE IN ACCORDANCE WITH DETAILS SHOWN IN 1/S3.2 AND REINSTALL USING THE MECHANICAL SPLICE 3/S3.2.
NEATLY REMOVE THE DETERIORATED SECTION OF THE TOP CHORD OF THE QUEENPOST VALLEY TRUSS. REHABILITATE IN ACCORDANCE WITH DETAILS SHOWN IN 1/S3.2 AND REINSTALL USING THE MECHANICAL SPLICE 3/S3.2.
REHABILITATE THE EXIST. SOUTHERN PINE 8x12
PROVIDE A SKEWED JOIST HANGER SIMILAR TO THE CLEVELAND SPECIALTY COMPANY NO. 1 SADDLE HANGERS AT EACH PURLIN WHICH IS CONNECTED TO A VALLEY RAFTER. (4) PURLINS PER VALLEY.
3 S3.5 VALLEY RAFTER 11 SOUTH
SCALE 3/8"=1'-0"
PROVIDE A LAP SPLICE REPAIR SIMILAR TO DETAIL 2/S3.2.
8x12 NOMINAL DIMENSION VALLEY RAFTER
1" DIA. WROUGHT IRON ROD
THE END OF THE WROUGHT IRON ROD IS FORGED TO A 3/8" THICK x 8" WIDE BENT PLATE. THE BENT PLATE ENGAGES THE RAFTER IN A 3/8" x 1 3/8" DEEP KERF, 8" LONG, FULL WIDTH OF THE RAFTER (TYPICAL).
PROVIDE A SKEWED JOIST HANGER SIMILAR TO THE CLEVELAND SPECIALTY COMPANY NO. 1 SADDLE HANGERS AT EACH PURLIN WHICH IS CONNECTED TO A VALLEY RAFTER. (4) PURLINS PER VALLEY.
2 S3.5 VALLEY RAFTER 4.5 NORTH
SCALE 3/8"=1'-0"
NOTE THAT VALLEY RAFTER 4.5 SOUTH CONTAINS SOME LARGE SPLITS AND SEVERE SLOPE OF GRAIN. REPAIRS TO THESE CONDITIONS SHALL BE INCLUDED IN THE BID PRICE FOR THE REPAIRS INCLUDED IN THIS PROJECT.
PROVIDE A LAP SPLICE REPAIR SIMILAR TO DETAIL 2/S3.2.
THE QUEENPOST CONSISTS OF A THREADED PIPE 3 1/2" O.D. W/ A CAST IRON SADDLE TO ACCEPT THE WROUGHT IRON ROD BOTTOM CHORD AND A CAST IRON ESCUTCHEON AT THE RAFTER.
THE 4 1/2" DIA. ESCUTCHEON HAS A 5/8" FLANGE CONNECTED TO THE PRINCIPAL RAFTER W/ (4) 5/8" DIA. LAG SCREWS (TYPICAL).
8x12 NOMINAL DIMENSION VALLEY RAFTER
1" DIA. WROUGHT IRON ROD
NEATLY REMOVE THE DETERIORATED SECTION OF THE TOP CHORD OF THE QUEENPOST VALLEY TRUSS ABOVE THE HEEL. REHABILITATE IT IN ACCORDANCE WITH DETAILS SHOWN IN 1/S3.2 AND REINSTALL USING THE MECHANICAL SPLICE 3/S3.2.
PROVIDE A SPLICE REPAIR SIMILAR TO DETAIL 3/S3.2.
SEQUENCE OF CONSTRUCTION:
1. SHORE AND BRACE THE AREA TO BE WORKED.
2. REMOVE THE REINFORCING CHANNELS AND TEMPORARILY REMOVE THE ORIGINAL MISCELLANEOUS WROUGHT AND CAST IRON, TAKING CARE TO MATCH MARK EACH PIECE TO FACILITATE REINSTALLATION IN THE ORIGINAL LOCATION. THE STEEL CHANNELS ARE NOT TO BE REINSTALLED.
3. CAREFULLY CUT OUT THE SECTION OF TOP CHORD TO BE REHABILITATED TAKING CARE TO ADEQUATELY SUPPORT ALL, INCLUDING PURLINS AND OTHER MEMBERS SUPPORTED BY THE QUEENPOST TRUSS.
4. REHABILITATE THE EXPOSED CUT ENDS OF THE TOP CHORD REMAINING IN PLACE WITH EPOXY CONSOLIDATE, FILLER (FOR SMALL VOIDS), OR SMALL DUTCHMEN REPAIRS.
5. CUT AWAY THE DETERIORATED WOOD FROM THE SECTION OF THE TOP OF THE TOP CHORD TO BE REPAIRED. PREPARE DUTCHMEN INSERTS TO CONFORM CLOSELY TO THE SHAPE OF THE VOID. STAGGER JOINTS PER DETAILS.
6. REHABILITATE THE EXISTING TIMBER MEMBERS USING A GAP FILLING ADHESIVE, INSTALLING THE DUTCHMEN INSERTS WITH ONLY ENOUGH SPACE TO ACCOMMODATE THE GAP FILLING EPOXY. SEE DETAILS.
7. REINSTALL THE REHABILITATED TOP CHORD AND FILLER PIECE AND REINSTALL THE IRON HARDWARE. REMOVE THE TEMPORARY BRACING WHEN THE BOTTOM CHORD IS TAUT.
8. PROVIDE TEMPORARY SHORING TO ACCOMPLISH THE FOLLOWING TASKS: PROVIDE REPLACEMENT QUEENPOST IRON WHERE MISSING AT VALLEY RAFTERS 8N, 8S, 11N, AND 11S. THESE ARE AT THE LANTERN LOCATIONS WHERE NO HISTORIC IRONWORK SURVIVES. REPLACE THE RECENT LAG SCREWED IRON WORK AT 1N, 4.5S AND ON THE PURLINS CONNECTED TO 8S AND 12.5N.
THESE STRUCTURAL REPAIRS ARE BASED ON INFORMATION CONTAINED IN THE WOOD INVESTIGATION REPORT DATED FEBRUARY 2006 BY:
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ANTHONY & ASSOCIATES, INC.
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FORT COLLINS, COLORADO 80527-1400
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FAX: (970) 377-2469
woodguy@anthony-associates.com
1S 4.5S 8S 11S 14.5S 17S
1N 4.5N 8N 11N 14.5N 17N
1 S3.5 KEY PLAN SHOWING VALLEY RAFTER DESIGNATIONS
NOT TO SCALE
05025
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S3.5

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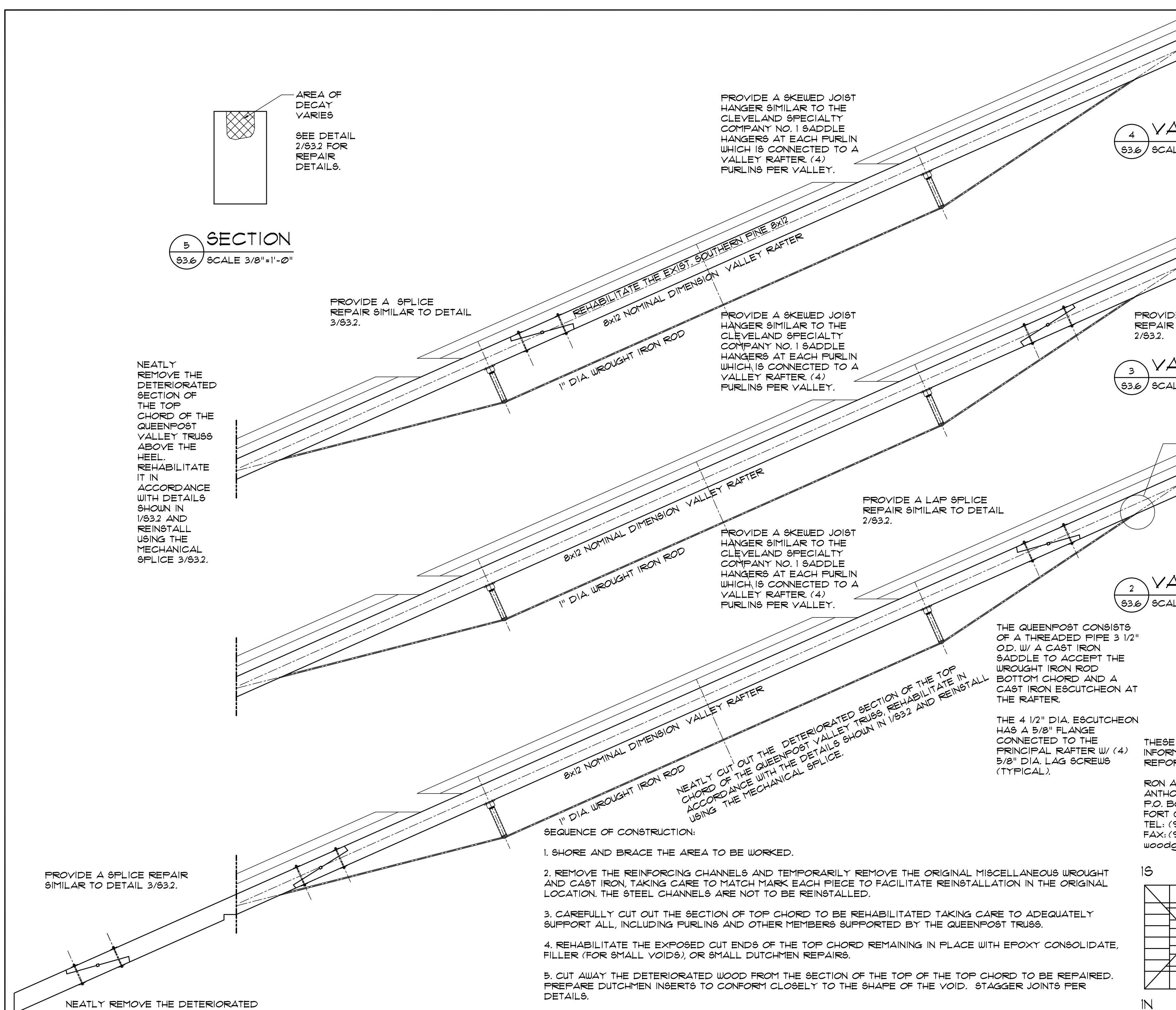

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SEQUENCE OF CONSTRUCTION:

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2. REMOVE THE REINFORCING CHANNELS AND TEMPORARILY REMOVE THE ORIGINAL MISCELLANEOUS WROUGHT AND CAST IRON, TAKING CARE TO MATCH MARK EACH PIECE TO FACILITATE REINSTALLATION IN THE ORIGINAL LOCATION. THE STEEL CHANNELS ARE NOT TO BE REINSTALLED.

3. CAREFULLY CUT OUT THE SECTION OF TOP CHORD TO BE REHABILITATED TAKING CARE TO ADEQUATELY SUPPORT ALL, INCLUDING PURLINS AND OTHER MEMBERS SUPPORTED BY THE QUEENPOST TRUSS.

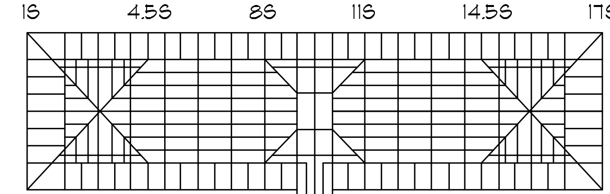
4. REHABILITATE THE EXPOSED CUT ENDS OF THE TOP CHORD REMAINING IN PLACE WITH EPOXY CONSOLIDATE, FILLER (FOR SMALL VOIDS), OR SMALL DUTCHMEN REPAIRS.

5. CUT AWAY THE DETERIORATED WOOD FROM THE SECTION OF THE TOP OF THE TOP CHORD TO BE REPAIRED. PREPARE DUTCHMEN INSERTS TO CONFORM CLOSELY TO THE SHAPE OF THE VOID. STAGGER JOINTS PER DETAILS.

6. REHABILITATE THE EXISTING TIMBER MEMBERS USING A GAP FILLING ADHESIVE, INSTALLING THE DUTCHMEN INSERTS WITH ONLY ENOUGH SPACE TO ACCOMMODATE THE GAP FILLING EPOXY. SEE DETAILS.

7. REINSTALL THE REHABILITATED TOP CHORD AND FILLER PIECE AND REINSTALL THE IRON HARDWARE. REMOVE THE TEMPORARY BRACING WHEN THE BOTTOM CHORD IS TAUT.

8. PROVIDE TEMPORARY SHORING TO ACCOMPLISH THE FOLLOWING TASKS: PROVIDE REPLACEMENT QUEENPOST IRON WHERE MISSING AT VALLEY RAFTERS 8N, 8S, 11N, AND 11S. THESE ARE AT THE LANTERN LOCATIONS WHERE NO HISTORIC IRONWORK SURVIVES. REPLACE THE RECENT LAG SCREWED IRON WORK AT 1N, 4.5S AND ON THE PURLINS CONNECTED TO 8S AND 12.5N.



KEY PLAN SHOWING VALLEY RAFTER DESIGNATIONS
1 S3.6 NOT TO SCALE

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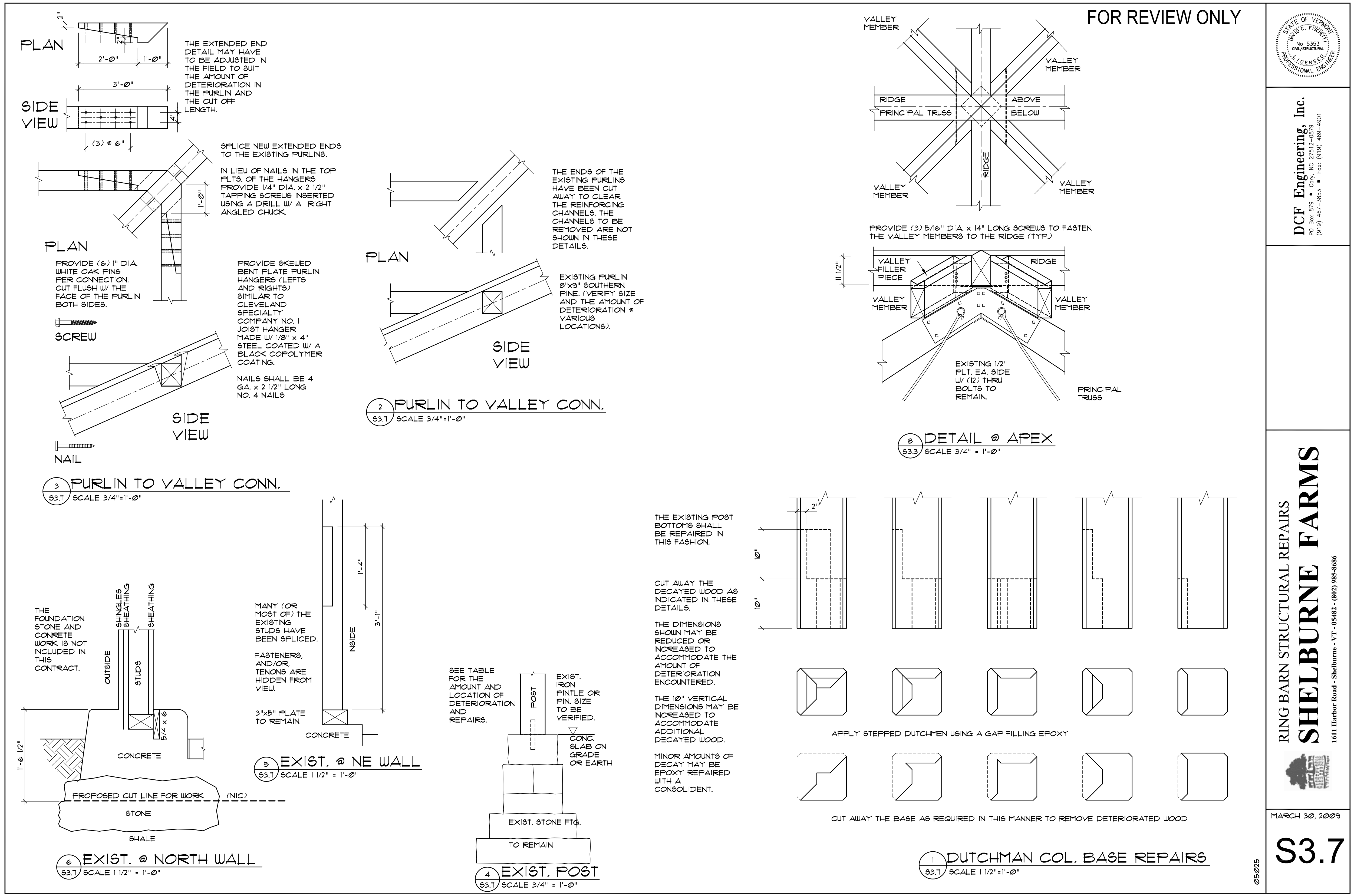
S3.6


FOR REVIEW ONLY
PLAN
2'-0" 1'-0"
3'-0"
SIDE VIEW
(3) @ 6"
THE EXTENDED END DETAIL MAY HAVE TO BE ADJUSTED IN THE FIELD TO SUIT THE AMOUNT OF DETERIORATION IN THE PURLIN AND THE CUT OFF LENGTH.
SPLICE NEW EXTENDED ENDS TO THE EXISTING PURLINS.
IN LIEU OF NAILS IN THE TOP PLTS. OF THE HANGERS PROVIDE 1/4" DIA. x 2 1/2" TAPPING SCREWS INSERTED USING A DRILL W/ A RIGHT ANGLED CHUCK.
1'-0"
PLAN
PROVIDE (6) 1" DIA. WHITE OAK PINS PER CONNECTION. CUT FLUSH W/ THE FACE OF THE PURLIN BOTH SIDES.
SCREW
PROVIDE SKEWED BENT PLATE PURLIN HANGERS (LEFTS AND RIGHTS) SIMILAR TO CLEVELAND SPECIALTY COMPANY NO. 1 JOIST HANGER MADE W/ 1/8" x 4" STEEL COATED W/ A BLACK COPOLYMER COATING.
NAILS SHALL BE 4 GA. x 2 1/2" LONG NO. 4 NAILS
SIDE VIEW
NAIL
3 PURLIN TO VALLEY CONN.
S3.7 SCALE 3/4"=1'-0"
THE ENDS OF THE EXISTING PURLINS HAVE BEEN CUT AWAY TO CLEAR THE REINFORCING CHANNELS. THE CHANNELS TO BE REMOVED ARE NOT SHOWN IN THESE DETAILS.
PLAN
EXISTING PURLIN 8"x9" SOUTHERN PINE. (VERIFY SIZE AND THE AMOUNT OF DETERIORATION @ VARIOUS LOCATIONS).
SIDE VIEW
2 PURLIN TO VALLEY CONN.
S3.7 SCALE 3/4"=1'-0"
VALLEY MEMBER
VALLEY MEMBER
RIDGE
ABOVE
PRINCIPAL TRUSS
BELOW
RIDGE
VALLEY MEMBER
VALLEY MEMBER
PROVIDE (3) 5/16" DIA. x 14" LONG SCREWS TO FASTEN THE VALLEY MEMBERS TO THE RIDGE (TYP.)
11 1/2"
VALLEY FILLER PIECE
RIDGE
VALLEY MEMBER
VALLEY MEMBER
EXISTING 1/2" PLT. EA. SIDE W/ (12) THRU BOLTS TO REMAIN.
PRINCIPAL TRUSS
8 DETAIL @ APEX
S3.3 SCALE 3/4" = 1'-0"
THE FOUNDATION STONE AND CONRETE WORK IS NOT INCLUDED IN THIS CONTRACT.
SHINGLES
SHEATHING
SHEATHING
OUTSIDE
STUDS
5/4 x 6
1'-6 1/2"
CONCRETE
PROPOSED CUT LINE FOR WORK (NIC)
STONE
SHALE
6 EXIST. @ NORTH WALL
S3.7 SCALE 1 1/2" = 1'-0"
MANY (OR MOST OF) THE EXISTING STUDS HAVE BEEN SPLICED.
FASTENERS, AND/OR, TENONS ARE HIDDEN FROM VIEW.
3"x5" PLATE TO REMAIN
CONCRETE
INSIDE
1'-4"
3'-1"
5 EXIST. @ NE WALL
S3.7 SCALE 1 1/2" = 1'-0"
SEE TABLE FOR THE AMOUNT AND LOCATION OF DETERIORATION AND REPAIRS.
POST
EXIST. IRON PINTLE OR PIN. SIZE TO BE VERIFIED.
CONC. SLAB ON GRADE OR EARTH
EXIST. STONE FTG.
TO REMAIN
4 EXIST. POST
S3.7 SCALE 3/4" = 1'-0"
THE EXISTING POST BOTTOMS SHALL BE REPAIRED IN THIS FASHION.
CUT AWAY THE DECAYED WOOD AS INDICATED IN THESE DETAILS.
THE DIMENSIONS SHOWN MAY BE REDUCED OR INCREASED TO ACCOMMODATE THE AMOUNT OF DETERIORATION ENCOUNTERED.
THE 10" VERTICAL DIMENSIONS MAY BE INCREASED TO ACCOMMODATE ADDITIONAL DECAYED WOOD.
MINOR AMOUNTS OF DECAY MAY BE EPOXY REPAIRED WITH A CONSOLIDENT.
2"
10"
10"
APPLY STEPPED DUTCHMEN USING A GAP FILLING EPOXY
CUT AWAY THE BASE AS REQUIRED IN THIS MANNER TO REMOVE DETERIORATED WOOD
1 DUTCHMAN COL. BASE REPAIRS
S3.7 SCALE 1 1/2"=1'-0"
05025
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MARCH 30, 2009
S3.7

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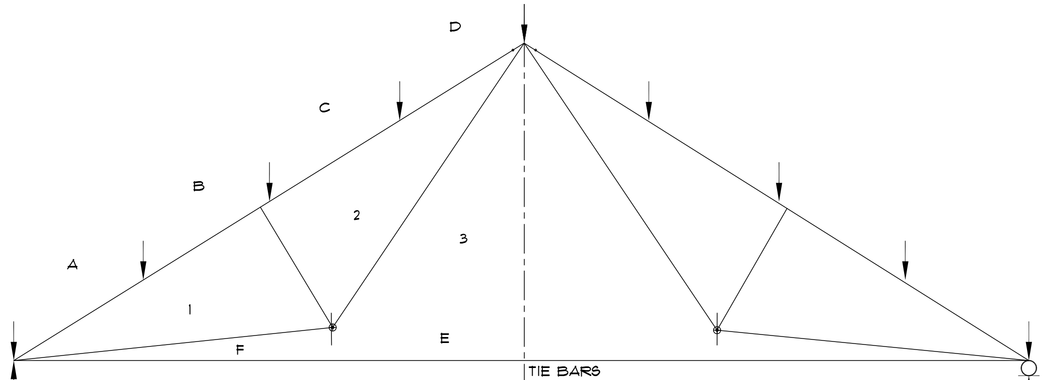
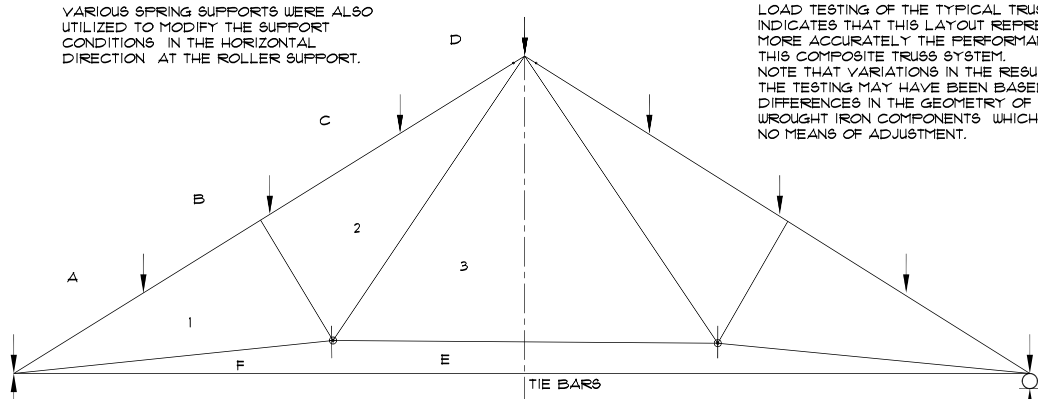


SUPPORT CONDITIONS USED IN THE ANALYSIS WERE A HINGED SUPPORT ON ONE SUPPORT AND A ROLLER SUPPORT IN THE X DIRECTION ON THE OTHER.

VARIOUS SPRING SUPPORTS WERE ALSO UTILIZED TO MODIFY THE SUPPORT CONDITIONS IN THE HORIZONTAL DIRECTION AT THE ROLLER SUPPORT.

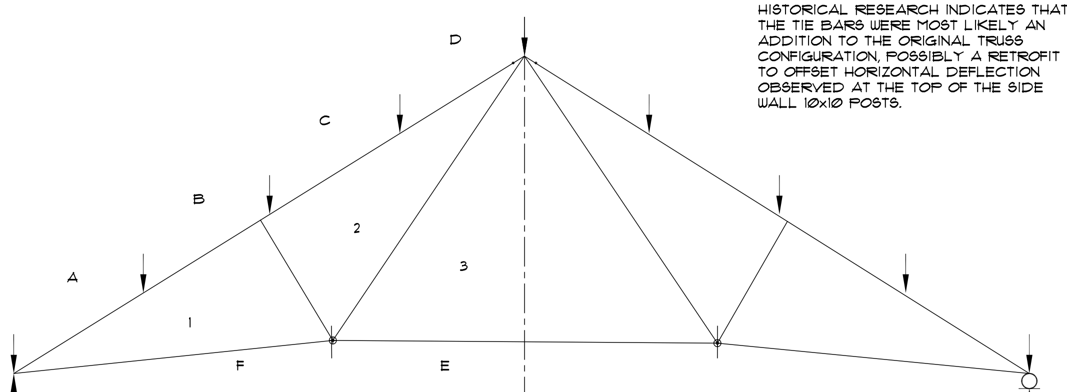
3 / S3.8 CASE 3
SCALE 3/16" = 1'-0"

LOAD TESTING OF THE TYPICAL TRUSS INDICATES THAT THIS LAYOUT REPRESENTS MORE ACCURATELY THE PERFORMANCE OF THIS COMPOSITE TRUSS SYSTEM.
NOTE THAT VARIATIONS IN THE RESULTS OF THE TESTING MAY HAVE BEEN BASED ON DIFFERENCES IN THE GEOMETRY OF THE WROUGHT IRON COMPONENTS WHICH HAVE NO MEANS OF ADJUSTMENT.



2 / S3.8 CASE 2
SCALE 3/16" = 1'-0"

HISTORICAL RESEARCH INDICATES THAT THE TIE BARS WERE MOST LIKELY AN ADDITION TO THE ORIGINAL TRUSS CONFIGURATION, POSSIBLY A RETROFIT TO OFFSET HORIZONTAL DEFLECTION OBSERVED AT THE TOP OF THE SIDE WALL 10x10 POSTS.



NOTE THAT GRAPHICAL ANALYSIS REQUIRES THAT THE TRUSS MODEL BE LOADED ONLY AT JOINTS (PANEL POINTS).
THE PURLIN POINT LOADS PLACED BETWEEN THE JOINTS CAUSE BENDING IN THE TOP CHORD WHICH MAY NOT HAVE BEEN ACCOUNTED FOR IN THE ORIGINAL ANALYSIS.

1 / S3.8 CASE 1
SCALE 3/16" = 1'-0"

TRUSS ANALYSIS MEMBER FORCES

MEMBER	CASE 1 FORCES (KIPS.)	CASE 2 FORCES (KIPS.)	CASE 3 FORCES (KIPS.)
A 1	73221 C	64466 C	66017 C
B 1	68129 C	59372 C	60923 C
C 2	63603 C	54783 C	56346 C
D 2	58531 C	49711 C	51274 C
E 3	36275 T	7811 T	N/A
F 1	61553 T	14405 T	22759 T
1 2	18945 C	19310 C	19245 C
2 3	27446 T	39547 T	22987 T
TIE BARS	N/A	N/A	30429 T

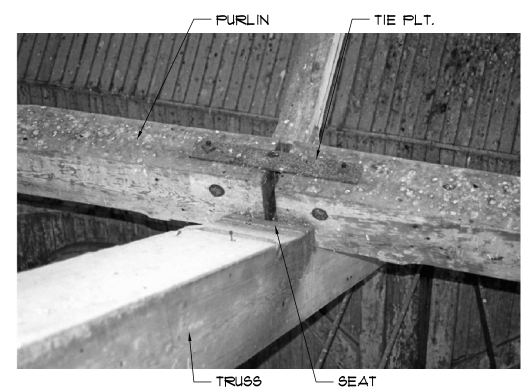
"C" DENOTES "COMPRESSION".
"T" DENOTES "TENSION".
DEAD LOAD= 15 PSF
SNOW LOAD= 30 PSF

THE ANALYSIS INCLUDED THE DOUBLE ANGLE BOLSTER BEAMS.

LOAD TESTS INDICATE THAT LOAD CASE 3 DESCRIBES BEST THE TRUE ACTION OF THE COMPOSITE TRUSS.

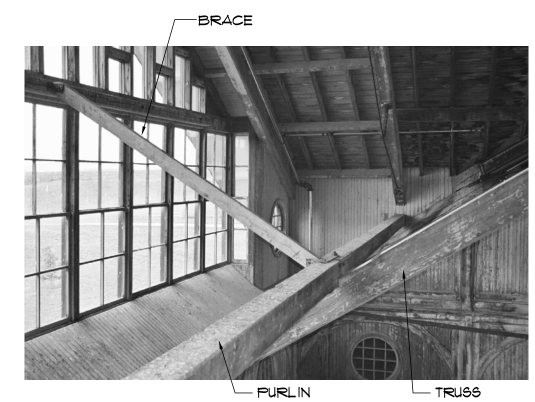
REWORK THE TRUSS END BY SPLICING IN A NEW PIECE WITH A SCARF JOINT ABOVE THE KNEE BRACE

7 / S3.8 BOTTOM CHORD REPAIR
NOT TO SCALE



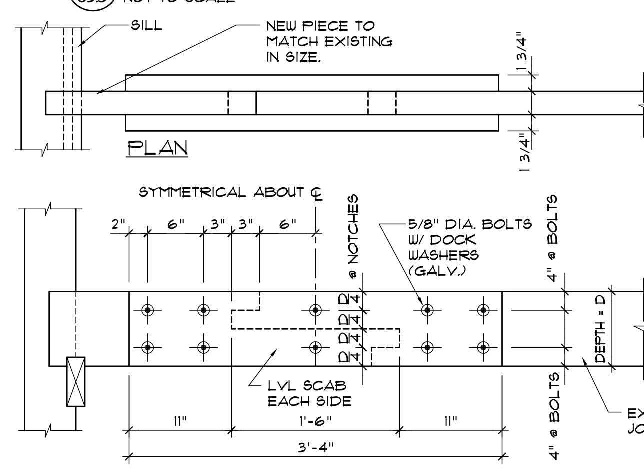
PROVIDE A TIE PLATE AND SEAT SIMILAR TO THE EXISTING HARDWARE AT OTHER LOCATIONS. TEMPORARILY DISMANTLE EXISTING JOINTS TO OBTAIN SAMPLES TO BE USED FOR DUPLICATION.

6 / S3.8 SEAT / TIE PURLINS
NOT TO SCALE



REPLACE THE PURLINS WHICH ARE MISSING. THE REPLACEMENT MEMBERS SHALL MATCH THE EXISTING IN SIZE AND DENSITY (RECYCLED TIMBER IF NECESSARY)

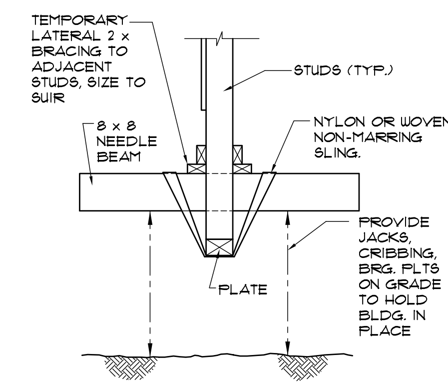
5 / S3.8 PURLIN BRACE DETAIL
NOT TO SCALE



THIS SPLICE MAY BE USED FOR DETERIORATED JOIST ENDS.

FOR SEVERE DETERIORATION BEYOND THE 1/4 POINT OF A JOIST, REPLACE RATHER THAN SPLICE.

4 / S3.8 FLOOR JOIST SPLICE
SCALE 1 1/2" = 1'-0"



8 / S3.8 SUGGESTED TEMPORARY WALL SUPPORT
NOT TO SCALE

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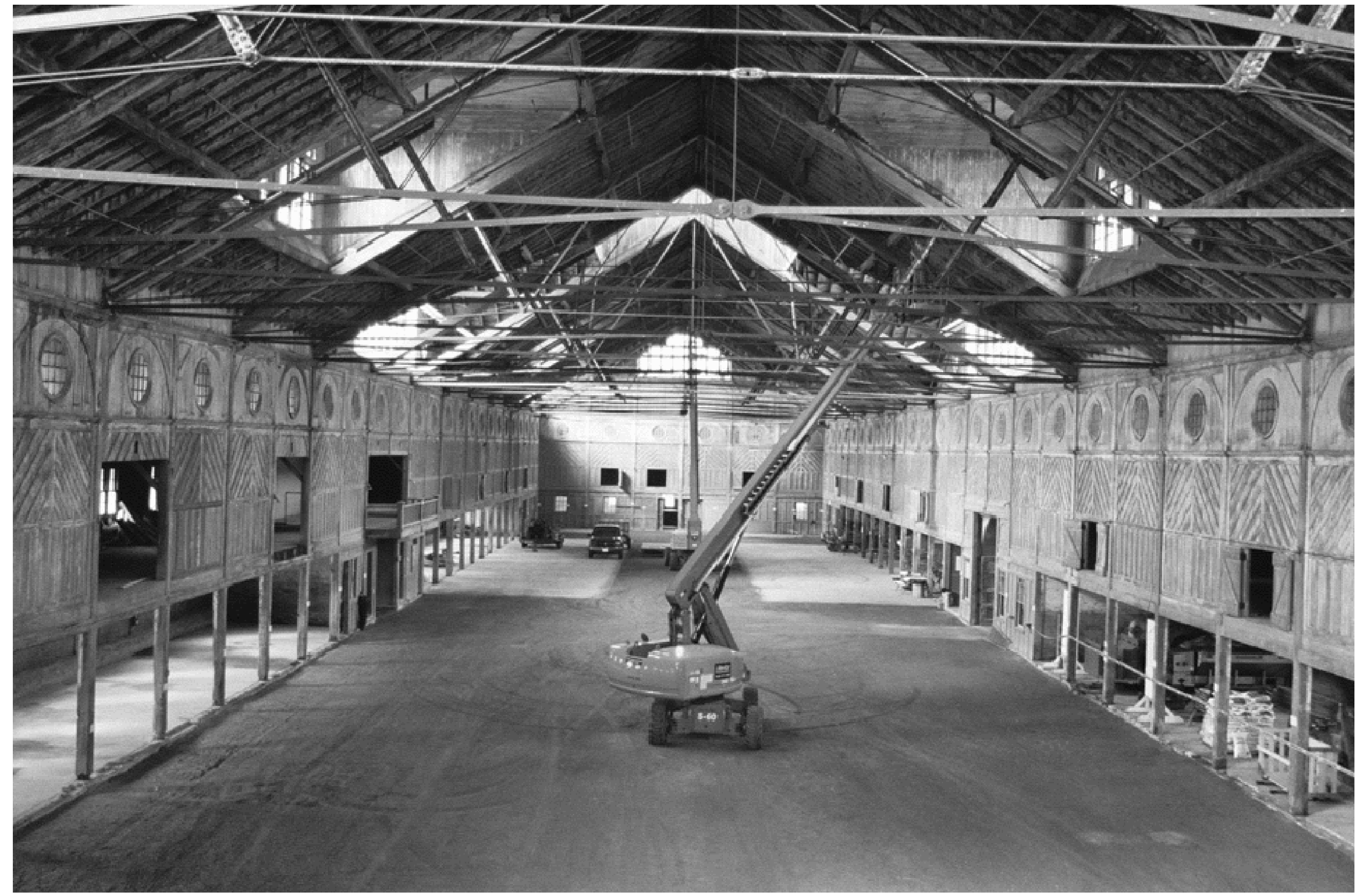
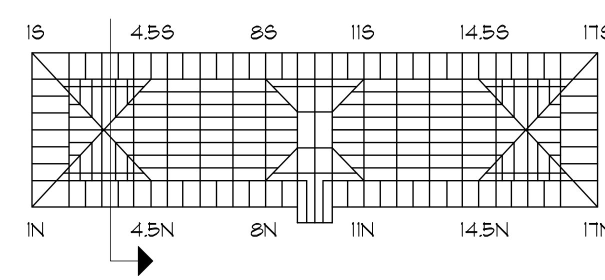
3 INTERIOR (CA. 1891)
P1.0 NOT TO SCALE



2 INTERIOR VIEW LOOKING WEST
P1.0 NOT TO SCALE



1 KEY PLAN
P1.0 NOT TO SCALE

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P1.0

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VALLEY RAFTER 1 NORTH VIEWED FROM THE APEX
1 / P1.1 NOT TO SCALE

VALLEY RAFTER 1 NORTH AT EAST WALL, NOTE DEFORMED LAGS SCREWS
2 / P1.1 NOT TO SCALE

VALLEY RAFTER 1 SOUTH VIEWED FROM THE APEX
3 / P1.1 NOT TO SCALE

VALLEY RAFTER 1 SOUTH, SEASON CHECK IN THE SOUTH FACE
4 / P1.1 NOT TO SCALE

VALLEY RAFTER 4.5 NORTH VIEWED FROM THE APEX
5 / P1.1 NOT TO SCALE

HEEL OF VALLEY RAFTER 4.5 NORTH AT THE NORTH WALL SHOWING DECAY IN THE RAFTER AND TOP PLATE
6 / P1.1 NOT TO SCALE

HEEL OF VALLEY RAFTER 4.5 NORTH, NOTE CORRODED NAILS NOT FULLY SECURING THE RAFTER TO THE TOP PLATE
7 / P1.1 NOT TO SCALE

VALLEY RAFTER 4.5 SOUTH AS VIEWED FROM THE APEX
8 / P1.1 NOT TO SCALE

THIS INFORMATION IS BASED ON A WOOD INVESTIGATION REPORT DATED FEBRUARY 2006 BY:
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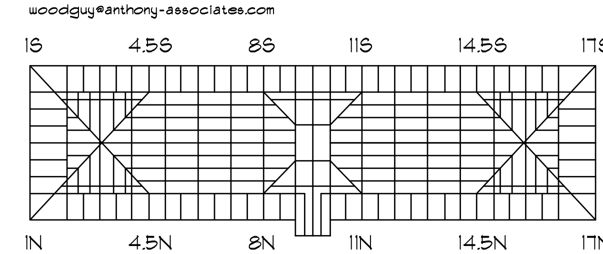

KEY PLAN SHOWING VALLEY RAFTER DESIGNATIONS
9 / P1.1 NOT TO SCALE

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P1.1

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SEASON CHECK INDICATING SEVERE SLOPE OF GRAIN (1.5" IN 10" RUN) IN VALLEY RAFTER 4.5 SOUTH

1 P1.2 NOT TO SCALE

VALLEY RAFTER 8 NORTH VIEWED FROM MID-LENGTH

2 P1.2 NOT TO SCALE

VALLEY RAFTER 8 NORTH AT DRILLING LOCATION D84

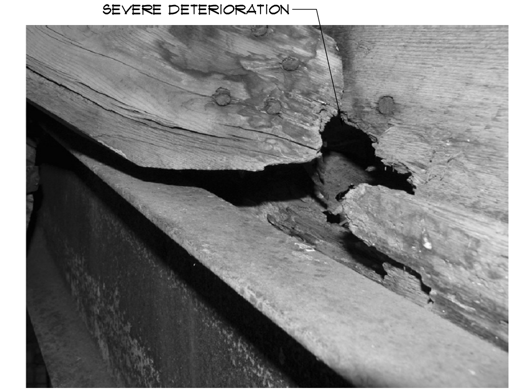
3 P1.2 NOT TO SCALE

VALLEY RAFTER 8 SOUTH VIEWED FROM THE APEX

4 P1.2 NOT TO SCALE

VALLEY RAFTER 11 NORTH SHOWING SEASON CHECKS

5 P1.2 NOT TO SCALE

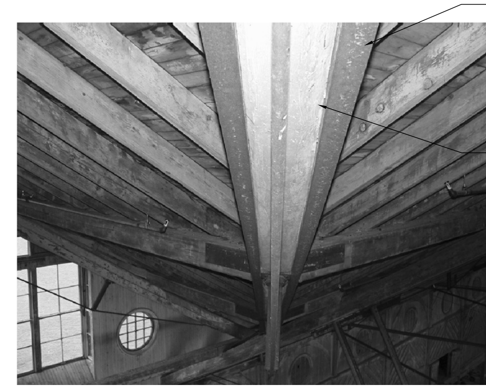


TOP OF VALLEY RAFTER 11 SOUTH NEAR DRILLING D74

6 P1.2 NOT TO SCALE

UPPER PURLIN TWISTED AT THE JUNCTION WITH VALLEY RAFTER 11 SOUTH

7 P1.2 NOT TO SCALE



VALLEY RAFTER 14.5 NORTH VIEWED FROM THE APEX

8 P1.2 NOT TO SCALE

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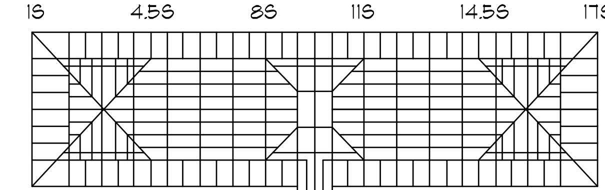

KEY PLAN SHOWING VALLEY RAFTER DESIGNATIONS

9 P1.2 NOT TO SCALE

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P1.2

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VALLEY RAFTER 17 NORTH, ABOVE DRILLING LOCATION D12 WITH A SPLIT END

1 / P1.4 NOT TO SCALE

FRACTURE NEAR THE APEX OF VALLEY RAFTER 17 NORTH BELOW THE END SPLIT SHOWN IN 3/P1.3

2 / P1.4 NOT TO SCALE

VALLEY RAFTER 17 SOUTH AS VIEWED FROM THE APEX

3 / P1.4 NOT TO SCALE

VALLEY RAFTER 17 SOUTH BELOW DRILLING D23

4 / P1.4 NOT TO SCALE

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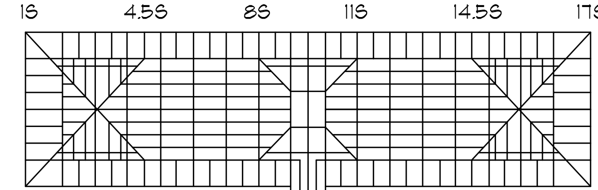

KEY PLAN SHOWING VALLEY RAFTER DESIGNATIONS

5 / P1.4 NOT TO SCALE

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Section II: Reports and Test Data

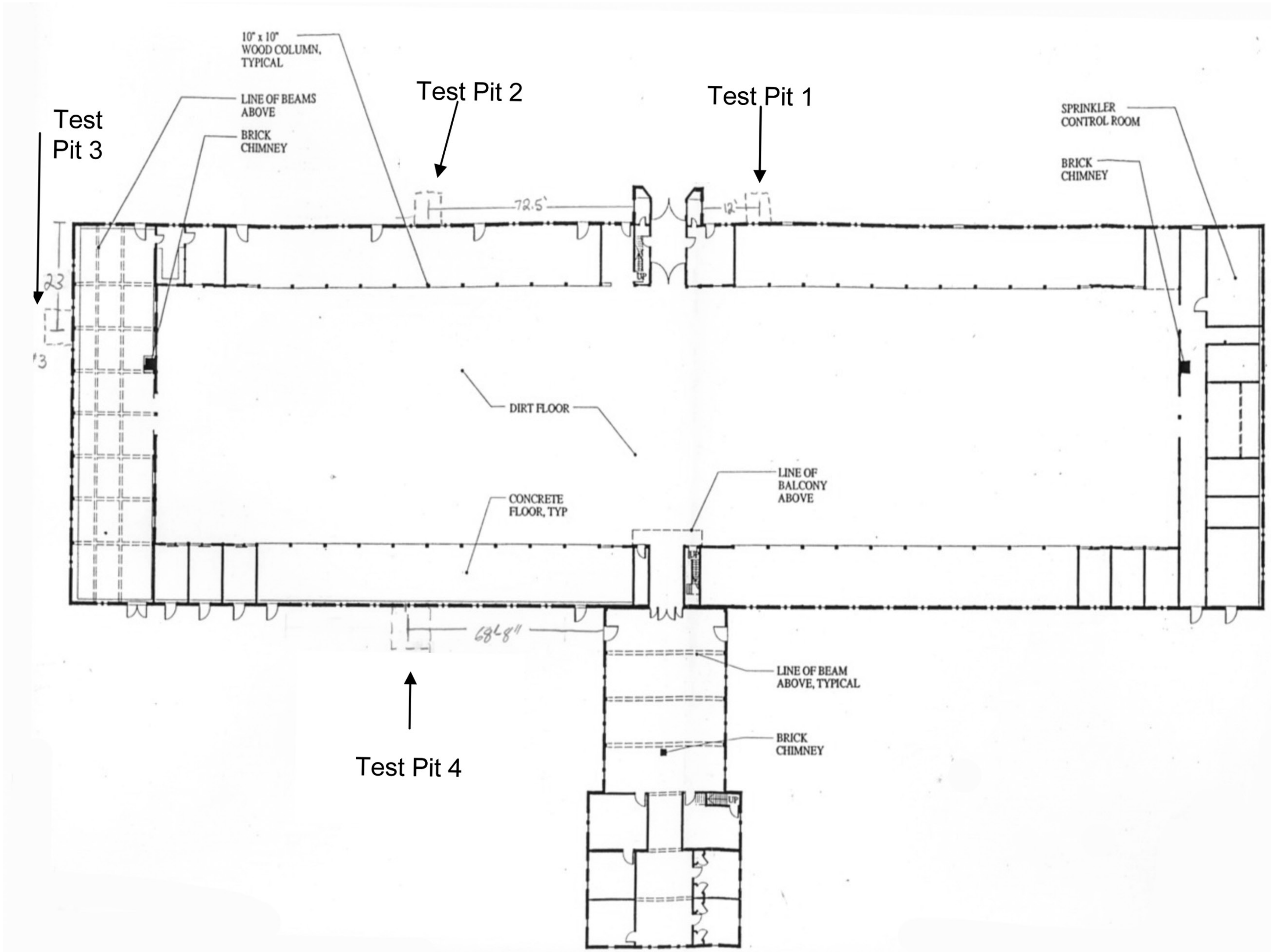
Appendix E: Geotechnical Investigation

Above-grade foundation stonework on the east, west, and south facades of the main block, as well as on the annex, were in fair to good condition prior to treatment. During the latter half of the 20th century, significant changes were made to the foundations on the north façade. East of the main entrance, above-grade stonework was replaced with a reinforced concrete grade beam placed directly on the original subsurface stonework.

West of the entrance, a reinforced concrete counter wall was added to foundation stonework on the building exterior, above and below grade, and a concrete curb was added against the timber sill on the interior. The counter wall was poorly detailed, and encased the timber sills and the bottoms of columns and studs. Most of the encased timber was decayed as a result, and there were visible displacements in the north wall frame.

Foundation settlement has been minimal and in the opinion of the design team, extensive geotechnical investigations were unnecessary. In 2006, test pits were dug on north, south and west sides of the main block to inspect subsurface conditions. In general, foundation stonework was placed on a layer of ledge at varying depths (typically 3-5 feet). Working with a consulting engineer and geotechnical faculty at the University of Vermont, foundation construction was documented, site soils were characterized, strength-in-shear was determined (ASTM D-2573; Test Method for Field Vane Shear Test in Cohesive Soil), and building loads were calculated.

This appendix includes a ground plan showing test pit locations, soil characterization worksheets, vane-shear test results, and calculations of building loads.



Test Pit Locations

Test pit 3 Bulk sample

Seive No.	Sieve opening (mm)	Mass of Empty	Mass of (sieve	Mass of soil	Percent of mass	Cumulative percent	Percent finer.
4	4.75	444.5	589	144.5	10.31	10.31	89.69
10	2	491.8	752.4	260.6	18.59	28.89	71.11
40	0.425	462.2	1038.5	576.3	41.10	70.00	30.00
60	0.25	371.9	519.3	147.4	10.51	80.51	19.49
80	0.18	364	442.4	78.4	5.59	86.10	13.90
140	0.106	350.4	430.7	80.3	5.73	91.83	8.17
200	0.075	343.2	395.5	52.3	3.73	95.56	4.44
Pan		365.2	428	62.8			
	Sum of mass			1402.6			
	Mass loss			-0.04%			

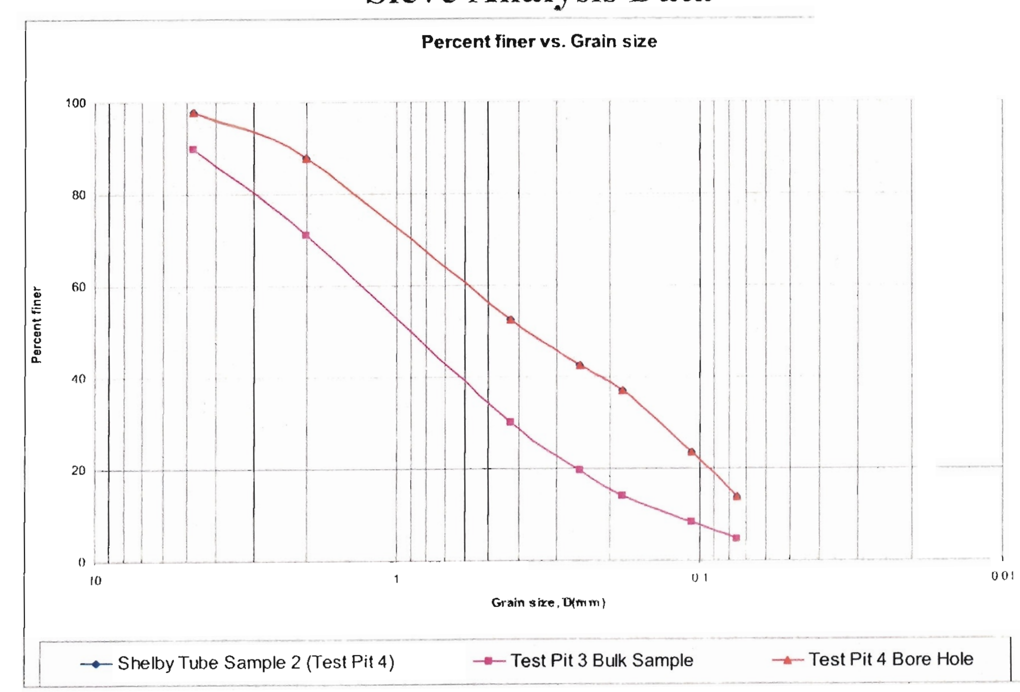
Test pit 4 Bulk sample

Mass of the dry specimen:			M2-M1= 887.54g				
Seive No.	Sieve opening (mm)	Mass of Empty Sieve(M3)	Mass of (sieve and dry soil)(M4)	Mass of soil retained on each sieve, Wn	Percent of mass retained on each sieve, Rn	Cumulative percent retained, Sum Rn	Percent finer, 100-(sum Rn)
4	4.75	511.7	551.1	39.4	4.44	4.44	95.56
10	2	492.8	574	81.2	9.15	13.59	86.41
40	0.425	392	691.9	299.9	33.79	47.38	52.62
60	0.25	371.4	465.6	94.2	10.61	57.99	42.01
80	0.18	436.1	493.8	57.7	6.50	64.49	35.51
140	0.106	350	443.5	93.5	10.53	75.03	24.97
200	0.075	341.7	389.9	48.2	5.43	80.46	19.54
Pan		364.86	537.6	172.74			
	Sum of mass			886.84			
	Mass loss			0.08%			

Test pit 4 Shelby tube sample.

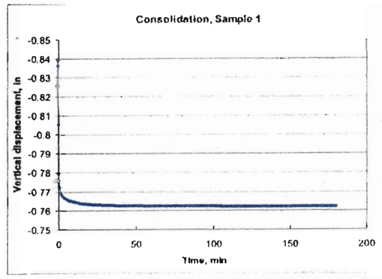
Seive No.	Sieve opening (mm)	Mass of Empty Sieve(M3)	Mass of (sieve and dry soil)(M4)	Mass of soil retained on each sieve, W_n	Percent of mass retained on each sieve, Rn	Cumulative percent retained, Sum Rn	Percent finer, 100-(sum Rn)
4	4.75	511.7	529.5	17.8	2.28	2.28	97.72
10	2	492.8	570	77.2	9.90	12.18	87.82
40	0.425	392	668.3	276.3	35.43	47.61	52.39
60	0.25	371.4	448.4	77	9.87	57.49	42.51
80	0.18	436.1	478.6	42.5	5.45	62.94	37.06
140	0.106	350	455	105	13.46	76.40	23.60
200	0.075	341.7	418.6	76.9	9.86	86.27	13.73
Pan		364.8	470.3	105.5			
	Sum of mass			778.2			
	Mass loss			0.21%			

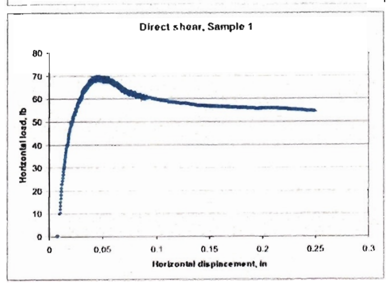
Sieve Analysis Data

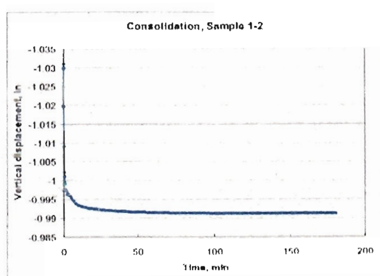


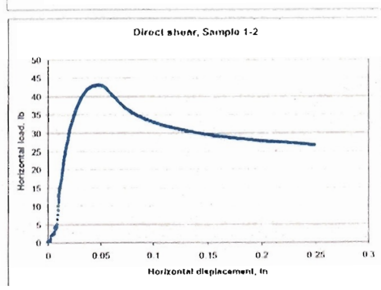
Direct Shear Test Reports

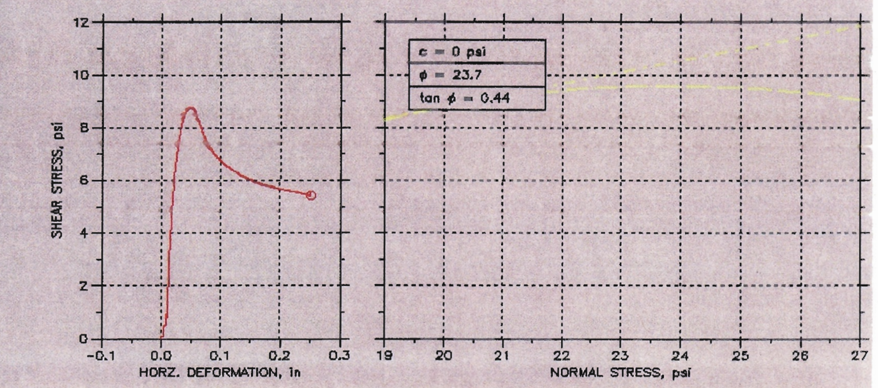
Test Pit 1









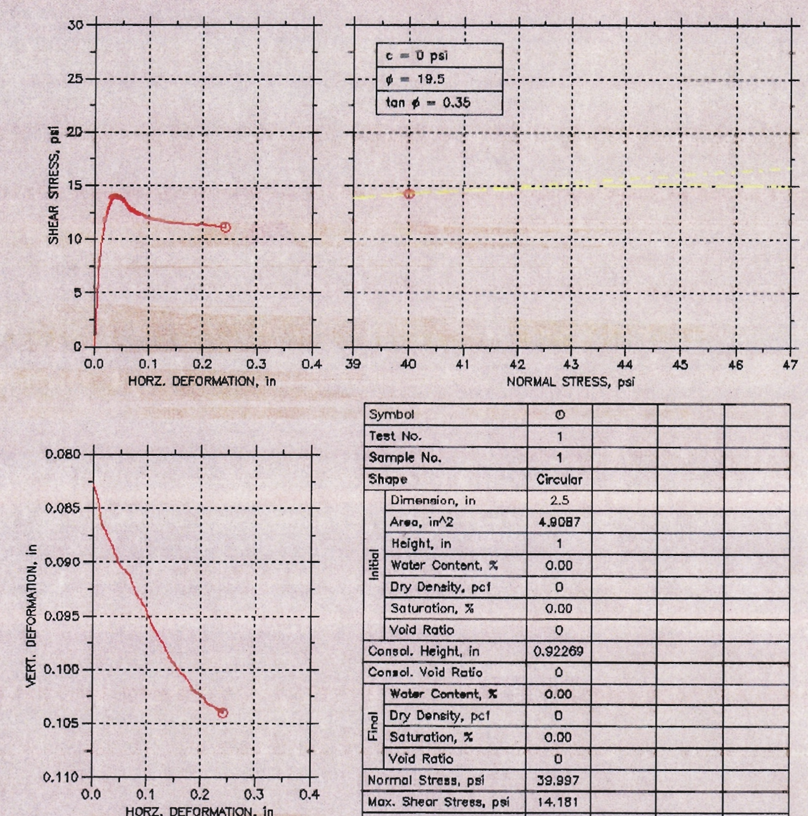


DIRECT SHEAR TEST REPORT

c = 0 psi
ϕ = 23.7
tan ϕ = 0.44

Symbol		⊙			
Test No.		2			
Sample No.		1			
Shape		Circular			
Initial	Dimension, in	2.5			
	Area, in^2	4.9087			
	Height, in	1			
	Water Content, %	0.00			
	Dry Density, pcf	0			
	Saturation, %	0.00			
	Void Ratio	0			
Consol. Height, in		0.96012			
Consol. Void Ratio		0			
Final	Water Content, %	0.00			
	Dry Density, pcf	0			
	Saturation, %	0.00			
	Void Ratio	0			
Normal Stress, psi		19.998			
Max. Shear Stress, psi		8.7634			
Ult. Shear Stress, psi		5.4232			
Time to Failure, min		173			
Disp. Rate, in/min		0.0008			
Measured Specific Gravity		0.00			
Liquid Limit		---			
Plastic Limit		---			
Plasticity Index		---			

Project: breeding barn
Location:
Project No.: pit 1
Boring No.: 1
Sample Type: shelby
Description: test test #1
Remarks:



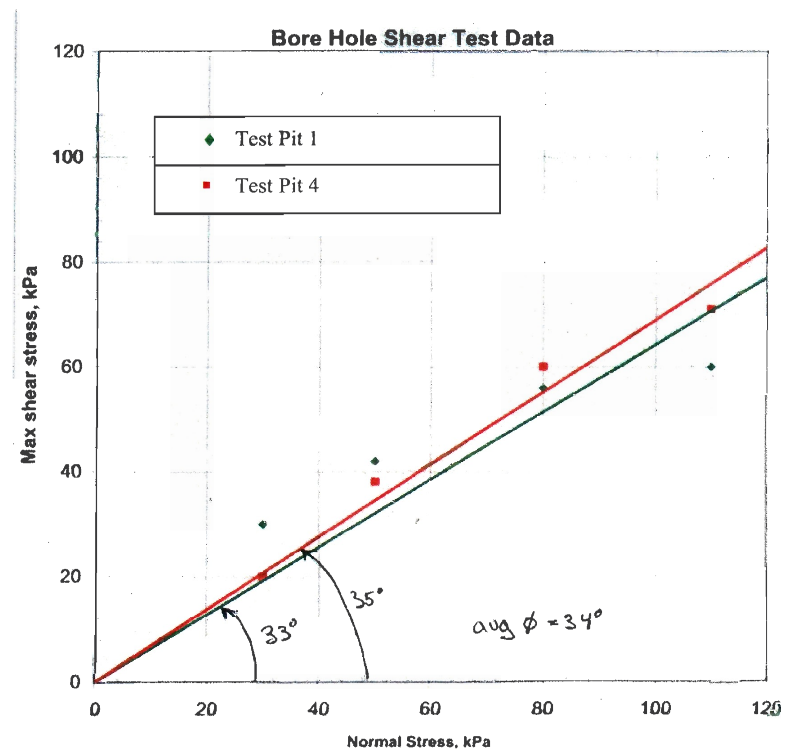
DIRECT SHEAR TEST REPORT

c = 0 psi
ϕ = 19.5
tan ϕ = 0.35

Symbol		⊙			
Test No.		1			
Sample No.		1			
Shape		Circular			
Initial	Dimension, in	2.5			
	Area, in^2	4.9087			
	Height, in	1			
	Water Content, %	0.00			
	Dry Density, pcf	0			
	Saturation, %	0.00			
	Void Ratio	0			
Consol. Height, in		0.92269			
Consol. Void Ratio		0			
Final	Water Content, %	0.00			
	Dry Density, pcf	0			
	Saturation, %	0.00			
	Void Ratio	0			
Normal Stress, psi		39.997			
Max. Shear Stress, psi		14.181			
Ult. Shear Stress, psi		11.081			
Time to Failure, min		122			
Disp. Rate, in/min		0.0008			
Measured Specific Gravity		0.00			
Liquid Limit		---			
Plastic Limit		---			
Plasticity Index		---			

Project: breeding barn
Location:
Project No.: pit 1
Boring No.: 1
Sample Type: shelby
Description: test test #1
Remarks:

Test Pit 1		Test Pit 4	
Normal Stress, kPa	Shear Stress, kPa	Normal Stress, kPa	Shear Stress, kPa
30	30	30	20
50	42	50	38
80	56	80	60
110	60	110	71



Load Calculations

Roof truss   truss spacing = 24'   14 trusses From Plan

Pine → 8" × 9"   8" × 9" × (70' × 12") = 60480 in³ = 35 Ft³ × 14 = $\boxed{490\ Ft^3}$

Iron Roof truss   1.5 d rod   45' long   9.33'   24'

(πr² × Length) 2 = (1.8 × (45 × 12)) = 972 in³ × 2 = 1.125 Ft³
(1.8 × (9.33 × 12)) = 202 in³ = 0.24 Ft³
+ (1.8 × (24 × 12)) = 519 in³ = 0.60
Σ = ≈ 2 Ft³ × 14 trusses

Total $\boxed{V = 28\ Ft^3}$ Iron

419' / (16/12) = 314 studs   Volume = 3" × 4" = 12 in² / 144 × 18.833
V = 1.57 Ft³
V × # Studs = 314 × 1.57 = 493
Studs total V = $\boxed{493\ Ft^3}$ wall studs

Side truss + Floor

Truss 6" × 8" spaced 12'   8' + 9' long   419/12 = 35 trusses
Floor 3" × 10" Spaced 18"   8' long   (419 × 12)/18 = 279 Floor

each truss   6 × 8 = 48 in² × (8 × 12) + (48 × (9 × 12)) = 4644 in³
V = 2.7 Ft³ × 35 trusses = $\boxed{94.5\ Ft^3}$ Side truss

each Floor   3 × 10 = 30 in² × (12 × 8) = 2880 in³ = 1.7 Ft³
V = 1.7 × 279 Floors = $\boxed{474.3\ Ft^3}$ Floors

Roof   3" × 6" 24 in Spacing   length 70' bottom to peak
Area = 419 × 70 = 29330 Ft²
3 × 6 = 24 in² × (70 × 12) = 11.66 Ft³   Cu density = 15 lb/Ft²
11.66 × 419/2 = $\boxed{2443\ Ft^3}$ Roof Studs   W = 15 × 29330 = $\boxed{440\ kips\ Cu}$
Snow W = $\boxed{879.9\ kips\ Snow}$
29330 Ft² × 30 PSF

Added Live load on exterior Floor

Area of Floor – 8' × 419' = 3352. Ft²

Storage live = 250. lb/Ft²   Live on Floor = $\boxed{838 \text{ kips}}$

Foundation weight -

Volume – 2' × 5' × 419' = 4190 Ft³

γ_{conc} = 150 pcf   4190 × 150 = $\boxed{628.5 \text{ kips}}$

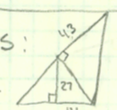
reinforced stone

Plywood on ceiling and walls

Wall Vol – h × l × t = 18.83' × 419' × (.75/12)' = $\boxed{493.11 \text{ Ft}^3}$

Ceiling Vol – l × w × t = 419' × 70' × (.75/12)' = $\boxed{1833 \text{ Ft}^3}$

Dormers:



Face – = 0.5 × 27 × 14 = 189 × 6 halves = 1134 Ft²

43
27
14.66

t = 3/4   Vol = 1134 × (.75/12) = $\boxed{70.87 \text{ Ft}^3}$ pine

Roof – $\sqrt{27^2 + 14^2}$ = 30.5   A = 0.5 × 43 × 30.5 = 665.75

6 halves = 3934.5 Ft² copper × 15 lb/Ft² × $\frac{1}{1000}$ ≈ $\boxed{60 \text{ kips}}$ cu

Vol = 3934.5 × (.75/12) = $\boxed{246 \text{ Ft}^3}$ pine

Totals.

Pine Ft³	Iron Ft³	Copper kip	Found^ kip	Snow kip	Live kip
490	28	440	628.5	880	838
493		60			
94.5					
474.3					
2443					
70.9					
246					
1833					
493					
Σ 6550 Ft³	28 Ft³	500 k	628.5 k	880 k	838 k

γ_{pine} = 36.83 γ_{Iron} = 491.56 lb/Ft³

Pine – 6550 Ft³ × 36.83 lb/Ft³ × $\frac{1}{1000}$ = 241.2 kips

Iron – 28 × 491.56 × $\frac{1}{1000}$ = 13.7 kips

		kips
Sum of	Pine –	241.2
	Iron –	13.7
	Copper –	500
	Found^ –	628.5
	Snow –	880
	Live –	838

Dead = 2264 kips

Live = 838 kips

LRFD = 1.2D + 1.6L

Total load = 1.2(2264) + 1.6(838) = 4058 kips

Total load per Foot = 4058 / 419 = $\boxed{9.7 \text{ kips/Foot}}$

l

Vesic Bearing Capacity of Soil Calculations

$$q_{ult} = c N_c S_c d_c + \sigma_{zD} N_q S_q d_q + \tfrac{1}{2} B \gamma N_\gamma S_\gamma d_\gamma$$

<u>Assume:</u>

$c = 0$ b/c of sand

$\gamma = 105\ \mathrm{lb/ft^3}$ averaged from pg. 99 of <u>Geotechnical Engineering</u>, Coduto

$\phi = 34^\circ$ from Borehole Shear Test Graph F.S. = 2.5 from pg. 191 <u>Foundation Design</u>, Coduto

<u>Given</u>

$B = 2$ ft $k = \tan^{-1}(4/2) = 1.107$

$D = 4$ ft

<u>Calculations</u>

$$q_{ult} = 0 N_c S_c d_c + 4(\gamma)29.4(1)(1.373) + \tfrac{1}{2}2\gamma(41.1)(1)(1)$$

$$= 0 + 16953.804 + 4326$$

$$= 21279.804\ \mathrm{lb/ft^2}$$

$$= 21.279804\ \mathrm{k/ft^2}$$

$$q_a = \frac{q_{ult}}{F.S}$$

$$= \frac{21.279804}{2.5}$$

$$= 8.5119216\ \mathrm{k/ft^2}$$

The allowable bearing capacity of this soil is $8.5\ \mathrm{k/ft^2}$ based upon a Factor of saftey equalling 2.5.

27

D.IV

BEARING CAPACITY OF SHALLOW FOUNDATIONS												Unit conversion	1000
Terzaghi and Vesic Methods													
												Gamma w =	62.4
Date												phi (radians)	0.593412
Identification		Breeding Barn											
												Terzaghi Computations	
Input					Results							a theta =	4.011409
	Units of Measurement						Terzaghi		Vesic			Nc =	52.63745
		E	SI or E				Bearing Capacity					Nq =	36.50441
						q ult =	19489.0868	lb/ft^2	20264.84	lb/ft^2		N gamma =	39.59272
	Foundation Information					q a =	7795.63471	lb/ft^2	8105.937	lb/ft^2		gamma' =	105
	Shape	CO	SQ, CI, CO, or RE									coefficient #1 =	1
	B =	2	ft			Allowable Wall Load						coefficient #3 =	0.5
	L =		ft			P/b =	15.5912694	k/ft	16.21187	k/ft		sigma zD' =	420
	D =	4	ft										
	Soil Information											Vesic Computation	
	c =	0	lb/ft^2									Nc =	42.16373
	phi =	34	deg									sc =	1
	gamma =	105	lb/ft^3									dc =	1.442859
	Dw =	50	ft									Nq =	29.43979
												sq =	1
	Factor of Safety											dq =	1.290215
	F =	2.5										N gamma =	41.0638
												s gamma =	1
Copyright 2000 by Donald P. Coduto												d gamma =	1
												B/L =	0
												k =	1.107149
												W sub f	0

28

Appendix F: Repair Mortar

The timber-framed Breeding Barn is supported on an uncoursed ashlar stone foundation of red Monkton quartzite and fieldstone in a Portland-lime-sand mortar (constructed in 1890). A 2006 condition assessment revealed that the stonework on east, west, and south facades of the main block and annex was in fair condition. Deterioration conditions included cracked and spalling mortar, mortar losses particularly below roof valleys, and damage by farm machinery. On the north façade of the main block, the stonework could not be evaluated: on the northeast (east of the main entrance) above-grade masonry had been replaced with a reinforced concrete grade beam; on the northwest, masonry was covered by a reinforced concrete counterwall, which was removed above grade and replaced with a new stone masonry stemwall as part of this project.

In addition to constructing the new stemwall, repair strategies for the Breeding Barn foundations included spot repointing all sides of the building and resetting displaced stones. Formulating a mortar for the work was based on a number of factors including the:

- original constituents
- nature and condition of the existing masonry
- environmental conditions to which the mortar is exposed
- the performance of mortars in test walls and other repointing projects at Shelburne Farms, and
- technical research results

In 2008, US Heritage Group characterized a sample of the mortar; the report follows in this appendix. Based on chemical analysis and petrographic examination, they determined the mortar to be made of Portland cement, hydrated lime, and sand in volumetric proportions of 1.0 : 2.2 : 9.4 respectively. The aggregate is a finely graded natural sand consisting of quartz, feldspar, granite, siltone, pyroxene, basalt and ironstone. In addition to the US Heritage analysis, the project masons carried out simple acid digestion tests to visually evaluate overall aggregate gradation, angularity and appearance, and used the results to select the sand for the repair mortar.

While the historic mortar had performed relatively well over the last century, and though, in many cases, it is technically and aesthetically appropriate to carry out repairs using a mortar matching the existing or original material, the decision was made not to replicate the historic mortar. Instead a mortar made of natural hydraulic lime and local sand was used. This decision was based on recent experience at Shelburne Farms, and on the experiences of others working with Portland cement-gauged lime mortars in cold climates.

Historically, the gauged mortars used at Shelburne Farms have not always given long periods of service; the mortar used in the courtyard wall of the Farm Barn, constructed the year before the Breeding Barn, showed efflorescence-staining over much of the wall surface in early photographs and today requires extensive repair and rebuilding. The cement-gauged mortars used in reconstructing a portion of that wall, and in the construction of a test wall,[1] both in 2007, were prone to the same kinds of behavior, with extensive calcite efflorescence on the stone surfaces appearing during the first winter. In contrast, a test wall constructed onsite with hydraulic lime-based mortar is performing well after exposure to three winters. This result is consistent with the experience of the masons who worked on the repointing of the Breeding Barn; they have found that cement-gauged mortars are less predictable in performance than the NHL:sand repair mortar that was used.

Secondly, research on lime mortars conducted by English Heritage found that the strength and durability of Portland cement-gauged mortars are reduced unless the volume of cement in the mix is at least 0.5 cement to 1.0 lime. Lime-based mortars with smaller proportions of cement were also more susceptible to salt damage, and so may be especially prone to frost damage as well.[2] In the Breeding Barn, the option of increasing the volume of Portland cement in the mortar to increase its strength and durability was not considered due to other unacceptable properties imparted by the cement such as excessive hardness, low modulus of elasticity and low water vapor permeability relative to the original materials. St. Astier NHL 3.5 (a moderately hydraulic lime) has low compressive strength, does not require gauging, and is known to be durable in cold climates. Volumetric proportions of the repair mortar mix is 1:3 (NHL:local sand); the aggregate is 'mortar sand' from Hinesburg Sand and Gravel in Hinesburg,Vermont.

The Breeding Barn was pointed in 2008 and construction of the northwest stemwall was completed in 2010. The stonework is performing very well: there is no efflorescence, the joints are solid without cracks or losses, and the mortar is well bonded to the stone.

---

[1] In 2007 two test walls were constructed just south of the barn, one using a 1:3 natural hydraulic lime [St. Astier 3.5] to sand mortar, and the other, a 1:1:6, Portland cement to hydrated lime [Type S] to sand mortar. After 3 1/2 years exposure to weather, the hydraulic lime mortar is performing well and has clean joints without cracks or losses. The cement-gauged mortar, on the other hand, left extensive calcite efflorescence on the stone surface during the first winter following its construction.

[2] Teutonico, J.M, Ashall, G; Garrod, E., Yates, T.A. 1999. A comparative study of hydraulic lime-based mortars, in International RILEM Workshop on Historic Mortars; Characteristics and Tests, Paisley, Scotland, 12th-14th May 1999. pp. 339-350

Teutonico, J.M. 1996. 'The Smeaton project, in A future for the past; a joint conference of English Heritage and the Cathedral Architects Association, 25-26 March 1994,pp. 3-29.

Teutonico, JM, McCaig, I, Burns, C, and Ashurst, J. 1993. The Smeaton project: factors affecting the properties of lime-based mortars, in APT Bulletin, Vol. 25, No. 3-4, pp. 32-49.

Project: USHG #08044

August 11, 2008

Douglas Porter
University of Vermont
341 Votey Hall, 33 Colchester Avenue
Butlington, VT 05405
Phone:802-324-7528

EVALUATION OF MORTAR COMPOSITION – ASTM C1324
BREEDING BARN AT SHELBURNE FARMS
1611 Harbor Road, Shelburne, VT 05482

1.0 INTRODUCTION

We are pleased to present the results of our laboratory testing of sample of mortar removed from the Breeding Barn at 1611 Harbor Road, Shelburne, Vermont.
We understand that this section of the building was originally constructed in 1890 and is currently undergoing renovations to its exterior.

The following report summarizes the methods of testing and the results herein on the sample provided for this examination.

2.0 METHODOLOGY

The sample was analyzed according to chemical procedures and petrographic examination methods of ASTM C1324, "*Standard Test Method for Examination and Analysis of Hardened Masonry Mortars*".

3.0 RESULTS

3.1 PETROGRAPHIC EXAMINATION

Paste
The mortar has a light tan color, due to the fine aggregate (sand) content. The paste consists of hydrated portland cement and hydrated lime, and has a light gray – to – white color. The entire paste is carbonated. The paste has soft hardness with poor paste-aggregate bond and marginal firmness. The contains a low number of pockets of original hydrated lime measuring up to 1 mm in size. The pockets may be result of using slaked lime (lime putty). Brick fragments are not present on mortar surfaces. The degree of hydration is highly advanced. The aggregate volume appears to be high and the paste volume appears to be low.

Aggregate
The fine aggregate is a very finely grated natural sand with a 0.6 mm maximum grain size and a modal grain size (most frequently occurring) of 0.24 mm. The sand consists of quartz, feldspar, granite, siltone, pyroxene, basalt and ironstone. The aggregate is in a chemically stable condition. The grading appears to finer than the natural sand grading specified in ASTM C144.

Air Content
The mortar is not air-entrained. The mortar has a total entrapped air content of 3.3%.

3.2 CHEMICAL ANALYSIS

The mortar sample was chemically analyzed for portland cement content according to the soluble silica method in ASTM C1324, "Standard Test Method for Examination and Analysis of Hardened Masonry Mortars".

The Portland cement was assumed to contain 63.5% calcium oxide (CaO) and 21.0% silicon dioxide (SiO_2). The hydrated lime present in this sample was estimated to contain 43% calcium oxide (CaO) and 29% magnesium oxide (MgO)

The densities (loose volume basis) of the mortar ingredients were assumed to be those listed in ASTM C270. Eighty lbs. of oven-dry sand was assumed to be equal to one cubic foot of damp loose sand. The slaked lime putty was estimated to contain 50% hydrated lime (calcium hydroxide) and 50% water, with a loose volume density of 80 lbs. per cubic foot.

The results of the chemical analysis indicate that this is most similar to a Type O mortar mixture of portland cement and hydrated lime.

U.S. Heritage Group, Inc., 3516 North Kostner Ave., Chicago, IL 60641 Phone: 773-286-2100 Fax: 773-286-1852

The volumetric proportions of the sample (determined according to ASTM C270) are as follows:

Volumetric Proportions:

Portland cement:	1.0 parts
Hydrated Lime: (Slaked Lime Putty)	2.2 parts (2.2 parts)
Natural Sand:	9.4 parts

3.3 PROPOSED REPLACEMENT MIX

In light of these findings and the intended use of the replacement material, U.S. Heritage Group recommends specifying a replication mortar formulation consisting of 1 part portland cement, 2 parts slaked lime putty and 8 parts sand.

This mix design would fall under the classification "Type O" in ASTM C270 Proportion Specification. The portland cement must meet ASTM C150; the non-hydrated lime is required to meet ASTM C207; and the sand should match the original sand as closely as possible in terms of color, size and shape.

Adjustments to the gradation curve should be considered when a mortar joint width exceeds ½ inch. The rationale in recommending this mortar is based upon the nature of the repairs and considering the National Park Service guidelines (set-forth below) that recommend that a replacement mortar be formulated to be softer in compressive strength than that of the original to protect the adjacent masonry units.

*** "In creating a repointing mortar that is compatible with the masonry units, the objective is to achieve one that matches the historic mortar as closely as possible, so that the new material can coexist with the old in a sympathetic, supportive and, if necessary, sacrificial capacity."*

"The new mortar must be as vapor permeable and as soft or softer (measured in compressive strength) than the historic mortar. (Softness or hardness is not necessarily an indication of permeability; old, hard lime mortars can still retain high permeability.)"

** Preservation Briefs #2 Repointing Mortar Joints in Historic Masonry Buildings, Technical Preservation Services, National Park Service, 1998.

4.4 JOBSITE MOCK-UP SAMPLE

The replacement mortar sample should be field-tested through a jobsite mock-up. The mock-up sample should be installed by a qualified craftsperson who understands the curing and application details of Type N mortars. Once the mock-up sample is installed, appropriate precautions should be taken to ensure that the mortar is protected from wind, sun, rain and frost to enable slow curing (i.e. carbonation) to take place.

*The sample should be allowed to **cure in the wall for a minimum of seven but preferably fourteen days** before final color match is approved.*

The sand gradation charts illustrating the sand isolated from your samples were sent by overnight mail last week. We look forward to providing you with a custom, ready-to-use, historically correct mortar for your project. When inquiring about this match please use the project number USHG#08044.

Respectfully,

U.S. Heritage Group, Inc.	Nelson Testing Laboratories
Tom Glab Laboratory Manager	Michael F. Pistilli Chemist, Petrographer

Table 1. Chemical Analysis of Mortar Samples

Constituent	Percent by Mass %
	Sample #1 USHG # 528-1"white aggregate"
Silica - SolubleSiO_2	2.02
Calcium Oxide - CaO	10.25
Brucite – $Mg(OH)_2$	4.00
Insoluble Residue	76.56
Magnesium Oxide – MgO	2.67
Loss on Ignition	
At 0-110ºC	0.0
At 110-550ºC	2.11
AT 550-1000ºC	3.65
Calculated Constituents	
Portland Cement	9.60
Hydrated Lime	9.19
Fine Aggregate	76.56
Volumetric Proportions (according to ASTM C270) **– Loose Volume Ratios**	
Portland Cement : Hydrated Lime : Sand	1.0 : 2.2 : 9.4
Mortar Type	Type O

Appendix G: Iron Characterization and Testing

To address overstresses in the truss elements, investigators characterized samples of the period iron in the Breeding Barn to establish reasonable design values for structural ironwork. Each truss has wrought iron tension members, struts, and raised bottom chord. Purlins and valley members are trussed with wrought iron ties across one (king-rod) or two (queen-rod) pipe struts. Visual inspection confirmed that forge welds in tension elements were generally in good condition, and that heel connections were intact and in good condition.

Small portions of two samples, one from a strut and the other from a lower chord tension element, were collected for metallographic characterization. Oriented oxide inclusions exhibited in the microstructure of the samples indicate that lower chord and web members are constructed of wrought iron. Chemical analysis indicates a low-carbon material; the closest SAE-AISI alloy designation for the material sampled is 1005, with maximum carbon content of 0.06%. Both samples have relatively high levels of phosphorus, which typically results in increased strength and hardness and decreased ductility and notch impact toughness in the as-rolled condition. Iron used for rolled tension elements included copper in a proportion exceeding 0.20%, which contributes corrosion resistance and also adversely affects hot-working operations like forge welding.

Architect Robertson originally called for a single truss to support inboard dormer framing for major dormer pairs located at the east and west ends of the ring. Sometime subsequent to original construction, but early in the history of the building, a second truss was added at each inboard dormer location to support dormer framing not carried on the end walls. Installation of the additional trusses required shortening of the truss rods for each of the purlins intersected by the new trusses.

Samples were obtained from the cut-off ends of four of these shortened truss rods for conducting strength-in-tension tests. Test procedures for determining strength-in-tension are provided in ASTM A 370-05 (Test Methods and Definitions for Mechanical Testing of Steel Products). Test coupons were prepared from each of the samples. Three of the samples were tested in the Materials Testing Lab in the School of Engineering at the University of Vermont; the fourth sample was sent to an independent lab. Test results indicated average yield strength of about 33.2 ksi, an average maximum tensile strength of 47.3 ksi and an average MOE of 30.2 Mpsi, which compare relatively well to design values in period code and design manuals.

This appendix includes 1) a letter report from the consulting metallurgist; 2) lab results resulting from the metallographic characterization of two samples collected from a strut and a tension member by Aston Metallurgical Services; 3) lab results for strength-in-tension tests conducted by the New Hampshire Materials Testing Lab and the Materials Testing Lab, School of Engineering, University of Vermont.

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14 April 2006

Douglas Porter
Graduate Program in Historic Preservation
The University of Vermont
Wheeler House
133 South Prospect
Burlington, VT 05405

Ref: Metallurgical Analysis of Iron Samples
Breeding Barn
Shelburne Farms

Doug,

Attached please find the results of the metallurgical analysis conducted on the two samples of iron from the Shelburne Farms Breeding Barn. For the analysis, the samples were identified as SFBB-1 and SFBB-2. SFBB-1 is from the strut of original truss number 15.0/H.2. SFBB-2 was taken from one of the 1" diameter rods.

The oriented oxide inclusions exhibited in the microstructure of both samples indicates that both are wrought.

The very low carbon content of both samples, as determined by chemical analysis, identifies them as low-carbon steel. By definition, low-carbon steels contain up to 0.30% carbon. The samples from the Breeding Barn have carbon contents of 0.04% and 0.01%, respectively. This puts them at the low end of the low-carbon range. The closest SAE-AISI alloy designation is 1005. 1005 carbon steel has a maximum carbon content of 0.06%, maximum manganese content of 0.35%, maximum phosphorus of 0.040%, and maximum sulfur content of 0.050%.

Carbon, which has a major effect on steel properties, is the principal hardening element in all steel. Tensile strength in the as-rolled condition increases as carbon content increases. Ductility and weldability decrease with increasing carbon.

Both samples are within the specified limits for manganese and sulfur for 1005 carbon steel. Manganese contributes to strength and hardness , but to lesser degree than carbon. Increases in manganese content decreases ductility and weldability, but to a lesser degree than does carbon. Increased sulfur content lowers transverse ductility and notch impact toughness but has only a slight effect on longitudinal mechanical properties. Weldability decreases with increasing sulfur content.

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Interestingly, these samples contain relatively high levels of phosphorus. Increasing phosphorus content increases strength and hardness and decreases ductility and notch impact toughness in the as-rolled condition.

Silicon is one of the principal deoxidizers used in steelmaking; therefore, the amount of silicon present is related to the type of steel. Silicon is somewhat less effective than manganese in increasing as-rolled strength and hardness.

Copper in appreciable amounts is detrimental to hot-working operations. Copper adversely affects forge welding, but it does not seriously affect arc or oxyacetylene welding. Copper is, however, beneficial to atmospheric corrosion resistance when present in amounts exceeding 0.20%, as is the case with SFBB-2 taken from the 1" diameter rod.

Regards,

T. Scott Kreilick

T. Scott Kreilick
President & CEO
Kreilick Conservation, LLC

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Metallurgical Problem Solving & Consulting
Failure Analyses & Corrosion Investigations
Certifications & A2LA Accredited Testing

ASTON Metallurgical Services Co., Inc.
200 Larkin Drive Unit A
Wheeling IL 60090
847/353-8100

Laboratory Report

MR. SCOTT KREILICK
KREILICK CONSERVATION
519 TOLL ROAD
ORELAND, PA 19075-2343

March 15, 2006

Lab 603783

Subject:

Two samples identified as Shelburne Farms Breeding Barn received on March 6, 2006 were submitted for metallurgical evaluations as directed.

Chemical Testing:

Test	SFBB-1	SFBB-2
Carbon	0.04%	0.01%
Manganese	0.18	0.06
Phosphorus	0.175	0.143
Sulfur	0.029	0.026
Silicon	0.19	0.48
Copper	<0.05	0.21
Nickel	0.05	<0.05
Chromium	<0.05	<0.05
Molybdenum	<0.05	<0.05

Page 1 of 7

Alan Stone
President
ASTON

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Metallography:

The samples were cross-sectioned, mounted, ground, polished and examined under a metallurgical microscope at magnifications of up to 1,000X in both the unetched and etched conditions.

The examinations revealed both samples to contain numerous oriented oxide inclusions. Sample SFBB-2 contained more spheroidal inclusions as compared to SFBB-1. Etching revealed SFBB-1 to be ferritic with some pearlite whereas SFBB-2 appeared to be ferritic without observable pearlite. See the accompanying photomicrographs.

Microhardness Testing:

KHN_{300} Approximations to HRB		
Reading	SFBB-1	SFBB-2
1	91	82
2	92	85
3	92	79

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FIGURE 1

50X Unetched

SFBB-1
Large oriented oxide inclusions.

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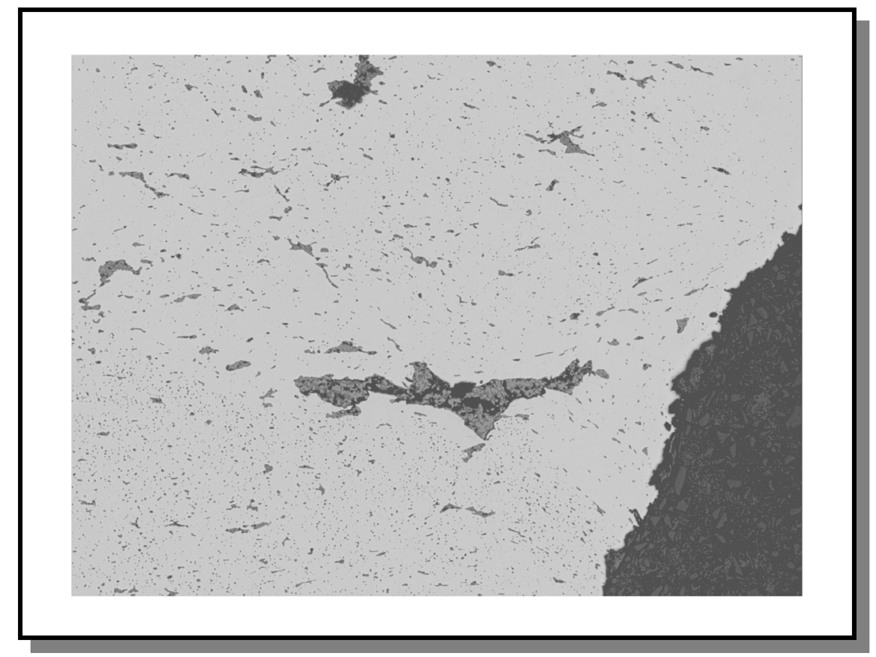
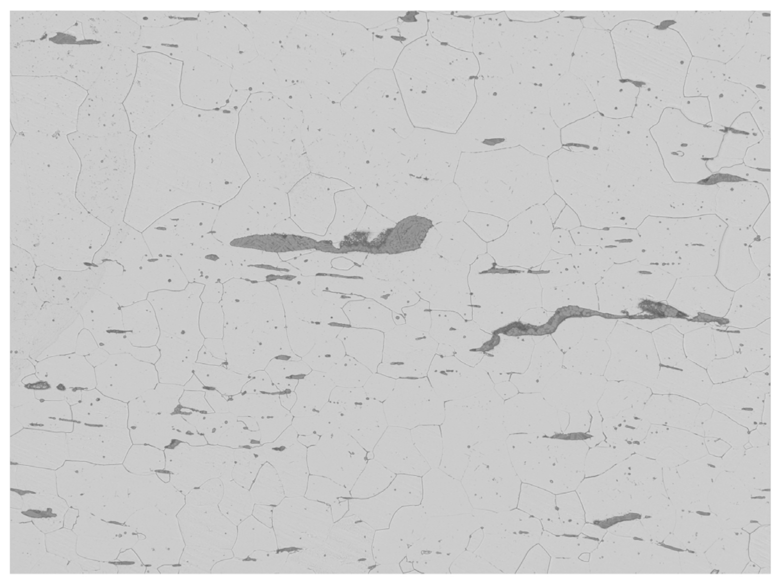
FIGURE 2



50X Unetched

SFBB-2
In addition to the large oriented oxide inclusions observed in SFBB-1, there are numerous fine spheroidal oxide inclusions.

FIGURE 3



100X Nital

SFBB-1
Etching reveals the ferritic matrix.

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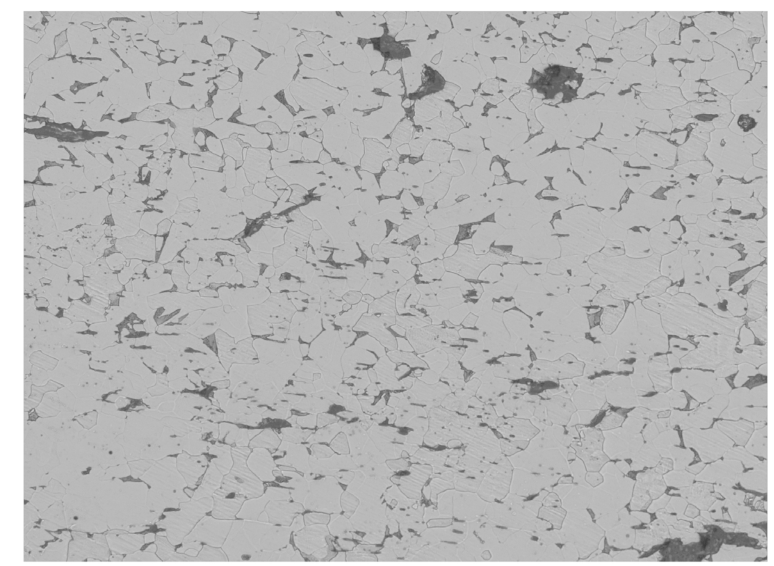
Certifications & A2LA Accredited Testing

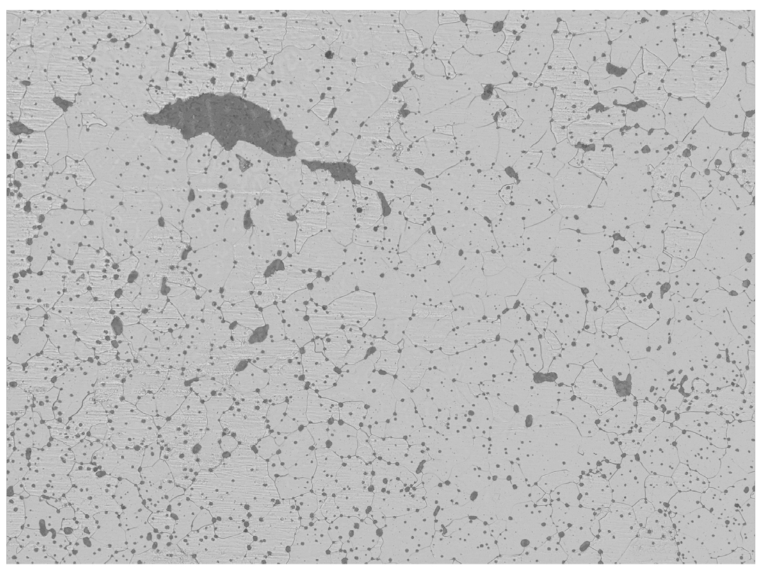
FIGURE 4



100X Nital

SFBB-1
The grain size is finer at this location. There is also some pearlite mixed in with the ferrite.

FIGURE 5



100X Nital

SFBB-2
Etching reveals the ferritic matrix. Note the oxide inclusions.

The following ASTM test procedures was used:
E1019, E415, E1621, E3, E407, E384 and E140.
A2LA certificates 277-01/02. Some procedures
performed by our associates.

Lab Results: Strength in Tension

New Hampshire MATERIALS LABORATORY, INC.
Your Problem Solving Partner

Test Report

September 7, 2007

Mr. Doug Porter
301 Votey Hall, Room 341
33 Colchester Avenue
Burlington, VT 05405

NHML File No 24269
P.O. No
Phone: 802-324-7528
douglas.porter@uvm.edu

Overview
Samples Received: (1) 0.502" diameter wrought iron tensile bar
Analysis Requested: Tensile test per ASTM E8
Sample Disposition: Discard 30 days from date of report

Analysis Results

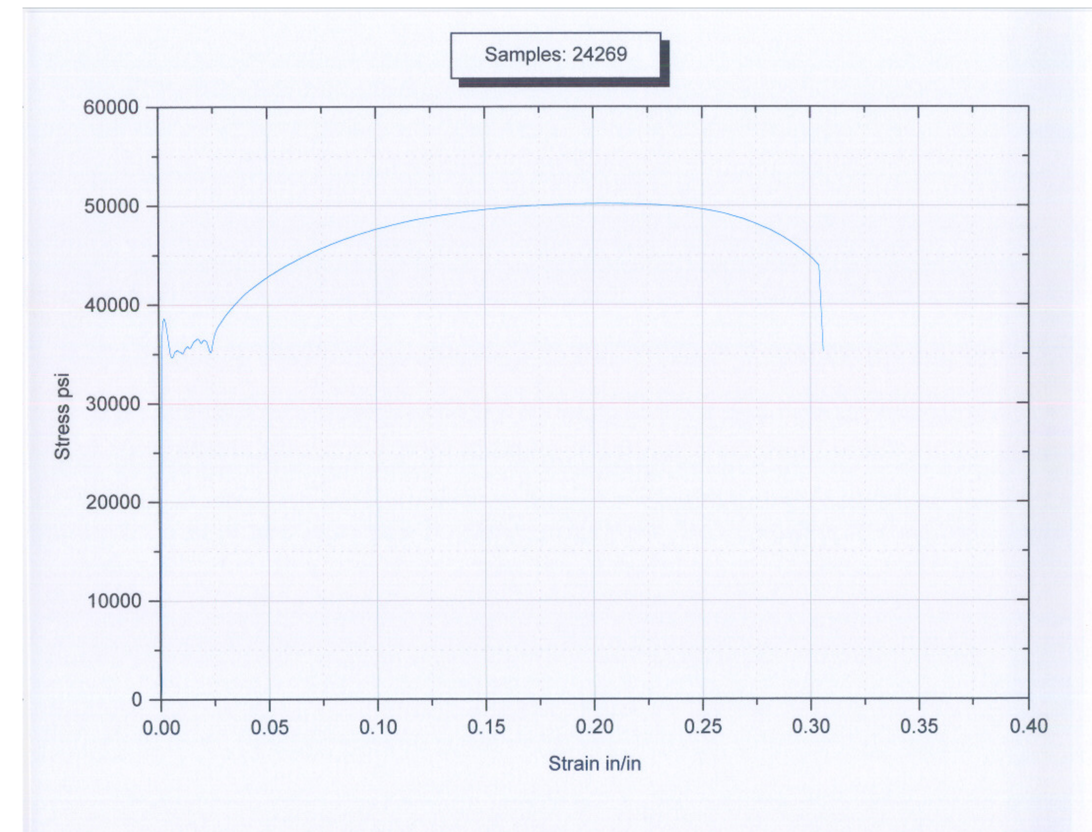
Sample ID	0.2% Yield (ksi)	Tensile (ksi)	% Elongation (in 4D)	Modulus (Mpsi)
A	37.7	50.2	25	29.3

A stress-strain plot is also attached.

Submitted by:

Timothy M. Kenney
Director of Laboratory Services

22 Interstate Drive
Somersworth, NH 03878-1209
800-334-5432 603-692-4110
603-692-4008 fax
lab@nhml.com www.nhml.com



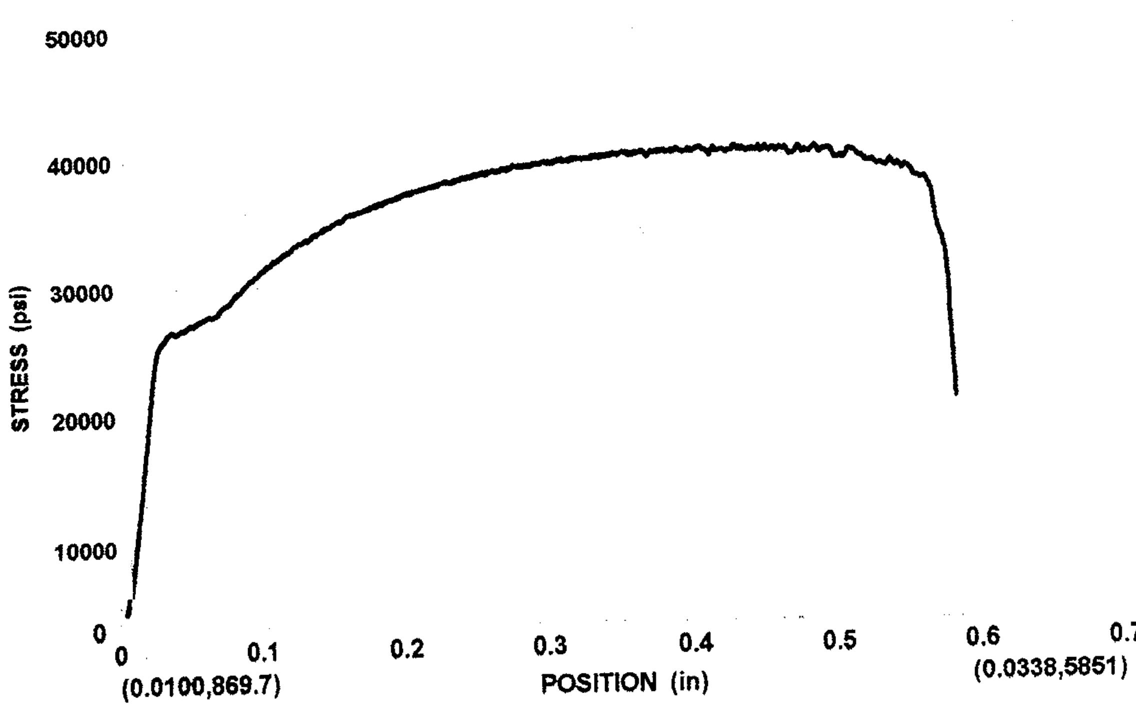
Materials Lab
School of Engineering, University of Vermont

Specimen: 1- H.2/15.0

Material:	Wrought iron
Geometry:	Round
Diameter:	0.5050 in
Gage Length:	2.0000 in
Area:	0.2003 sq in

Date:01/03/99

Peak Load:	8218 LB
Peak Stress:	41029 psi
Modulus of elasticity:	2020689
Yield @ 20% offset:	25966 psi



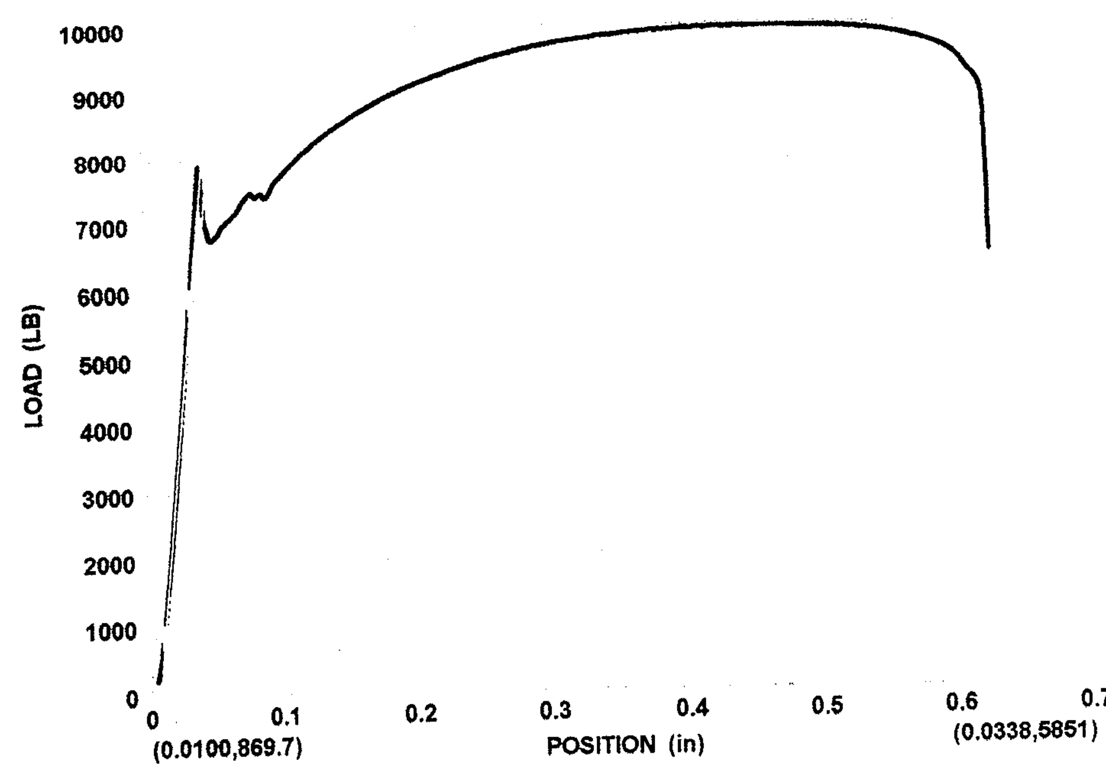
Materials Lab
School of Engineering, University of Vermont

Specimen: 2- G.2/15.0

Material:	Wrought iron
Geometry:	Round
Diameter:	0.5050 in
Gage Length:	2.0000 in
Area:	0.2003 sq in

Date:01/03/99

Peak Load:	9967 LB
Peak Stress:	49760 psi
Modulus of elasticity:	2774562
Yield @ 20% offset:	35735 psi

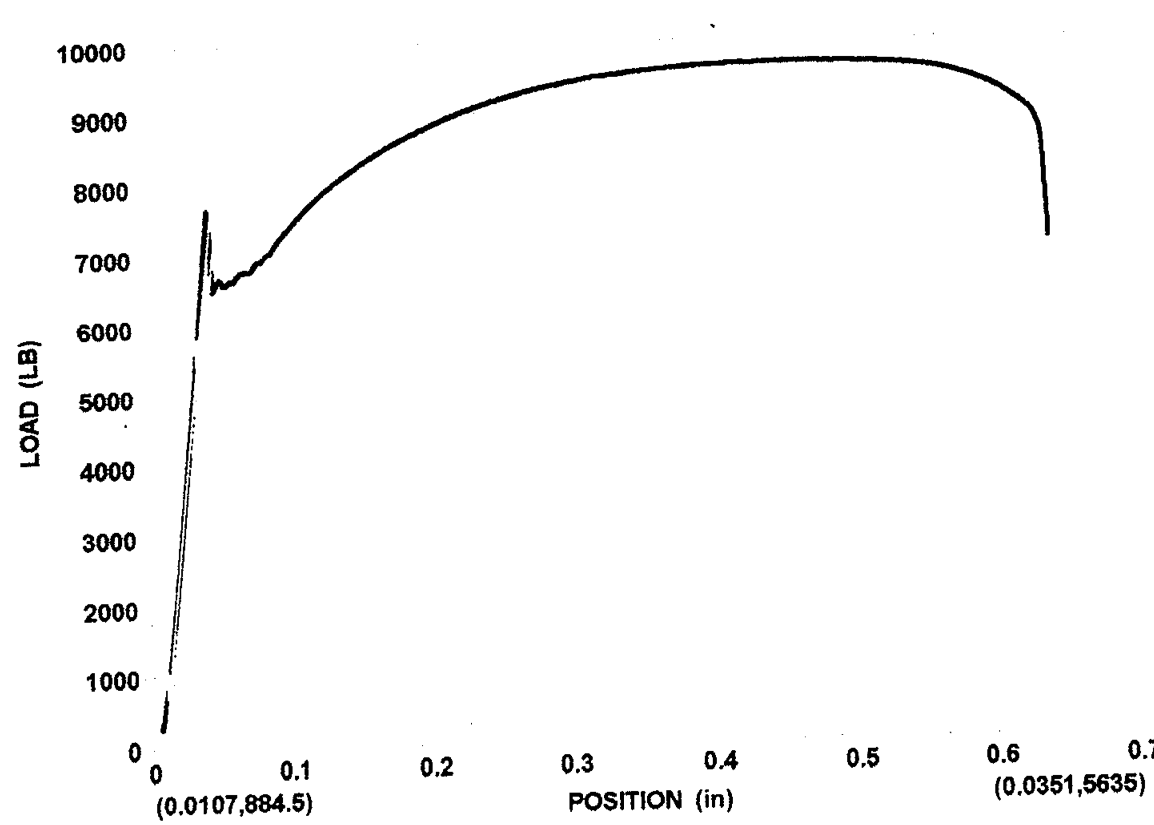


Materials Lab
School of Engineering, University of Vermont

Specimen: 3- D.8/15.0	
Material:	Wrought iron
Geometry:	Round
Diameter:	0.5050 in
Gage Length:	2.0000 in
Area:	0.2003 sq in

Date:01/03/99

Peak Load:	9681 LB
Peak Stress:	48332 psi
Modulus of elasticity:	2580265
Yield @ 20% offset:	33181 psi



APPENDIX H: Wood Assessment

Initial condition assessment of the Breeding Barn indicated deterioration of valley rafters at the lantern and at dormer pairs at each end of the barn was one of the chief problems to be addressed in designing and implementing repairs. Steel channels bolted to either side of each valley member prevented direct examination of those surfaces, so that investigators were uncertain as to the magnitude of the problem.

To quantify the extent of deterioration, a systematic survey was conducted using a resistance drill (IML-RESI System). Resistance drilling is a quasi-nondestructive technique for quantifying the loss of material in wood. It is considered quasi-nondestructive because, although a small needle penetrates the wood (approximately 1/8-inch diameter), virtually no wood fiber is removed. The resistance drill measures the relative density of wood; as the needle is pushed through the wood, the amount of torque encountered by the motor is recorded on a resistograph strip.

This technology is extremely useful for quantifying the amount of deterioration within timbers and for identifying patterns of deterioration. The drill records evidence of intermittent small voids associated with insect damage, and can also indicate if moisture has come from above and deteriorated the top of a timber, creating a "V" shaped pattern of deterioration through the cross section or if moisture has wicked in from the exposed ends of the timber, leaving a shell of sound wood around a decayed core. This technology is especially suited to determining internal problems in timbers that do not show obvious signs of deterioration, such as surface decay.

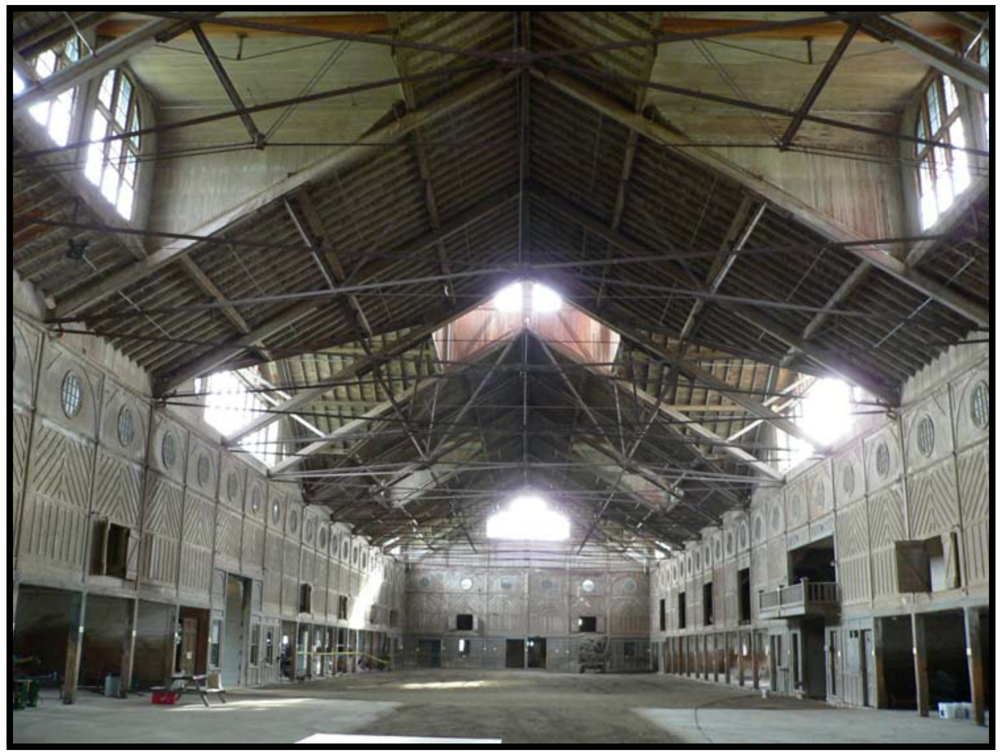
In the Breeding Barn, valley timbers were drilled in the radial and transverse directions along their length in order to characterize decay patterns and quantify section loss. Where substantial voids were encountered, additional drilling was done to locate void boundaries to the extent possible (the steel channels limited access). Of the twelve timbers examined, six had varying degrees of loss on the upper surface due to decay; these losses appeared as decay channels (called 'channelizing') located in the upper half of the timber section, probably the result of water leaking through the roof and finding its way into drying checks. Of these, damage to two of the members (11.0 South, 17.0 South) was thought to be severe over a substantial portion of their length. Two of the members (4.5 North, 14.5 South) were severely deteriorated at rafter heels, where they bear on timber plates in the walls surrounding the riding ring.

To facilitate repair decisions a data summary was prepared for each valley member, and inspection results were summarized graphically, and in narrative form. The graphic was color-coded and color codes follow a simple green-to-red transition for the valley members in "good" to "poor" condition. Members or portions of members that have inspection data showing no visible or detected damage were color-coded green. Areas in yellow exhibited minor deterioration, with channelizing and loss of section of 2 inches or less. Areas in orange indicate deeper channelizing or a local failure. Areas colored red indicated severe deterioration and were the highest priority for the design team in developing repair options. Foloowing the data summaries for each valley member, resistance drill results for each of the drill sites are organized in a table.

This appendix includes the report from Anthony & Associates on the wood investigation of the Breeding Barn.

Report:

WOOD INVESTIGATION OF THE BREEDING BARN, SHELBURNE FARMS, SHELBURNE, VERMONT



Submitted to:

Shelburne Farms
1611 Harbor Road
Shelburne, VT 05482

Submitted by:

Anthony & Associates, Inc.
P. O. Box 271400
Fort Collins, CO 80527-1400

February 2006

Wood Investigation of the Breeding Barn, Shelburne Farms, Shelburne, Vermont

BACKGROUND AND PURPOSE

The 1300-acre historic Shelburne Farms site, founded by W. Seward and Lila Webb, is situated on the shoreline of Lake Champlain about seven miles south of Burlington, Vermont. The National Historic Landmark property is famous both for its landscape, originally planned by Frederick Law Olmstead, Sr., and for its historic Queen Anne and Shingle style buildings designed by Robert Henderson Robertson.

The Breeding Barn, one of the four major buildings that define the estate, was completed in 1891. It was thought to have the largest unsupported interior space in the U.S. for several decades after its completion. The Breeding Barn is a two-story rectangular building, 418 feet long by 107 feet wide. The interior space of 375 feet by 85 feet was used originally for a riding ring and interior stalls. The interior space is roofed by a central lantern whose base is 55 feet above the floor. On either side of the lantern is a complex hipped roof, with numerous dormers.

The purpose of the investigation was to determine the extent of deterioration in the timbers that make up the valley rafters of the roof in the Breeding Barn (the Barn). The findings will enable cost-effective repair and replacement decisions based on known deficiencies in the timbers constituting the valley rafters.

SCOPE OF WORK

Prior to this investigation, the extent of deterioration in the valley rafters at the Barn was not known. Based on discussions with Mr. Douglas Porter of the University of Vermont and Mr. David Fischetti of DCF Engineering, Inc., the wood investigation focused on resistance drilling, but included a combination of visual observations and probing to identify and quantify deterioration of the timbers in the 12 valley rafters.

The scope of work included:

- Determining the type and quantifying the extent of deterioration in the valley rafters through resistance drilling readings, augmented by visual observations and probing.

- Determining the likely causes of the deterioration for the purpose of establishing effective remedial treatments or repairs and long-term maintenance needs.
- Analyzing visual observations, moisture content measurements and resistance drilling data, then summarizing the findings in a report.

An on-site meeting was conducted between Anthony & Associates, Inc. staff and Mr. Douglas Porter of the University of Vermont to establish the priorities for the investigation. Suspect locations and critical wood members were identified prior to the wood investigation by a team of timber framers. The team's findings allowed for focusing the wood investigation on suspected areas of deterioration.

FIELD PROCEDURES

A key concern for the long-term structural integrity of the Breeding Barn, shown in Figure 1, is the condition of the timbers that make up the valley rafters. The valley rafters support the large roof structure (Figure 2). There is visual evidence of wood decay and possible insect damage, resulting in deteriorated wood along the length of the valley rafters. The purpose of this investigation was to assess the integrity of the timber used in the valley rafters based on findings from a preliminary inspection by a team of timber framers. Supported by visual inspection and probing, resistance drilling was the primary means of quantifying the extent of deterioration. Each method is described below.

Figure 1. Breeding Barn viewed from the northeast.

Figure 2. Interior of the Breeding Barn.

Visual Inspection and Probing

Visual inspection of the wood allows for identifying components that are missing, broken or in an advanced state of deterioration. Missing components are those which have been removed or have fallen away, frequently due to extensive deterioration. If missing components were intended to provide structural support or protection from the elements (e.g. prevent moisture intrusion), their replacement may be essential to prevent long-term damage to the structure. Visual inspection allows for the detection of past or current moisture problems, as evidenced by moisture stains on the exposed surface of the wood. Further, visual inspection enables detection of external wood decay fungi or insect activity as determined by the presence of decay fruiting bodies, fungal growth, insect bore holes or wood substance removed by wood-destroying insects. Visual inspection provides a rapid means of identifying areas that may need further investigation.

Probing the wood with a sharp pick enables rapid detection of voids in the wood that may not be visible on the surface. Internal decay or insect damage is often masked by the lack of evidence on the exposed surface of the wood. For advanced decay, where large internal voids are present near the surface, probing allows for detection of potentially serious deterioration. Even for the early stage of decay, termed incipient decay, probing is beneficial. Probing can often reveal areas of incipient decay in timber, which have experienced sufficient deterioration due to decay fungi to allow for easy entry of a sharp probe although no void is yet present. Wood without incipient decay tends to offer more resistance to probing due to the higher density and more intact internal wood structure.

Quantification of Deterioration using Resistance Drilling

Resistance drilling is a quasi-nondestructive technique for determining the relative density of wood. Resistance drilling has, unfortunately, numerous connotations. To timber framers and others well versed in timber inspection, the term resistance drilling often refers to use of a hand-held drill with a 1/8-inch to 1/4-inch bit for penetrating a wood member. The operator "feels" the resistance encountered as the drill bit is pushed into the wood. To an experienced inspector, the color and consistency of the wood shavings that are extracted from the drill hole reveal information about the soundness of the wood. This technique does not allow for quantification of the extent of deterioration, particularly when the wood is decayed but may not include a large void.

In this investigation, resistance drilling refers to use of the IML-RESI System, manufactured by IML GmbH of Germany (Figure 3). Resistance drilling is well suited for determining the extent of internal problems in timbers which do not show obvious signs of deterioration. Any internal void due to insect damage or decay at the location drilled can be detected by determining the relative density of the wood. The relative density is printed on a strip of paper as a small diameter needle penetrates the wood, as seen in Figure 4 for (a) a solid section and (b) a void. The technique is very reliable for quantifying the extent of voids in the timbers. Resistance drilling was conducted on the timbers in the 12 valley rafters. Due to the ability to quantify the extent of deterioration, this technique is better suited than other inspection methods for determining internal problems in timber that does not show obvious signs of deterioration.

Figure 3. Use of the IML-RESI System to inspect a timber.

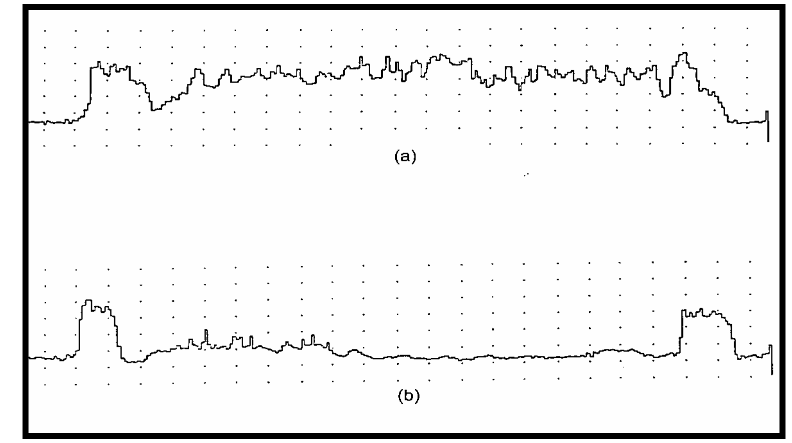


Figure 4. Results from resistance drilling showing (a) solid wood and (b) wood with internal deterioration.

Using these techniques, field work was conducted from October 19-21, 2005 by Ron Anthony, Wood Scientist, and Gretchen Lear, Field Assistant, from Anthony & Associates, Inc. Access to the valley rafters was made possible using a mechanical lift graciously provided by Shelburne Farms.

FINDINGS

Twelve valley rafters are used in the roof structure of the Barn. Four rafters are connected through bolted steel straps and connectors at the apex of the roof at both the east and west ends of the Barn (Figure 5). At the center of the Barn, four valley rafters extend from the interior walls to the corners of the base of the lantern. Some of the timbers making up the valley rafters are sandwiched between two steel channels, making visual inspection and probing with an awl difficult, particularly at the apex (Figure 6).

Figure 5. Apex of valley rafters, east end of the Barn.

Figure 6. Apex of valley rafters showing steel side channels, east end of the Barn.

The timbers that make up the valley rafters are generally in good to excellent condition. Each of the rafters was subjected to resistance drilling along its length to generate a schematic of the location and approximate extent of deterioration. Some of the rafters have deterioration on the upper face of the timber that penetrates to various depths, a condition called channelizing. Two of the valley rafters have severe deterioration of the heel where they bear on the interior wall.

Table 1 provides a general summary of the findings by rafter. Conditions identified in red should be priorities for engineering analysis and / or repair. The numbering of the valley rafters, shown in Figure 7, is based on the grid system used by other team members. Schematics of each valley rafter are color-coded to provide the reader with a visualization of the deterioration found. The choice of colors is subjective and is done only for the purpose of distinguishing the level of deterioration found. Areas colored green indicate no deterioration found. Areas in yellow exhibited minor channelizing (approximate depth of two inches or less) at the top of the rafter or minor deterioration elsewhere in the cross section. Orange areas indicate either local failure or deeper channelizing. Areas colored red on the schematics should be considered a priority for the structural engineer.

The schematics are the same as those used by other team members for their inspection and, therefore, have markings not relevant to this investigation (such as dimensions). Approximate resistance drilling test locations are marked on each schematic. The resistance drilling results for all of the tests are included in the appendix. Unless otherwise indicated, all resistance drilling measurements were taken by drilling vertically through the bottom face of the rafter.

Table 1. Summary of deterioration found on the Breeding Bar rafters.

Valley Rafter	General condition
1 North	Good, no deterioration found
1 South	Good, no deterioration found
4.5 North	Heel deteriorated, some crushing above truss 4
4.5 South	No deterioration but large splits and severe slope-of-grain
8 North	Minor channelizing
8 South	Minor channelizing
11 North	Good, no deterioration found
11 South	Channelizing and internal decay
14.5 North	Channelizing and failure at supplemental truss
14.5 South	Heel deteriorated and channelizing
17 North	Minor channelizing, end split at apex and deep check
17 South	Channelizing

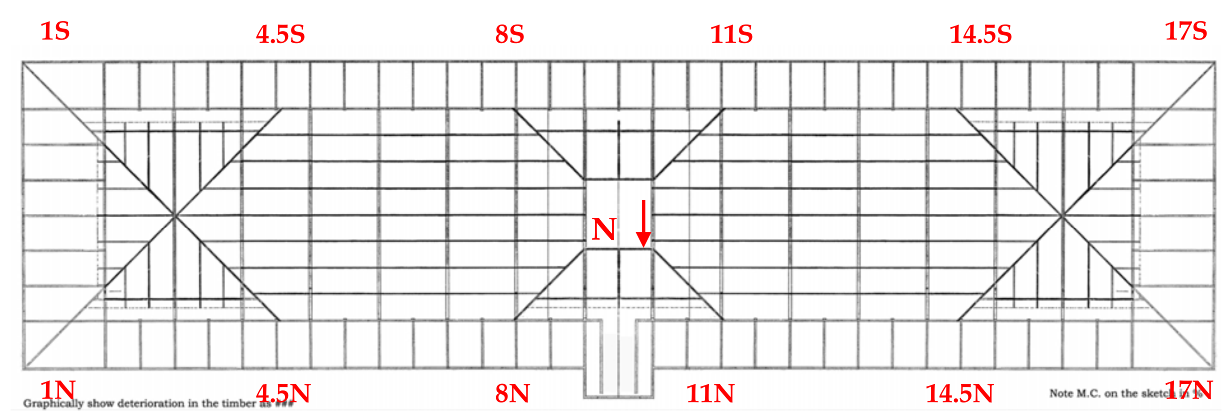


Figure 7. Plan view showing valley rafter designations.

Findings on Valley Rafter 1 North

This rafter, located at the east end of the Barn, is in good condition, as indicated by the green color along its length in Figure 8. The timber, seen from above in Figure 9, appears free of large checks, splits or slope of grain. The steel bar that appears along the center of the timber is the lower chord of the queen post truss illustrated on the bottom of Figure 8. The connector plate on the bottom of the queen post truss near the east wall has deformed lags, likely indicative of shifting loads over time (Figure 10).

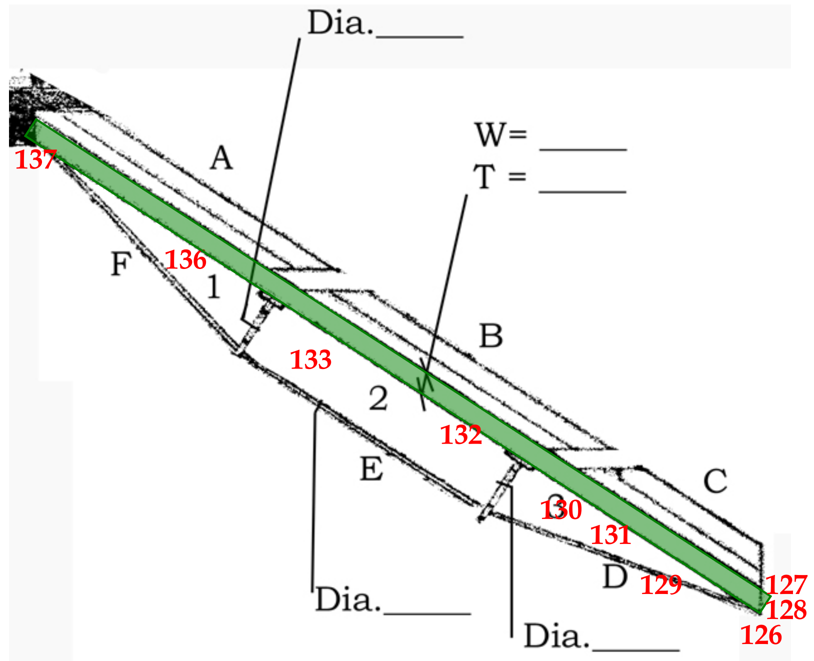


Figure 8. Inspection results from Valley Rafter 1 North.


Figure 9. Valley Rafter 1 North viewed from the apex.


Figure 10. Valley Rafter 1 North at east wall; note deformed lags.

Findings on Valley Rafter 1 South

This rafter is in good condition, as indicated by the green color along its length in Figure 11. The timber, seen from the apex in Figure 12, appears free of splits or slope of grain. The seasoning check shown in Figure 13 is typical for large cross-section timbers that dry in service and does not represent a failure in the timber.

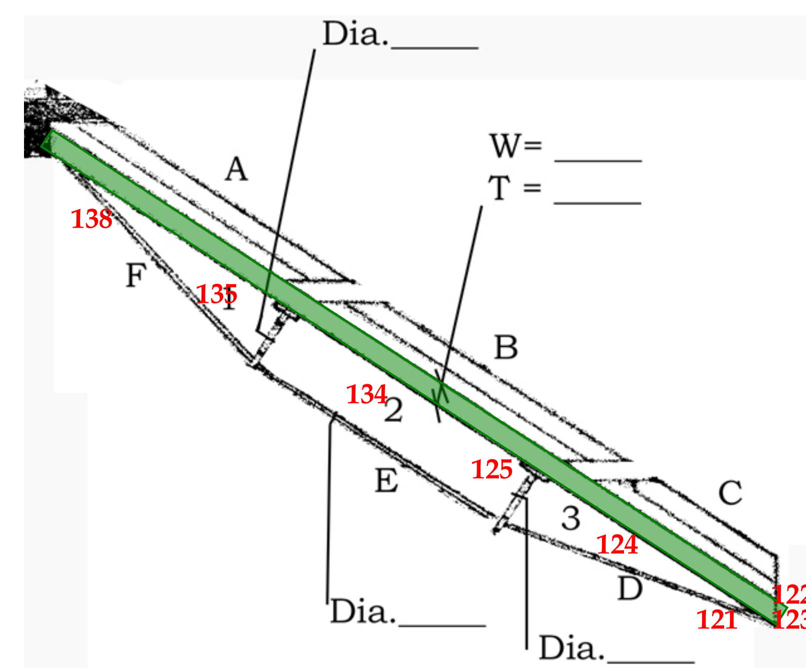


Figure 11. Inspection results from Valley Rafter 1 South.

Figure 12. Valley Rafter 1 South viewed from the apex.

Figure 13. Valley rafter 1 South, drying check on the south face.

Findings on Valley Rafter 4.5 North

Much of this rafter is in good condition (Figure 14). The timber, seen from the apex in Figure 15, appears free of splits or slope of grain. However, there is possible minor deterioration in the vicinity of Truss 4 (T4 on Figure 14) and some crushing of the timber has occurred. A more serious condition exists at the heel of the rafter, where it bears on the top plate of the north wall (Figure 16). The rafter is severely decayed and is secured to the top plate by four nails that are not in full contact with both the rafter and the top plate (Figure 17).

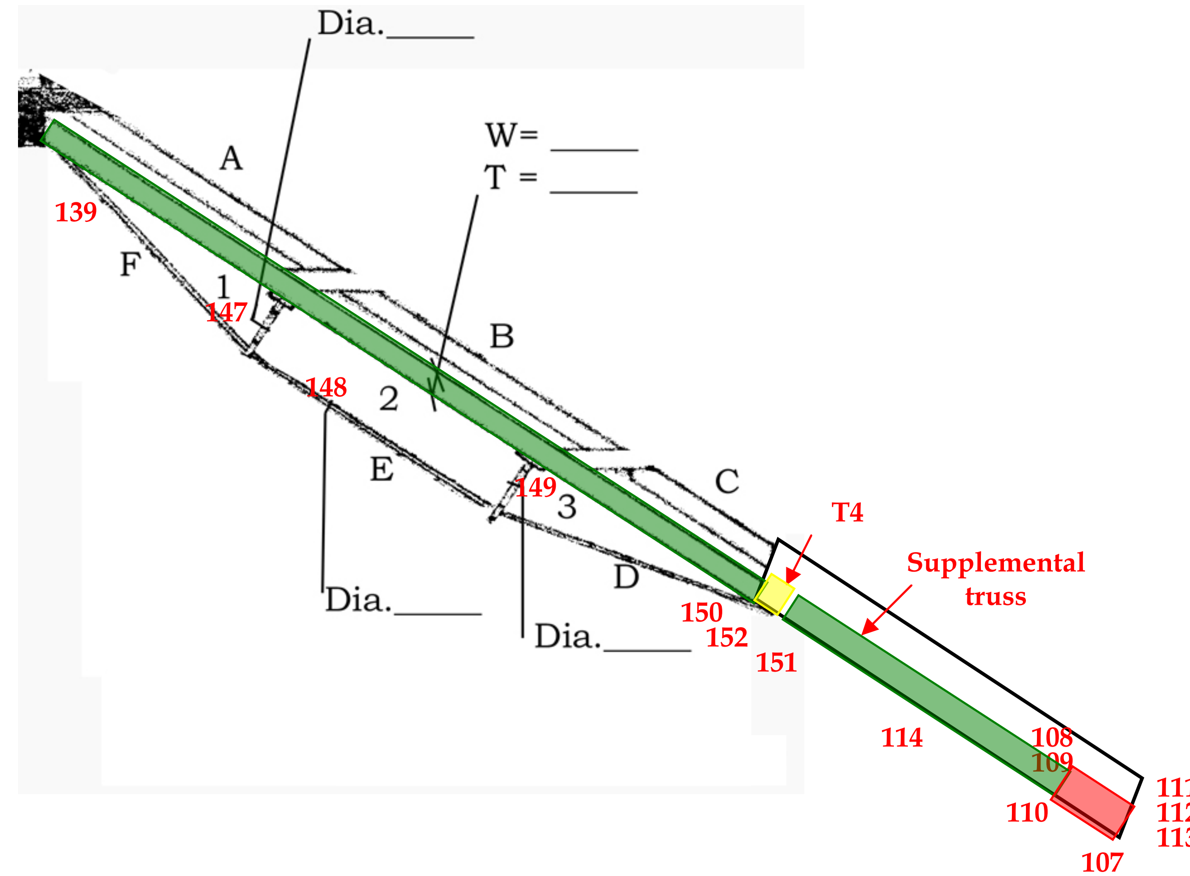


Figure 14. Inspection results from Valley Rafter 4.5 North.

Figure 15. Valley Rafter 4.5 North viewed from the apex.

Figure 16. Heel of Valley Rafter 4.5 North at the north wall showing decay in rafter and top plate.

Figure 17. Heel of Valley Rafter 4.5 North; note corroded nails not fully securing the rafter to the top plate.

Findings on Valley Rafter 4.5 South

This rafter is in good condition, as indicated by the green color along its length in the schematic shown in Figure 18. Although the general appearance is good (Figure 19), and no deterioration due to decay was found using resistance drilling, this timber has severe slope of grain as indicated by the seasoning check shown in Figure 20. Severe slope of grain will reduce the load-carrying capacity of the timber.

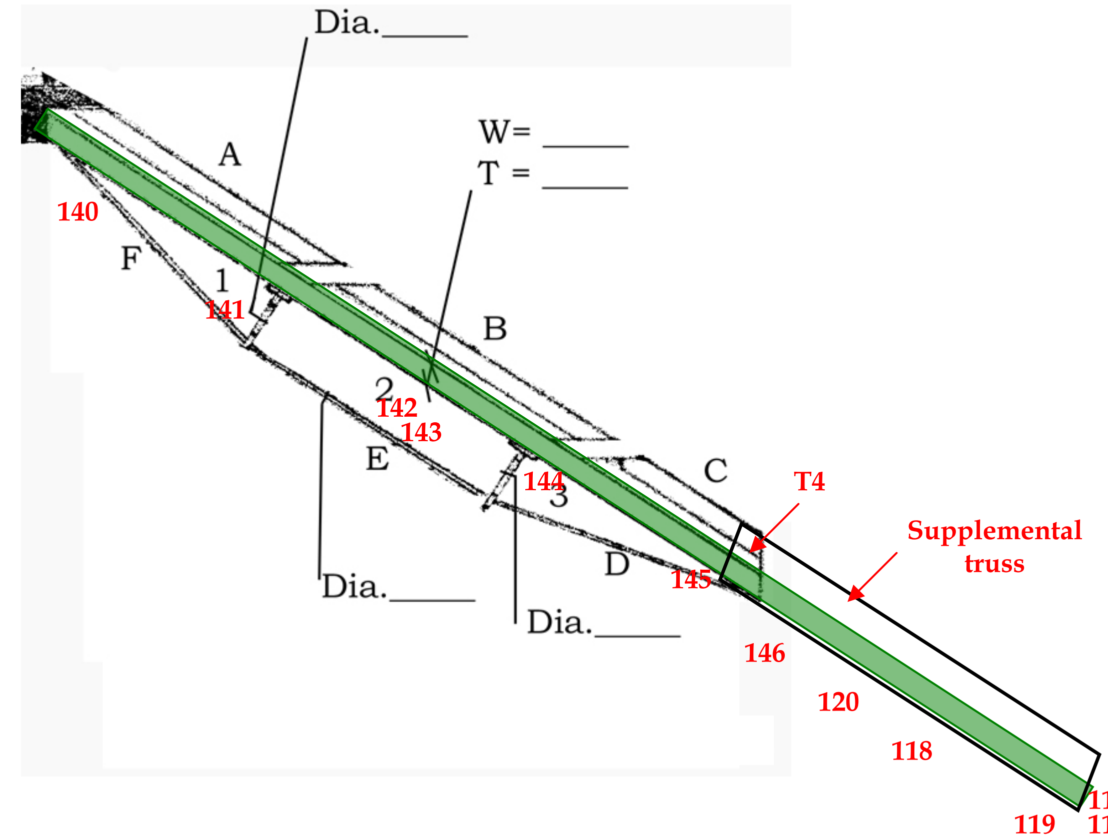


Figure 18. Inspection results from Valley Rafter 4.5 South.

Figure 19. Valley Rafter 4.5 South viewed from the apex.

Figure 20. Seasoning check indicating severe slope of grain (1.5″ in 10″) in Valley Rafter 4.5 South.

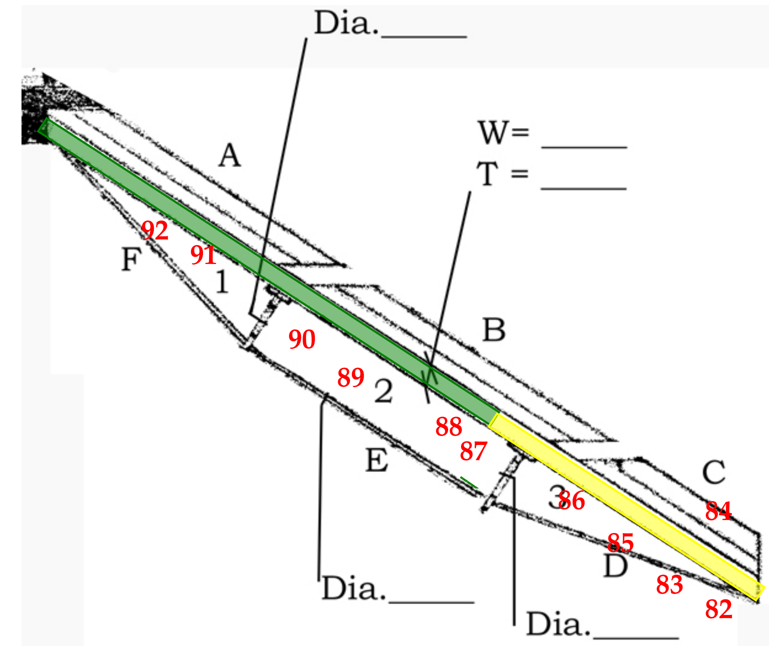


Figure 21. Inspection results from Valley Rafter 8 North

Figure 22. Valley Rafter 8 North viewed from mid-length.

Findings on Valley Rafter 8 North

As shown in Figure 21, this rafter has minor channelizing along the lower length of the rafter. Viewed from below, no deterioration is visible (Figure 22). Resistance drilling and probing revealed deteriorated wood on the upper surface of the timber from just above the lower queen post to the heel of the rafter. The deterioration, some of which can be seen in Figure 23, does not dramatically reduce the cross section of timber.

The likely cause of the decay is leaks in the roof that have since been repaired. So long as the moisture content of the timber is below 20 percent, the decay typically cannot be active. Proper maintenance of the roof covering is the key to keeping the timbers in the valley rafters dry. As with any of the deterioration found, the structural engineer should verify that the loss of cross section does not compromise the load-carrying capacity of the timber sufficiently to warrant reinforcement or replacement.

Figure 23. Valley Rafter 8 North at drilling location D84.

Figure 25. Valley Rafter 8 South viewed from apex.

Findings on Valley Rafter 8 South

As shown in Figure 24, this rafter also has minor channelizing along the lower length of the rafter. Viewed from below, no deterioration is visible (Figure 25). Resistance drilling and probing revealed deteriorated wood on the upper surface of the timber from just above the lower queen post to the heel of the rafter.

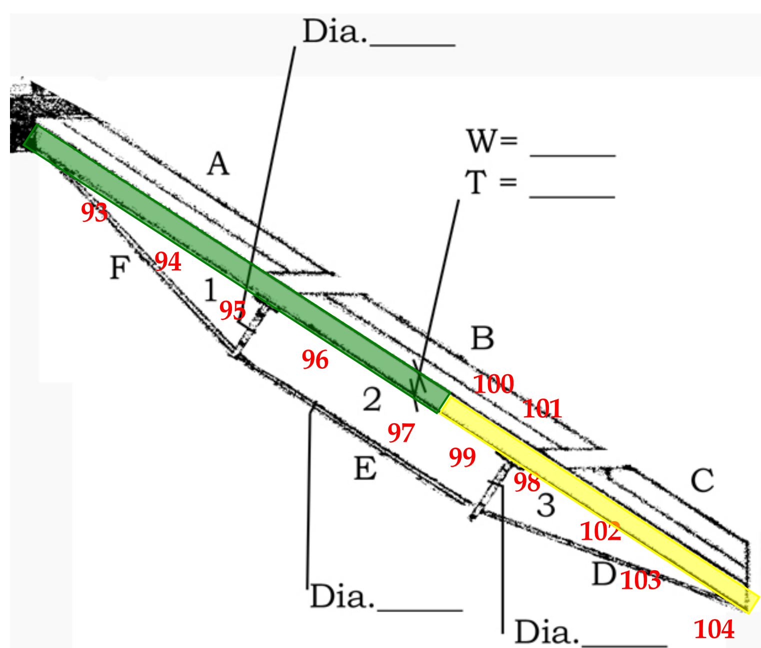


Figure 24. Inspection results from Valley Rafter 8 South.

Findings on Valley Rafter 11 North

This rafter is in good condition, as indicated by the green color along its length in Figure 26. The timber, seen in Figure 27, has seasoning checks but minor slope of grain. The seasoning checks visible in Figure 27 are typical for large cross-section timbers that dry in service and do not represent a failure in the timber.

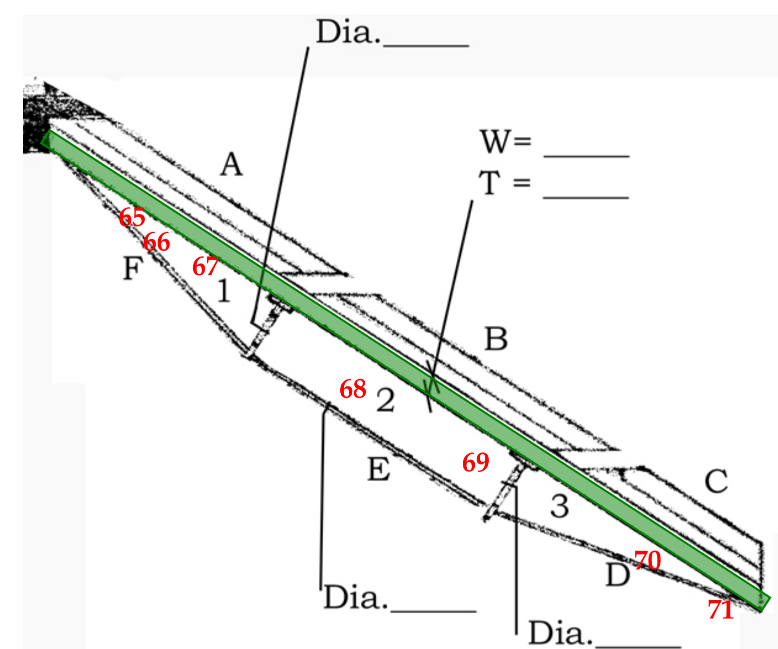


Figure 26. Inspection results from Valley Rafter 11 North.

Figure 27. Valley Rafter 11 North showing seasoning checks.

Findings on Valley Rafter 11 South

As shown in Figure 28, this rafter also has minor channelizing along the lower length of the rafter. Resistance drilling and probing revealed deteriorated wood on the upper surface of the timber from just above the lower queen post to the heel of the rafter. The area below the upper queen post has severe decay with significant loss of cross section (Figure 29). Additionally, the upper purlin is severely twisted where it meets the valley rafter (Figure 30).

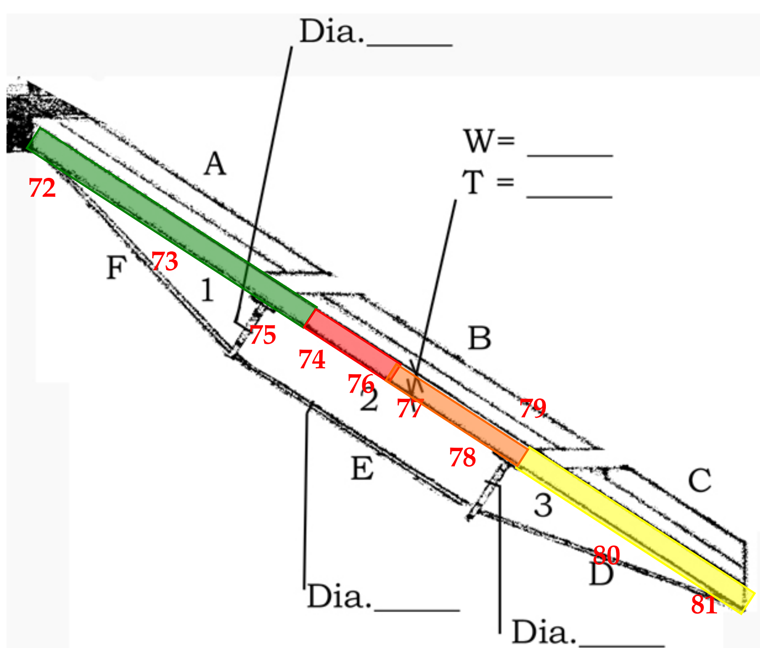


Figure 28. Inspection results from Valley Rafter 11 South.

Figure 29. Top of Valley Rafter 11 South near drilling D74

Figure 30. Upper purlin twisted at junction with Valley Rafter 11 South.

Findings on Valley Rafter 14.5 North

As shown in Figure 31, this rafter has minor channelizing along the lower length of the rafter. The upper length of the timber is in good condition (Figure 32); however, drilling near the apex (location D15) was inconclusive and it is possible that minor channelizing is present. Resistance drilling and probing revealed deteriorated wood on the upper surface of the timber from below the lower queen post to the heel of the rafter. There is a failure of the valley rafter at the supplemental truss (Figure 33). The heel of the rafter, where it bears on the top plate of the north wall, is decayed (Figure 34).

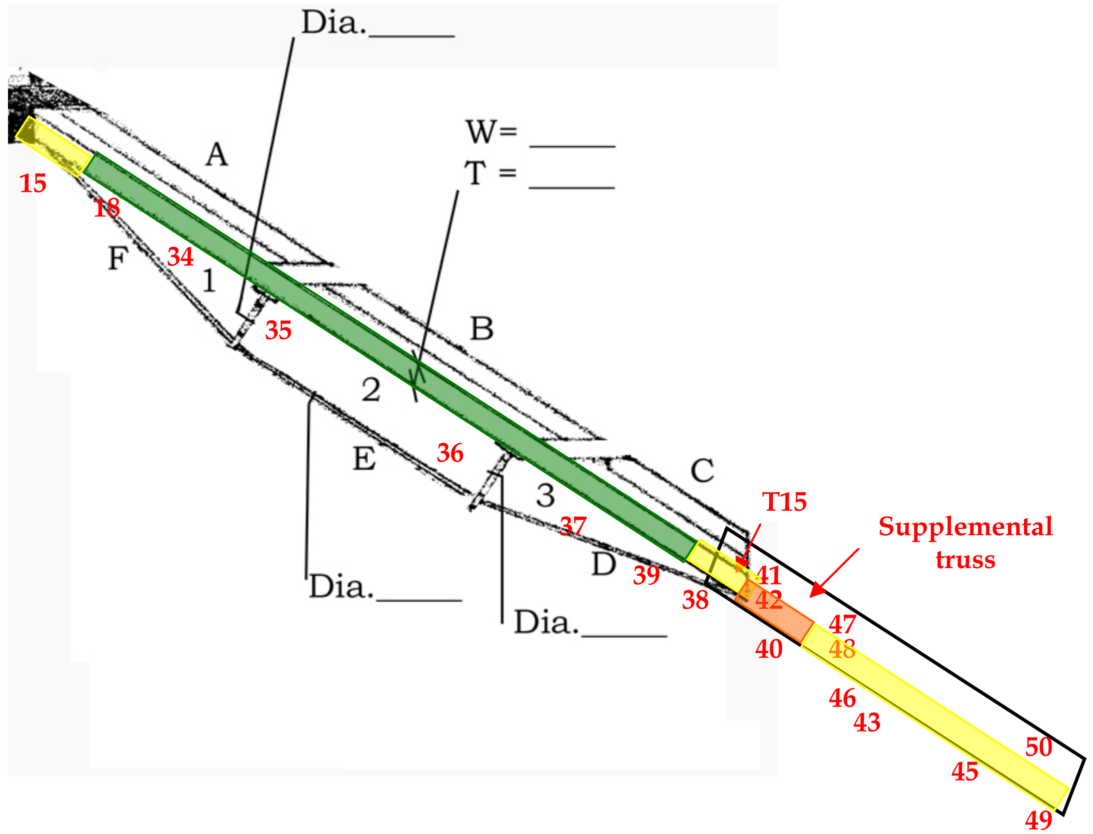


Figure 31. Inspection results from Valley Rafter 14.5 North

Figure 32. Valley Rafter 14.5 North from peak viewed from the apex.

Figure 33. Failure in Valley Rafter 14.5 North, east of supplemental truss.

Figure 34. Valley Rafter 14.5 North showing decay in the rafter and the top plate of the interior wall.

At a few locations on some of the valley rafters, transverse resistance drillings were conducted to establish the decay pattern that produced the channelizing. Figure 35 is a diagram of the drilling results from locations D40, D41 and D42 (which are shown on the schematic in Figure 35). The diagram is an approximation of the width and depth of the channel due to decay as indicated by the three drillings. Due to the loss of section in the timber, this is referred to as deep channelization.

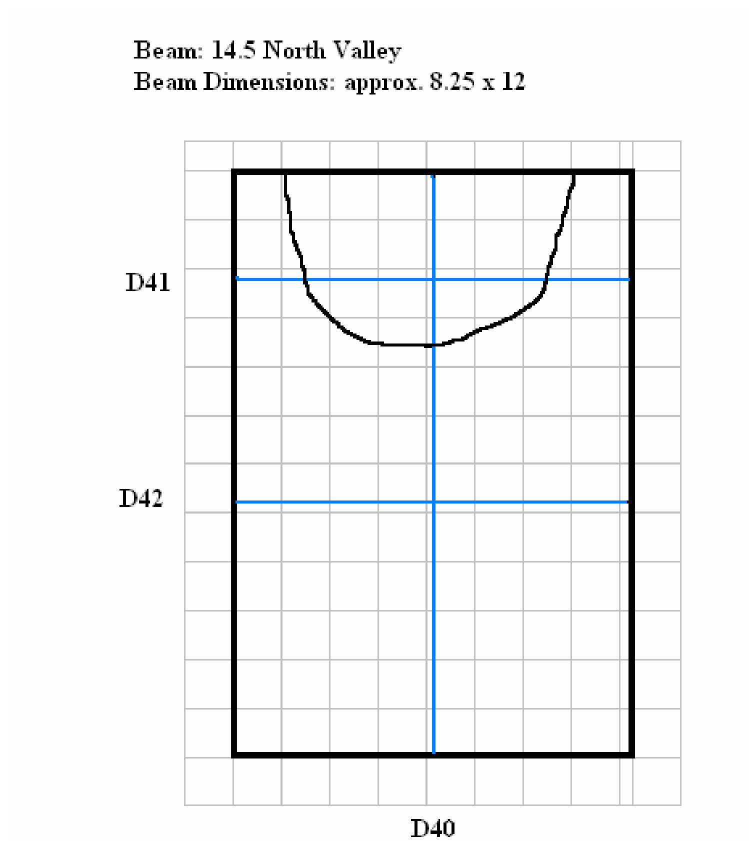


Figure 35. Diagram showing channelization pattern in Valley Rafter 14.5 North near the supplemental truss.

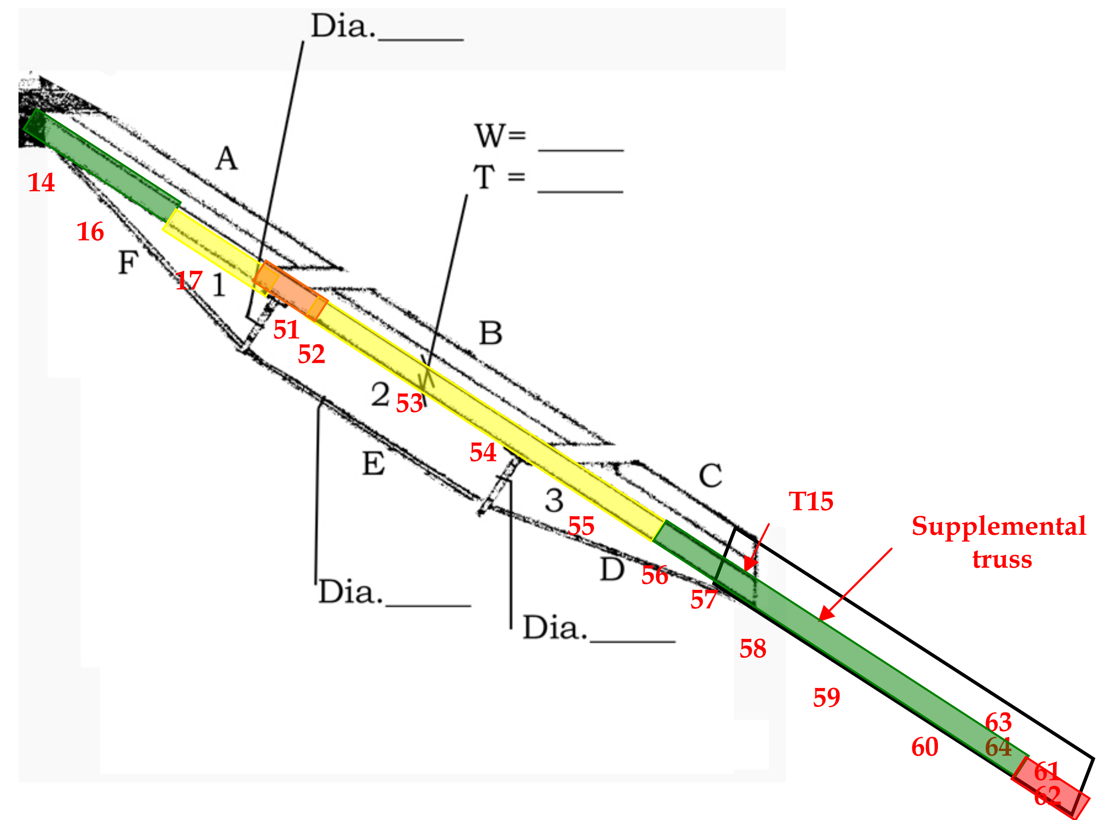


Figure 36. Inspection results from Valley Rafter 14.5 South.

Findings on Valley Rafter 14.5 South

This rafter has minor channelizing along the intermediate length of the rafter (Figure 36). The upper length of the timber above the upper queen post is in good condition (Figure 37). The heel of the rafter, where it bears on the top plate of the south wall, is decayed (Figures 38 and 39).

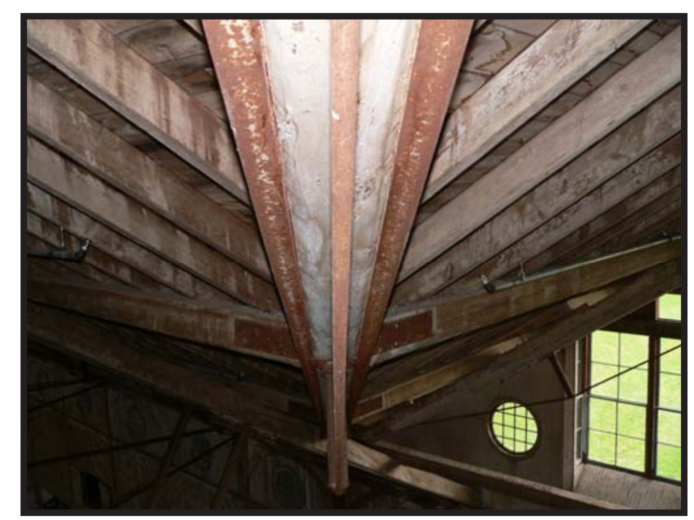


Figure 37. Valley Rafter 14.5 South viewed from the apex.

Figure 38. Valley Rafter 14.5 South where it meets interior wall.

Figure 39. Valley Rafter 14.5 South at the top plate of the interior wall.

Findings on Valley Rafter 17 North

This rafter has minor channelizing along the intermediate length of the rafter (Figure 40). A seasoning check is visible on the bottom face of the timber (Figures 41 and 42). The wood spacer between the valley rafter and the roof rafter above has deteriorated along much of its length. Between the queen posts the deterioration has extended into the top of the timber in the valley rafter, resulting in channelizing (Figure 43). The timber has an end split at the apex of the valley rafter (Figure 44). The split may have affected the capacity of this connection as a fracture is visible on the bottom of the timber just beyond the steel connector plate (Figure 45).

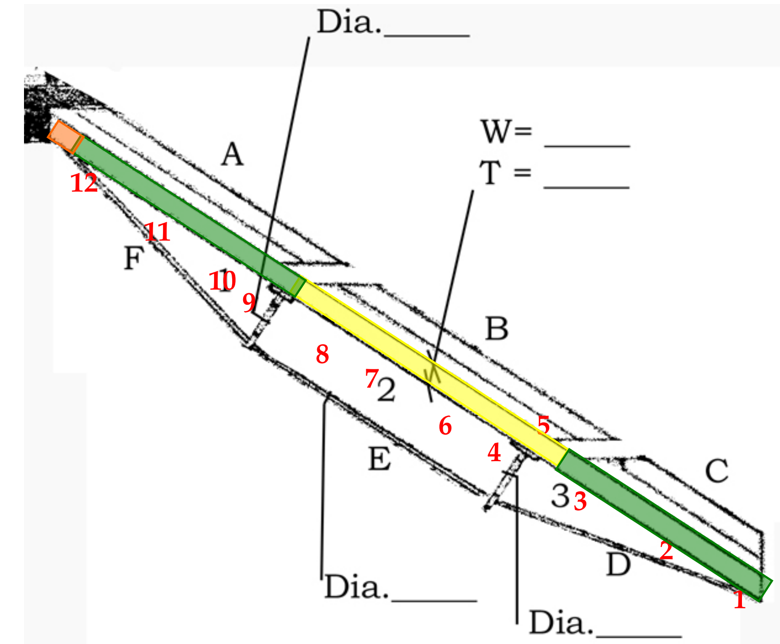


Figure 40. Inspection results from Valley Rafter 17 North.

Figure 41. Valley Rafter 17 North viewed from the apex.

Figure 42. Seasoning check in Valley Rafter 17 North.

Figure 43. Valley Rafter 17 North below drilling location D8.

Figure 44. Valley Rafter 17 North, above drilling location D12 showing end split.

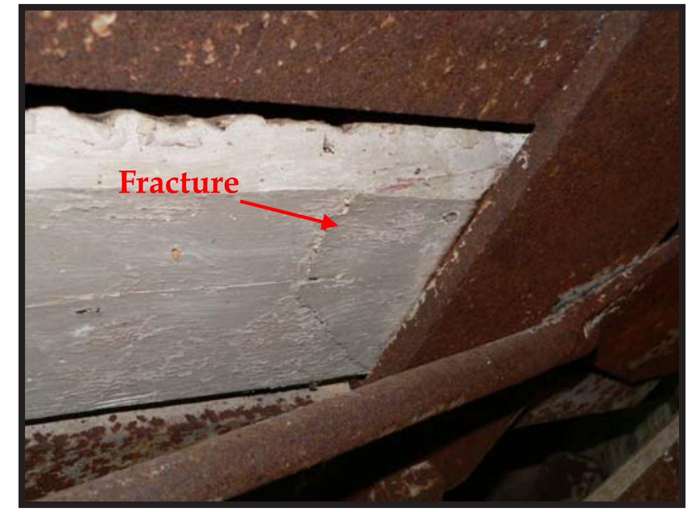


Figure 45. Fracture near the apex of Valley Rafter 17 North below the end split shown in Figure 44.

Findings on Valley Rafter 17 South

As shown in Figure 46, this rafter has a range of channelizing along the lower length of the rafter. Resistance drilling and probing revealed minor channelizing between the queen posts that progressively increased to the heel of the rafter. The upper length of the timber is in good condition (Figure 47). The wood spacer between the valley rafter and the roof rafter above has deteriorated along much of its length (Figure 48).

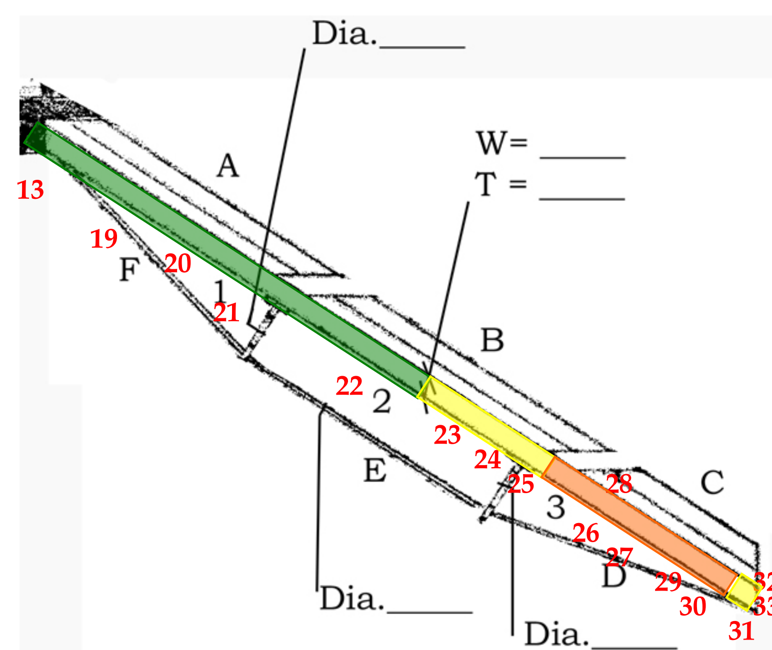


Figure 46. Inspection results from Valley Rafter 17 South.

Figure 47. Valley Rafter 17 South viewed from the apex.

Figure 48. Valley Rafter 17 South below drilling D23.

Figure 49 is a diagram of the drilling results from D31, D32 and D33 (which are shown on the schematic in Figure 46). The diagram is an approximation of the width and depth of the channel due to decay as indicated by the three drillings. Due to the depth of the decay pocket this is also referred to as deep channelization. Compare this pattern to that shown in Figure 35.

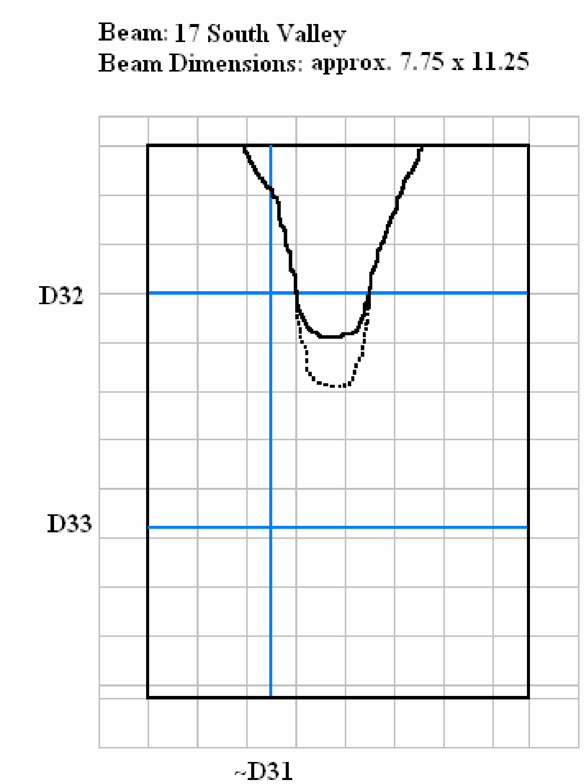


Figure 49. Diagram showing channelization pattern in Valley Rafter 17 South. Dotted line indicates likely pattern of the decay pocket, since only three drillings were conducted.

SUMMARY

Most of the timbers used in the valley rafters are in good condition. The majority of the deterioration found is due to decay fungi and most likely the result of previous roof leaks. Key findings include:

- The heels of Valley Rafters 4.5 North and 14.5 South are severely deteriorated.
- Valley Rafters 8 North, 8 South, 11 South, 14.5 South, 17 North and 17 South have various degrees of channelizing on the upper surface of the timber.
- Valley Rafter 17 North has an end split and fracture at the apex.
- Valley Rafter 4.5 South has severe slope of grain.
- Valley Rafter 4.5 North has crushing at Truss 4.
- Valley Rafter 14.5 North has failed at the supplemental truss.
- No deterioration was found in Valley Rafters 1 South, 1 North and 11 North.

APPENDICES

- **Resistance drilling results**
- **Use of digital radioscopy to examine connections and hidden conditions**

Resistance Drilling Results

Drilling Number	Valley Rafter	Location	Comment
D126	1 North	2.5" up from East wall truss	No void
D127	1 North	transverse 2.5" from top at East wall	No void
D128	1 North	transverse 8.5" from top at East wall	No void
D129	1 North	47" up from bottom plate	No void
D130	1 North	16" down from lower queen post plate	Notch at top
D131	1 North	32"down from lower queen post plate	No void
D132	1 North	29" up from lower queen post plate	No void
D133	1 North	21" down from upper queen post plate	No void
D136	1 North	40" up from upper queen post plate	No void
D137	1 North	9" down from top plate	No void
D121	1 South	7" up from bottom strap	No void
D122	1 South	transverse 2.5" from top, 3" from East wall	No void
D123	1 South	transverse 9.5" from top, 3" from East wall	No void
D124	1 South	47" down from lower queen post plate	No void
D125	1 South	11" up from lower queen post plate	No void
D134	1 South	30" down from upper queen post plate	No void
D135	1 South	41" up from upper queen post plate	No void
D138	1 South	22" down from top plate	No void
D107	4.5 North	vertical 2" from wall	5.5 " internal void
D108	4.5 North	transverse 8" from bottom, 14" from wall	No void
D109	4.5 North	transverse 6.5" from bottom, 14" from wall	No void
D110	4.5 North	34" up from heel	No void
D111	4.5 North	transverse 7" above heel, 8" from wall	No void
D112	4.5 North	transverse 5" above heel, 8" from wall	Void on north side, 5.5" solid wood
D113	4.5 North	transverse 3" above heel, 8" from wall	Void on north side, 4.5" solid wood
D114	4.5 North	74" up from heel	No void
D115	4.5 North	6" down from supplemental truss	No void
D139	4.5 North	7" down from top plate	No void
D147	4.5 North	13" up from upper queen post plate	No void
D148	4.5 North	73" up from lower queen post plate	No void
D149	4.5 North	5" down from lower queen post plate	No void, slope of grain = 0.9" in 10"
D150	4.5 North	5" up from bottom plate	No void
D151	4.5 North	12" down from truss 4	No void
D152	4.5 North	transverse at truss 4, 2" from top, 5" from joint	Possible minor deterioration @ interface
D116	4.5 South	transverse 7" above heel, 4" off South wall	No void
D117	4.5 South	transverse 2" above heel, 4" off South wall	No void
D118	4.5 South	70" up from heel	No void
D119	4.5 South	vertical 3" off wall	No void

Drilling Number	Valley Rafter	Location	Comment
D120	4.5 South	14" down from supplemental truss	No void
D140	4.5 South	22" down from top plate	No void
D141	4.5 South	14" up from upper queen post plate	No void
D142	4.5 South	46" down from upper queen post plate	No void
D143	4.5 South	56" down from upper queen post plate	No void
D144	4.5 South	6" down from lower queen post plate	No void
D145	4.5 South	20" up from lower plate	No void @ top, probable splice
D146	4.5 South	8" down from truss 4	No void
D82	8 North	2" up from bottom plate	Channelizing, 10" solid wood
D83	8 North	29" up from bottom plate	Channelizing, 10.5" solid wood
D84	8 North	transverse 1" from top, approx. 35" from lower plate	Channel profile, intermittent voids
D85	8 North	79" up from bottom plate	Minor channelizing, 11" solid wood
D86	8 North	approx. 14" down from lower purlin	Channelizing, 11.5" solid wood
D87	8 North	approx. 6" up from lower purlin	Channelizing, 9.75" solid wood
D88	8 North	approx. 48" up from lower purlin	No void
D89	8 North	approx. 48" down from upper purlin	No void
D90	8 North	approx. 3" down from upper purlin	No void
D91	8 North	approx. 38" down from top plate	No void
D92	8 North	approx. 7" down from top plate	No void
D93	8 South	approx. 6" down from top plate	No void
D94	8 South	approx. 50" down from top plate	No void
D95	8 South	12" up from upper purlin	No void
D96	8 South	29" down from upper purlin	No void - roof rafter above decayed
D97	8 South	approx. 65" up from lower purlin	No void
D98	8 South	12" up from lower purlin	Void @ top, 9.5" solid wood
D99	8 South	30" up from lower purlin	Minor channelizing, 10.75" solid wood
D100	8 South	transverse 1" down from top above D99	Minor channelizing
D101	8 South	transverse 1" down from top above D98	Channelizing
D102	8 South	13" down from upper purlin	Minor channelizing, 10.5" solid wood
D103	8 South	63" up from lower strap	Minor channelizing, 11" solid wood
D104	8 South	4" up from lower strap	Void @ top, 10.5" solid wood
D105	8 South	transverse 1" from top and 27" from bottom strap	Channelizing
D106	8 South	31" up from bottom strap	Minor channelizing, 11" solid wood
D65	11 North	14.5" from top plate	Possible knot hole near bottom
D66	11 North	18" down from top plate	No void
D67	11 North	68" down from top plate	No void
D68	11 North	approx. 32" down from upper purlin	No void

Drilling Number	Valley Rafter	Location	Comment
D69	11 North	approx. 8" up from lower purlin	No void
D70	11 North	approx. 40" down from lower purlin	No void
D71	11 North	3" up from bottom strap	No void
D72	11 South	10" down from top plate	No void
D73	11 South	63" down from top plate	No void
D74	11 South	4" down from upper purlin	Void @ top, 7" solid wood
D75	11 South	at upper purlin	No void
D76	11 South	40" down from upper purlin	Void @ top, 7" solid wood
D77	11 South	77" up from lower purlin	Channelizing, 10" solid wood
D78	11 South	12" up from lower purlin	Channelizing, 9" solid wood
D79	11 South	transverse 1" from top, above D78	Void, 4.5" internal void
D80	11 South	90" up from bottom plate	Channelizing, 10" solid wood
D81	11 South	8" up from bottom plate	Channelizing, 9.5" solid wood
D15	14.5 North	4" down from top plate	Possible channelizing
D18	14.5 North	45" down from upper queen post plate	No void
D34	14.5 North	29" up from upper queen post plate	No void
D35	14.5 North	30" down from upper queen post plate	No void
D36	14.5 North	19" up from lower queen post plate	No void
D37	14.5 North	62" up from lower strap	No void
D38	14.5 North	Just west of Truss 15	Void @ top, 10" solid wood
D39	14.5 North	12" up from lower strap	No void
D40	14.5 North	15" down from Truss 15	Void @ top, 8.5" solid wood
D41	14.5 North	transverse above D40, 9.75" from bottom	5" internal void
D42	14.5 North	transverse above D40, 5.25" from bottom	No void
D43	14.5 North	18" down from supplemental truss	No void
D45	14.5 North	61" down from supplemental truss	minor void at top, 11” solid wood
D46	14.5 North	8" down from supplemental truss	decayed above supplemental truss, 2" internal void
D47	14.5 North	transverse 9.75" from bottom above D46	No void
D48	14.5 North	transverse 4.25" from bottom above D46	No void
D49	14.5 North	32" up from wall	probable insect damage
D50	14.5 North	transverse above D49, 7" from bottom, 1.25" above splice	probable insect damage
D14	14.5 South	4" below top plate	No void
D16	14.5 South	37" down from top plate	No void
D17	14.5 South	33" up from upper queen post plate	Channelizing, 10.5" solid wood
D51	14.5 South	7" down from upper queen post plate	Void @ top, 9" solid wood
D52	14.5 South	37" down from upper queen post plate	Void @ top, 10" solid wood
D53	14.5 South	57" up from lower queen post plate	Minor void at top, 11" solid wood
D54	14.5 South	12" up from lower queen post plate	Minor channelizing, 11” solid wood
D55	14.5 South	29" down from lower queen post plate	Possible minor channelizing, 11.5” solid wood

Drilling Number	Valley Rafter	Location	Comment
D56	14.5 South	51" up from lower plate	No void
D57	14.5 South	8" up from lower plate	No void
D58	14.5 South	24" down from Truss 15	No void
D59	14.5 South	11" down from Supplemental Truss	No void
D60	14.5 South	74" down from Supplemental Truss	No void
D61	14.5 South	transverse 1.5" from top, 6" from South Wall	No void
D62	14.5 South	transverse 4" from top, 6" from South Wall	4" solid wood, then void
D63	14.5 South	transverse 3.5" from top plate at South Wall	No void
D64	14.5 South	transverse 5.5" from top plate at South Wall, transverse 12.5" from top plate at South Wall	No void
D1	17 North	3" up from lower metal strap	No void
D2	17 North	approx. 83" up from west wall cord	No void
D3	17 North	approx. 123" up from west wall cord	No void
D4	17 North	4" up from lower queen post plate	Channelizing @ top, 11" solid wood
D5	17 North	transverse above lower queen post plate	Channel profile
D6	17 North	41" up from lower queen post plate	Channelizing, 10.5" solid wood
D7	17 North	73" up from lower queen post plate	Channelizing, 11" solid wood
D8	17 North	15" down from upper queen post plate	Channelizing, 10.5" solid wood
D9	17 North	21" up from upper queen post plate	No void
D10	17 North	39" up from upper queen post plate	No void
D11	17 North	80" up from upper queen post plate	No void
D12	17 North	6" down from upper metal strap/ 38" down from end of valley rafter	No void
D13	17 South	4" below top plate	No void
D19	17 South	33" down from top plate	No void
D20	17 South	39" up from upper queen post plate	No void, spacer decayed
D21	17 South	4" up from upper queen post plate	No void
D22	17 South	40" down from upper queen post plate	No void
D23	17 South	40" up from lower queen post plate	Channelizing, spacer gone, 11" solid wood
D24	17 South	4" up from lower queen post plate	2.5" void @ top, 9.5" solid wood
D25	17 South	5" down from lower queen post plate	2" void @ top, 10" solid wood
D26	17 South	30" down from lower queen post plate	3.5" void @ top, 9" solid wood
D27	17 South	65" down from lower queen post plate	2.25" void @ top. 9.5" solid wood
D28	17 South	transverse 1" from the top, 21" down from lower queen post plate	Mostly void
D29	17 South	11" up from bottom plate	Voids, 8" solid wood
D30	17 South	7" up from bottom plate	Voids, 8.5" solid wood
D31	17 South	6" up from wall truss	Void @ top, 10.5" solid wood
D32	17 South	transverse 3" from top above D31	1.5" internal void
D33	17 South	transverse 3.5" from bottom above D31	No void

Drilling Number	Valley Rafter	Location	Comment
D153	Post: 11J	3.5" down from the tie beam housing	No void
D154	Tiebeam: 11J-K	4.5" from post 11J	decay in top 1.5"
D155	Post: 11J	8.5" down from tie beam housing	No void
D156	Post: 11J	3.5" up from bottom of the tie beam housing	Tenon is decayed
D157	Post: 11J	10" above bottom of tie beam housing	No void

Use of Digital Radioscopy to Examine Connection Details

Although not part of the scope of work for this investigation, an opportunity existed to demonstrate the application of portable x-ray technology (digital radioscopy) to examine hidden conditions in the Breeding Barn.

The portable x-ray source used for the demonstration was the XR200® x-ray source manufactured by Golden Engineering, Inc. This model is a single packaged, pulsed source, producing x-ray pulses of short duration (60 nanoseconds or 6 x 10^{-8} seconds each) with minimal dose (3.1 milliroentgens for each pulse at a distance of 12 inches from the front of the unit), with energy up to 150 kV).

The digital imaging system used was the EPIX Digital Imaging System manufactured by Logos Imaging. The system is composed of imaging plates, the EPIX scanner and a laptop with software to import and save the scanned images. The imaging plates are reusable, photo-stimulatable phosphor imaging surfaces, 8" by 17" in size. X-ray images are created on the imaging plates as the phosphor crystals capture the energy of x-rays passing through the object of study.

The second component of the EXIX Digital Imaging System is the EPIX scanner. After exposure, the imaging plate is mounted on a cylindrical carousel and inserted into the scanner. The scanner uses red laser light to cause the crystals to release their stored energy, which is released as blue light captured by the scanner. The scanning process can capture the image at either high or low resolution. The laptop and software associated with the EPIX system capture this image and save the file as a TIF image for post processing.

This imaging system produces digital radiographs that are available for viewing within five minutes. It is easy to shift the imager if needed when the area of concern is not included in the image, or to shift the imager along an object (such as the truss chord) to make sequential radiographs. The images are stored to allow for post-processing to enhance features of interest within the image.

Since the images are TIF files, they can be manipulated by any standard photographic-enhancement software. However, the control unit (the software that is included for the laptop) includes a package that can also be used to enhance the images so that subtle details of the x-ray can be investigated. This software includes not only the standard image-enhancement techniques (such as image sharpening and contrast stretching), but also features designed to assist specifically with x-ray enhancement (such as the ability to transmit all the grey tones of the x-ray into a full spectrum of colors, and edge detection algorithms).

The radioscopy system was configured to examine a post-beam connection. The placement of the imaging plate and x-ray source are shown in Figure A. The resulting radiograph is shown in Figure B. The large steel plate is clearly visible as is an iron rod. With further data collection and data interpretation it would be possible to determine the connection details as well as the condition of the wood and steel components of these types of connections.

Figure A. Setup for digital radioscopy examination of a post-beam connection.

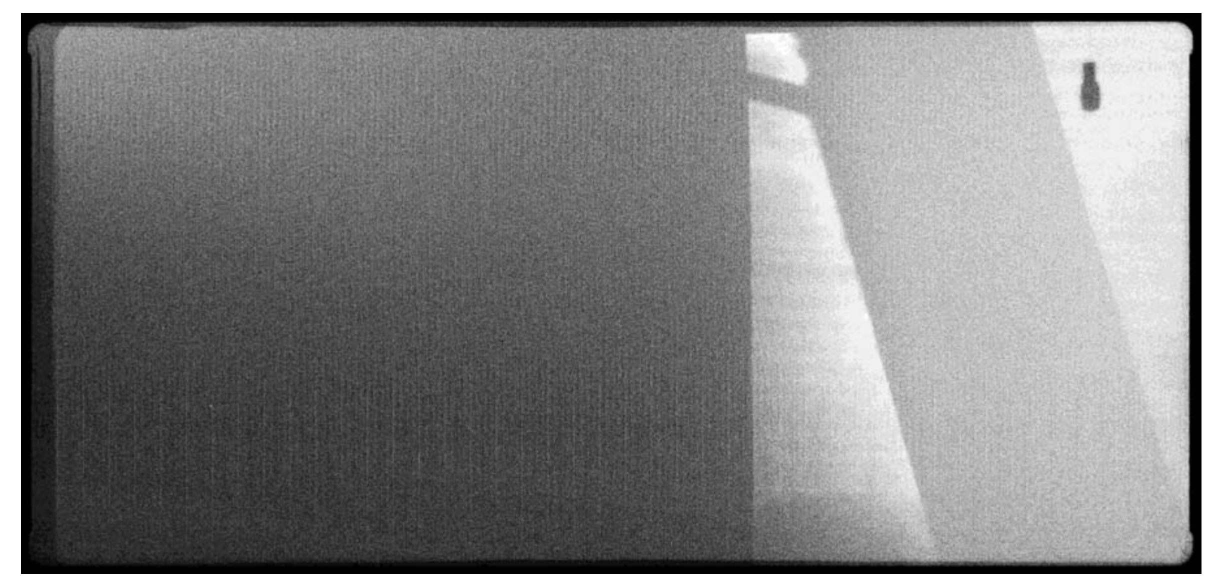


Figure B. Radiograph corresponding to the setup shown in Figure A.

Appendix I: Timber Repair Mockups and Testing

Repair options for deteriorated valleys included in-kind replacement, consolidation of decayed wood and filling of voids with resin-based fills, the addition of augmentation (such as a flitch), the splicing of new timber into decayed sections, and segmental infill of deteriorated sections using some form of dutchman repair. It was essential that repairs be executed in situ, have sufficient strength to carry roof loads, be as conservative of historic materials as possible, and have minimal impact on the visual integrity of the building interior. To determine which repair options might meet these requirements, a limited full-scale testing program was developed to evaluate the practicality of implementing a repair under field conditions while meeting strength requirements and minimizing visual impacts.

Of the potential repairs, replacement of entire rafters was considered a viable option in only one case, where the original timber was removed in an earlier repair campaign. Elsewhere, the deterioration in the valley rafters was limited to discrete sections of each rafter; therefore, selective augmentation, repair, or replacement seemed feasible. Use of a consolidant (like epoxy) to fill the voids was not considered viable because no destructive test data have been generated on the long-term performance of extensive repairs of structural timbers using consolidants.

The project team considered using fiber reinforced polymers (FRPs) to augment deteriorated valley members because of their diverse application to materials, including timber. A review of the use of FRPs led to the conclusion that they were not suitable for the Breeding Barn since they are most effective in tension and the necessary repairs were primarily on the compression face of the valley rafters. Additionally, the FRP would be visible on the bottom of the valley rafters, which was incompatible with the aesthetic objectives of the repairs.[1] Therefore, the preferred methods of repair included scarf splices and segmental infill using either solid timber or engineered lumber to provide adequate strength.

Test Procedure

Bending tests were conducted to determine the relative capacity of the various repair options. The intent was not to select a repair based solely on strength but also to assess the potential for conducting a field repair under the conditions expected in the Breeding Barn.

Test procedures for large timbers are provided in ASTM D 198 – 05A, Static Tests of Lumber in Structural Sizes. These test procedures are used when it is desirable to compare test results from different research programs and specify dimensions, load conditions, and the means to calculate the stresses at the time of failure. As such, the test procedure used was based on ASTM D 198 but incorporated some variation to better assess the performance of each repair type. The test configuration is shown in Figure 1. The test specimens were nominal 10-inch by 10-inch southern pine timbers. Actual dimensions were recorded for each specimen. The length was approximately 96 inches with a test span, L, of 84 inches. A load cell was used to record the pounds force applied at mid-span up to the failure load.

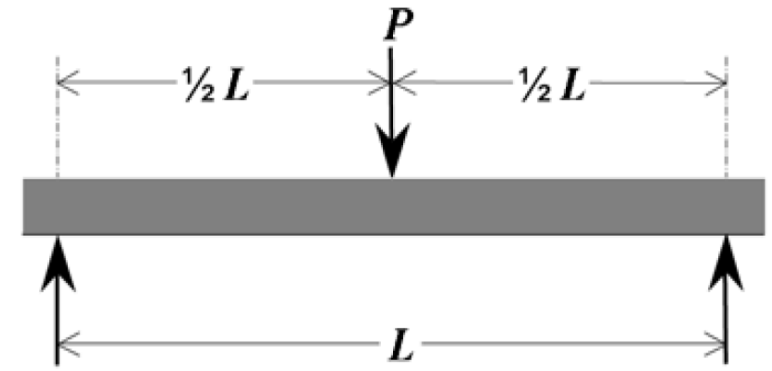


Figure 1. Test configuration used to determine the bending strength of the timber specimens (from ASTM 198 – 05A).

In reviewing the results, it may be helpful to draw attention to a few examples. Figure X2 is one of the solid timber control specimens (#14). The image on the left shows that this specimen had moderate slope of grain, which is known to reduce the strength of timber. The image on the right is a view from the side showing the failure on the tension side of the beam below where the load was applied. For simply supported beams, this failure pattern is typical, although there can be considerable variation due to the presence of local defects in the timber.

Figure 2. Control sample with moderate slope of grain (on the left) and failure on the tension zone (on the right).

---

1 Carbon fiber polymers, also called fiber-reinforced polymers, have been used for a number of years to strengthen timber, concrete, steel, masonry, and stone structural members. Typical applications of FRPs include column-beam connections, seismic retrofitting, repair of corrosion-damaged beams and columns, bridge decks, piles, precast prestressed concrete shells, and roof structures. FRPs are considered to have a number of advantages for use, including a wide range of products with specified tensile strengths, low mass, ease of fabrication, custom colors and coatings, custom geometry, resistance to corrosion, and low transportation costs. Additionally, FRPs can be made from recycled plastics. However, FRPs have some disadvantages. These disadvantages include high initial costs and the need for highly trained and specialized engineers to design the structural systems, as well as potential creep, rupture, and shrinkage issues.

Research has been conducted by the Constructed Facilities Center (CFC) and the Institute for the History of Technology and Industrial Archaeology (IHTIA) of West Virginia University (WVU) to develop methodologies to strengthen structural wood elements of historic covered bridges using Glass Fiber Reinforced Polymer (GFRP) composite materials. Between 2000 and 2004, laboratory experiments funded by FHWA were conducted to test the effectiveness of GFRP composite materials (both plates and rebars) on the bending and shear capacities of structural members. The specific objectives were to develop methods to strengthen truss and arch members and floor beams. The tests included the use of adhesive necessary to fix the GFRP composite materials in place.

The results of the testing were somewhat mixed. In small-scale bending tests, strength and stiffness was improved by bonding a GFRP plate to the tension face of the member, but large-scale tests indicated that considerable surface preparation was necessary to insure an adequate bond in order for the GFRP-bonded timber to perform better than the control sample. The bonding of the GFRP plate also required routing a cutout for the placement of the plate along the tension side of the test specimens. Initial bending tests with GFRP rebars at the top and bottom of the test specimens failed at the bond and did not improve member performance; this method, which may be more suitable for compression members in trusses, is similar to other types of doweled repairs.

Figure 3 is one of the scarf joint specimens (#3). The image on the left is a side view of the joint after failure. The timber keys tended to rotate as the load increased, allowing the scarf faces to slip relative to one another and open on the tension side of the beam, and ultimately fail at a relatively low load. The image on the right shows the top of the specimen after failure.

Figure 3. Failure of a scarf joint repair specimen.

Figure 4 is one of the LVL Dutchman repair specimens (#9). The image on the left is a top view showing the position of two sections of LVL that were glued into the notch cut into the solid timber to simulate the removal of the decayed wood. The importance of the gluing process is discussed below. The image on the right shows a side view of the failure, which is typical for the type of failure that can be observed in solid timber. The significance of this failure mechanism, in addition to the high load at failure, is that the test results indicate the repair could provide a reasonable representation of solid timber behavior. Since the repair is installed from the top of the timber, visual impact is minimized.

Figure 4. High-quality LVL Dutchman repair specimen after failure.

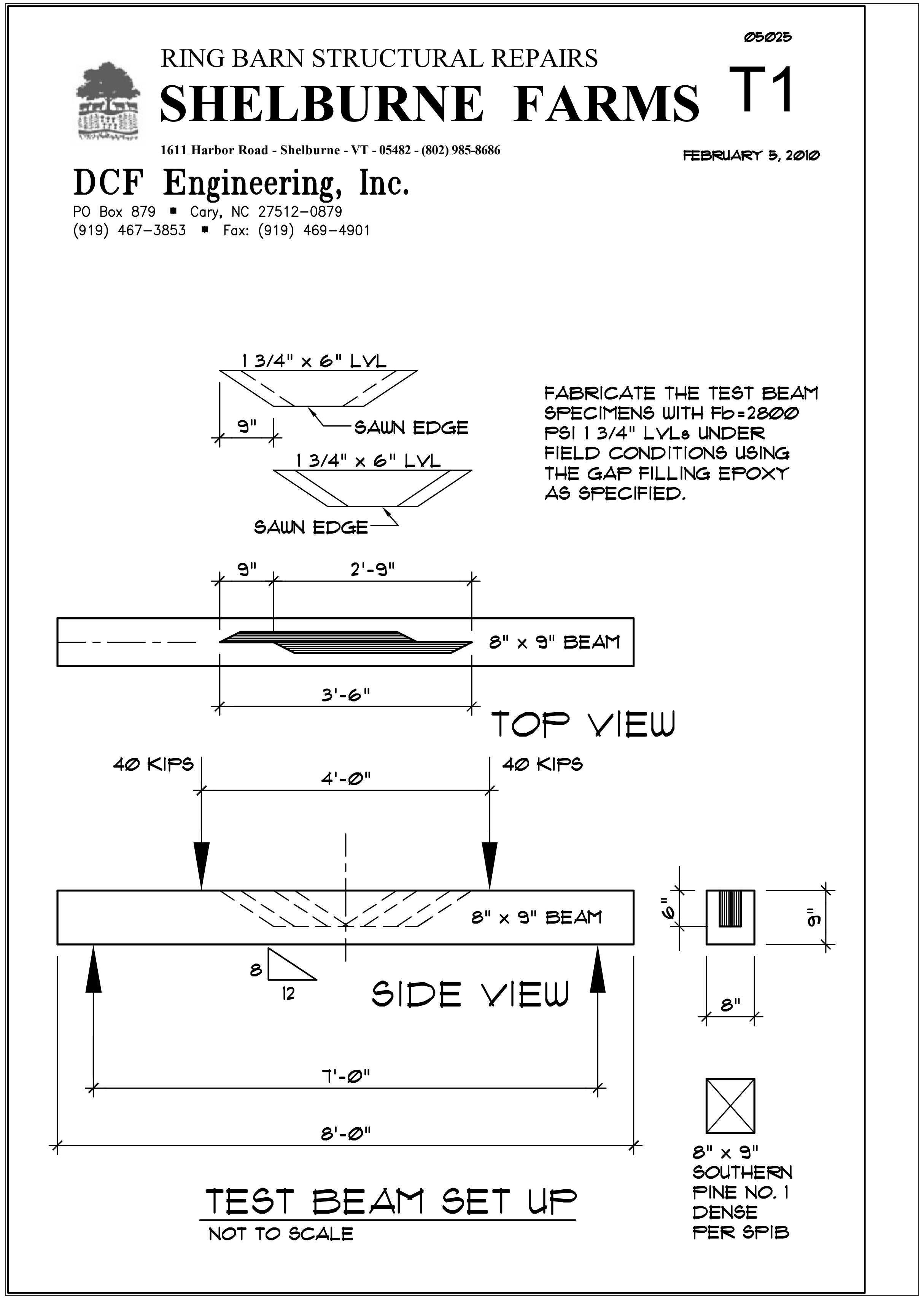
Not all of the LVL Dutchman specimens performed as well as the one described above. Figure 5 is another LVL Dutchman repair specimen (#2); one that failed at a load not significantly lower than the solid timber controls but failed in a manner that indicated that the repair was inadequate. This can be seen by the separation of the two sections of LVL with the section on the right in Figure 5 forced out of the notch as the load was applied. The bond quality between the timber and the LVL and between the sections of LVL was poor. In fact, it was easy to remove the LVL sections from the notch after cutting through the specimen (Figure 6).

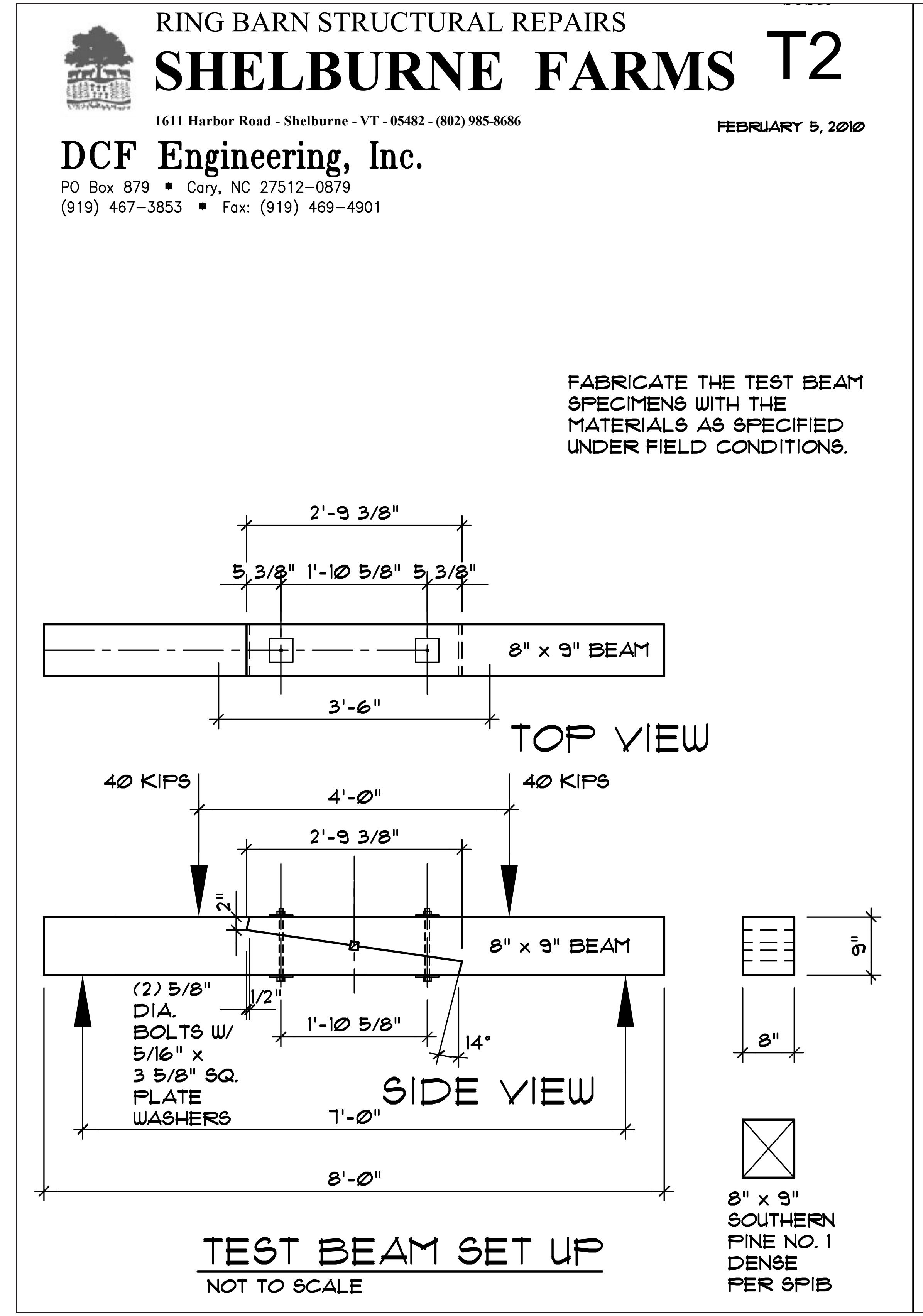
Figure 5. Poor-quality LVL Dutchman repair specimen after failure.

Figure 6. Poor bond as indicated by easy removal of the Dutchman.

The results of the segmental infill repair tests indicated that the gluing process was critical to a high-strength joint. If the gap was too large or uneven, the adhesive did not have the gap-filling properties to form an adequate bond and failure occurred in the glue line, rather than in the wood as intended. This led to adjustments of the process so that (1) the removal of the decayed wood from the valley rafters was done to provide a tight channel for inserting the sections of LVL and (2) the adhesive was applied carefully so that a strong bond could develop between the timber and the LVL. Additionally, detailing of dutchman infill pieces included square (rather than angled) crosscuts to improve the transfer of compressive stresses across the joint and reduce the possibility of infill pieces lifting under load.

The test specimens were placed into three general groups; controls (solid timbers with no repair), scarf joints, and segmental infill. A review of the data shows that the solid timber control specimens had relatively high bending stresses at failure. The scarf joint repairs did not perform well relative to the controls while the segmental infill repairs, generally, had comparable strength to the controls. In this appendix, test schematics illustrate typical test scenarios for samples with scarfed splices and with segmental infill repairs. The test results for each specimen are summarized in the accompanying table. Stress-strain diagrams and images of the samples at failure are included in the individual sample data sheets.


RING BARN STRUCTURAL REPAIRS
SHELBURNE FARMS
T1
05025
1611 Harbor Road - Shelburne - VT - 05482 - (802) 985-8686
FEBRUARY 5, 2010
DCF Engineering, Inc.
PO Box 879 ■ Cary, NC 27512-0879
(919) 467-3853 ■ Fax: (919) 469-4901
1 3/4" x 6" LVL
9"
SAWN EDGE
1 3/4" x 6" LVL
SAWN EDGE
FABRICATE THE TEST BEAM SPECIMENS WITH Fb=2800 PSI 1 3/4" LVLs UNDER FIELD CONDITIONS USING THE GAP FILLING EPOXY AS SPECIFIED.
9"
2'-9"
8" x 9" BEAM
3'-6"
TOP VIEW
40 KIPS
4'-0"
40 KIPS
8" x 9" BEAM
6"
9"
8
12
SIDE VIEW
8"
7'-0"
8'-0"
8" x 9" SOUTHERN PINE NO. 1 DENSE PER SPIB
TEST BEAM SET UP
NOT TO SCALE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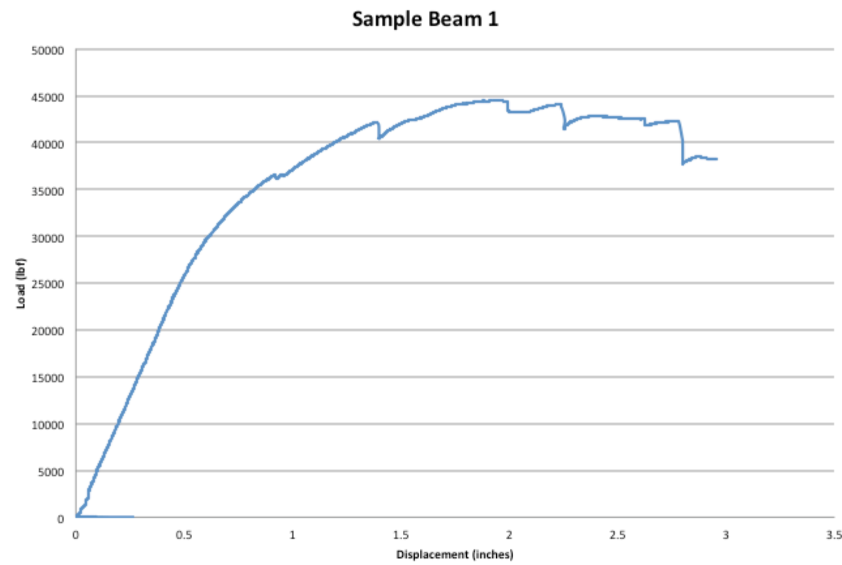
FEBRUARY 5, 2010
DCF Engineering, Inc.
PO Box 879 ■ Cary, NC 27512-0879
(919) 467-3853 ■ Fax: (919) 469-4901
FABRICATE THE TEST BEAM SPECIMENS WITH THE MATERIALS AS SPECIFIED UNDER FIELD CONDITIONS.
2'-9 3/8"
5 3/8" 1'-10 5/8" 5 3/8"
8" x 9" BEAM
3'-6"
TOP VIEW
40 KIPS
4'-0"
40 KIPS
2'-9 3/8"
2"
8" x 9" BEAM
9"
(2) 5/8" DIA. BOLTS W/ 5/16" x 3 5/8" SQ. PLATE WASHERS
1/2"
1'-10 5/8"
14°
8"
SIDE VIEW
7'-0"
8'-0"
8" x 9" SOUTHERN PINE NO. 1 DENSE PER SPIB
TEST BEAM SET UP
NOT TO SCALE

Test Results

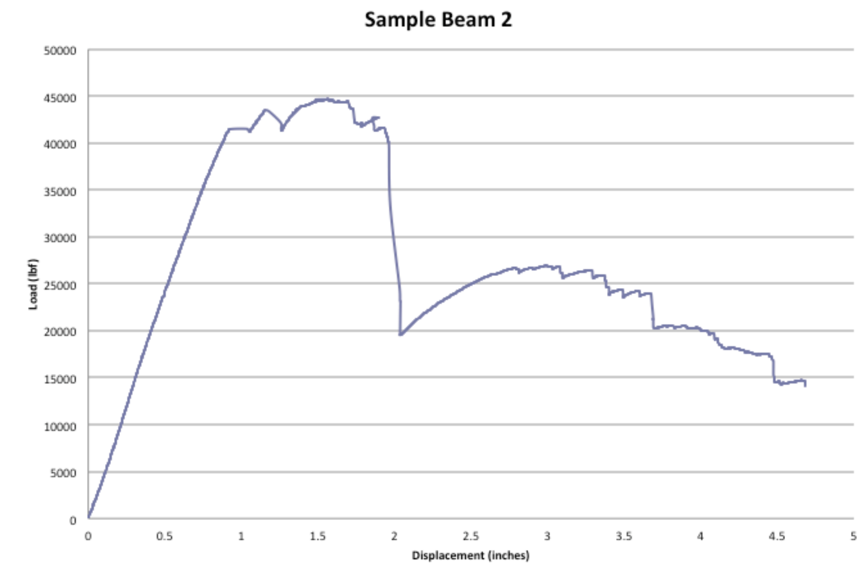
Specimen Type	Maximum Stress (pounds per square inch)	Comments	Sample Number
Control	5,278	shear failure	1
Control	6,089	tension failure	6
Control	6,152	shear failure	8
Control	6,274	tension failure; has moderate slope of grain	14
Scarf-single key	1,895	rolling of the key and opening of joint on tension side	3
Scarf-single key	1,738	tension failure	13
Scarf-two key	2,035	rolling of lower shear key, compression failure around washers, and shear failure	12
Scarf-nosed	2,204	shear failure at lower cog housing	4
LVL dutchman	5,360	tension and shear failure; adhesive was not applied in repair and did not bond to timber	2
LVL dutchman	5,916	tension failure	5
LVL dutchman	6,779	tension failure	7
LVL dutchman	7,845	tension failure	9
LVL dutchman	6,943	shear failure	10
LVL dutchman	6,925	tension failure	11



SAMPLE NO: 1 (10 3/16 x 10 ¼)

SAMPLE TYPE: Control

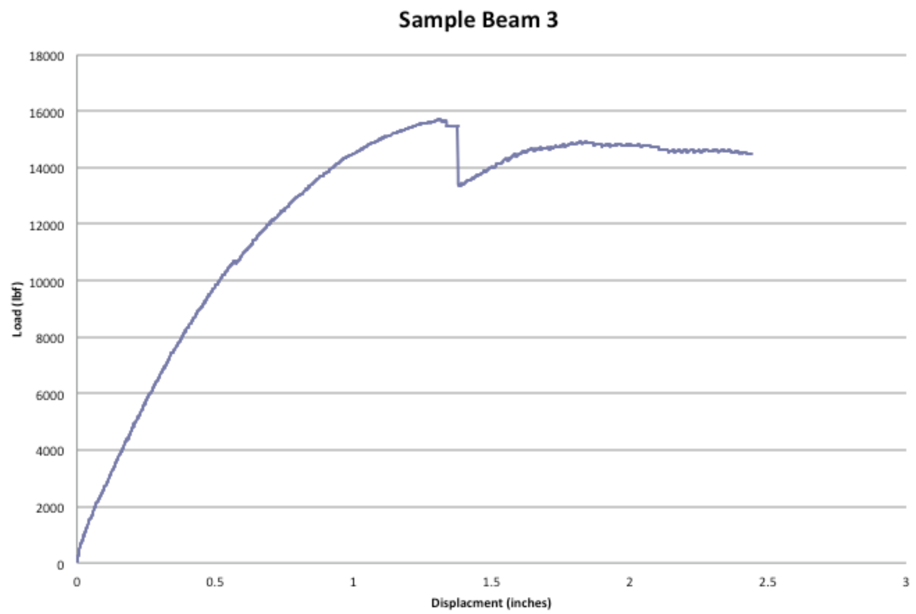
MODE OF FAILURE: Shear failure (sample left) at 44,000 lbf



SAMPLE NO: 2 (10 ¼ x 10)

SAMPLE TYPE: LVL dutchman

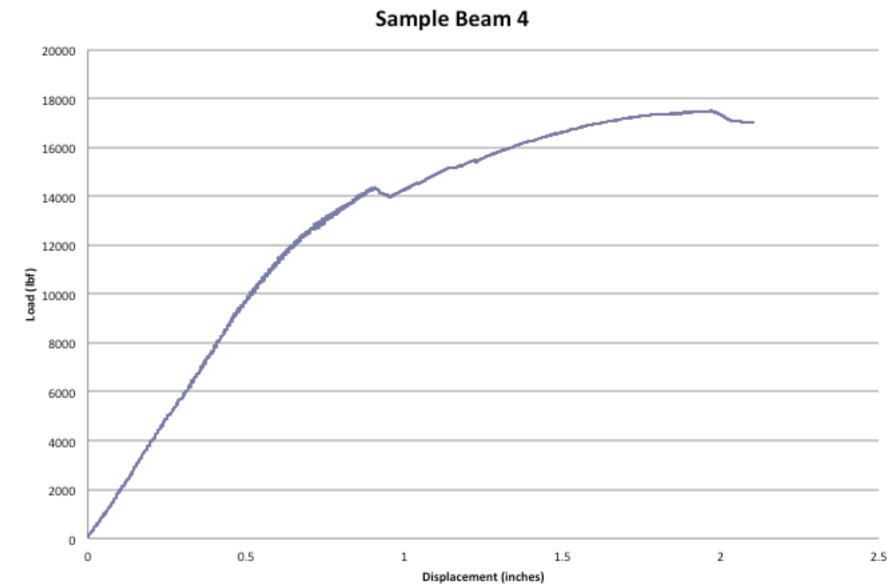
MODE OF FAILURE: Compression failure at load cell followed by tension failure shortly after at about 44,600 lbf; accompanied by shear failure (sample right). On opening the joint, it was clear that adhesive was never applied to the Dutchman, but only to the mortise. The lvl was easy to remove in pieces once the timber had been cut across the repair



SAMPLE NO: 3 (10 1/8 x 10 3/16)

SAMPLE TYPE: Scarf (single key, nearly square in section)

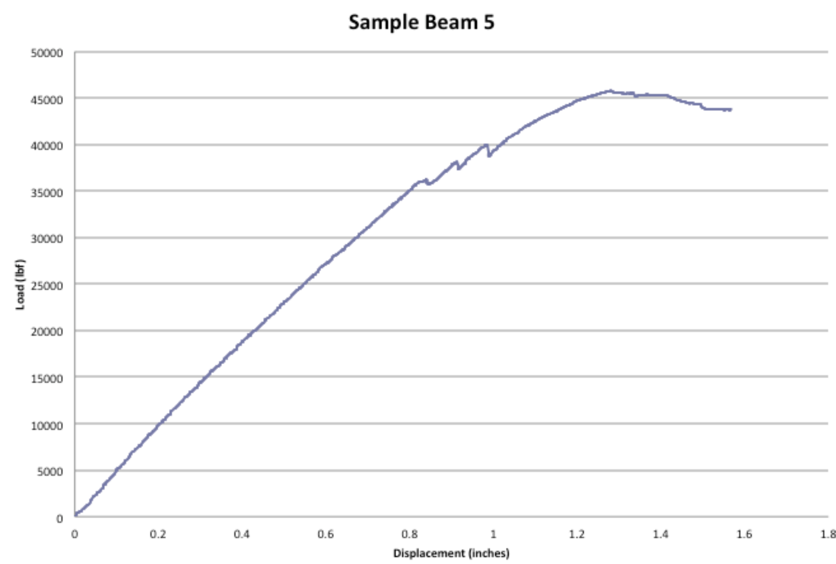
MODE OF FAILURE: Opening of joint on tension side accompanied by rolling of the key and failure of the upper half of joint at 15,600 lbf



SAMPLE NO: 4 (10 x 10)

SAMPLE TYPE: Nosed scarf

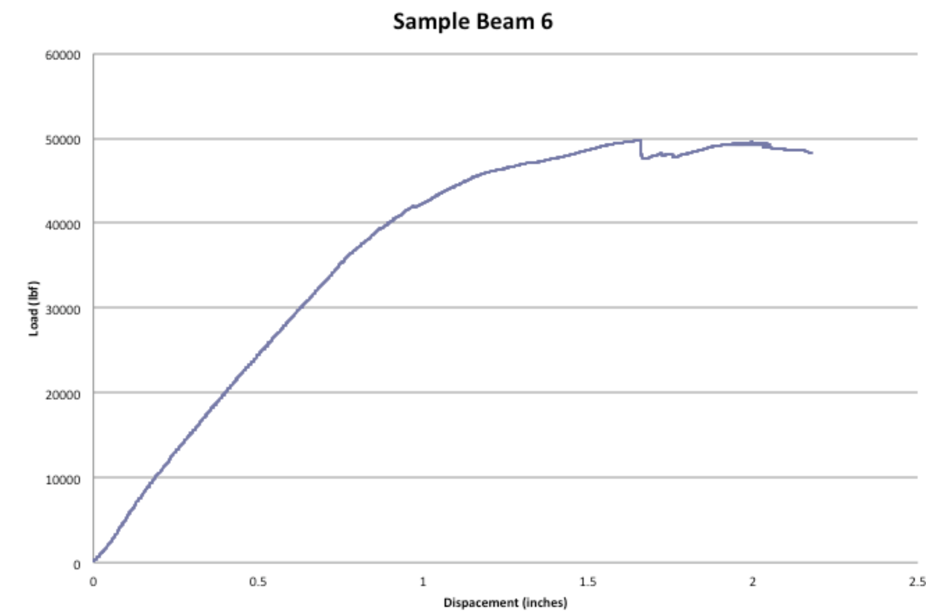
MODE OF FAILURE: Shear failure at lower cog housing at 17,500 lbf



SAMPLE NO: 5 (9 7/8 x 10)

SAMPLE TYPE: LVL dutchman

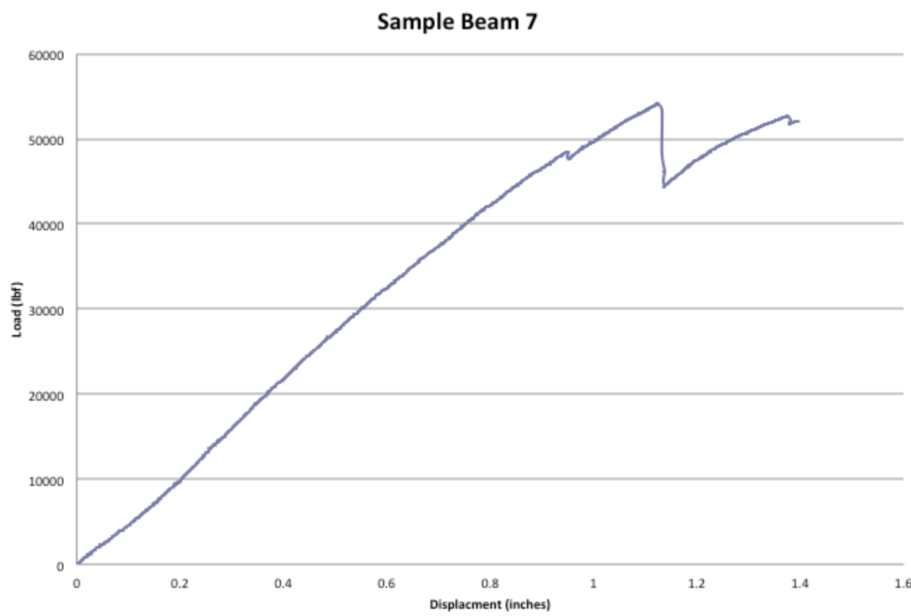
MODE OF FAILURE: Compression failure followed by tension failure at load cell at 45,800 lbf.



SAMPLE NO: 6

SAMPLE TYPE: Control

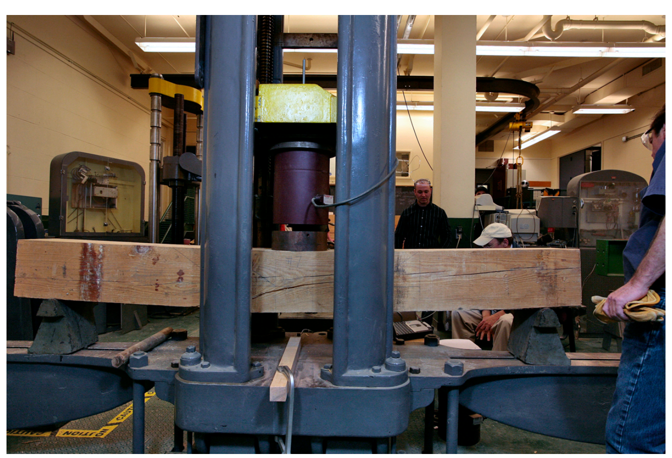
MODE OF FAILURE: Compression failure followed by tension failure at load cell at 50,000 lbf.

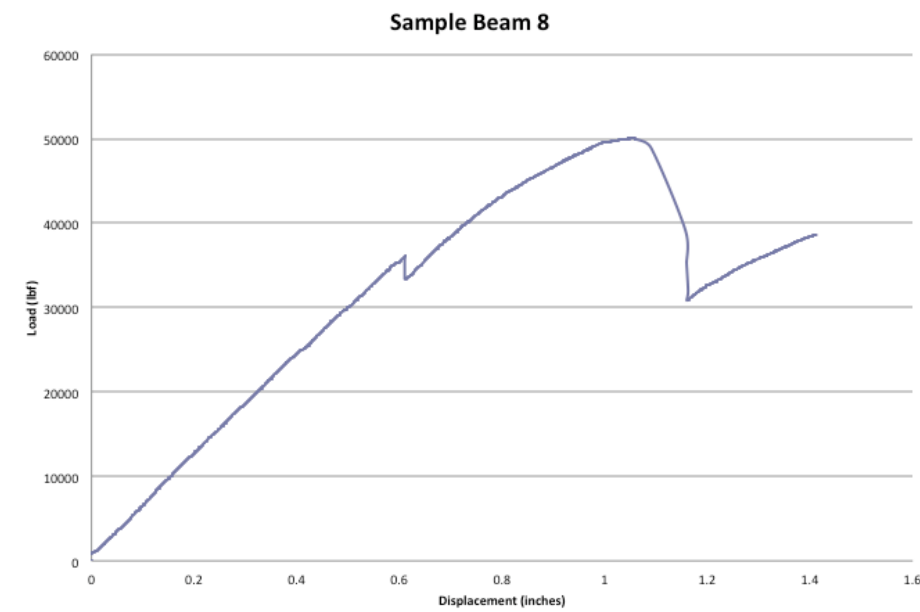


SAMPLE NO: 7 (10 x 10 1/16)

SAMPLE TYPE: LVL dutchman

MODE OF FAILURE: Tension failure initiated in lower third of sample depth at about 47,500 lbf. Glue joints at top surface began to open at about 48,000 lbf. A second crack below the neutral axis but originating at about the same level as the bottom of the trench appeared at 53,000 lbf.

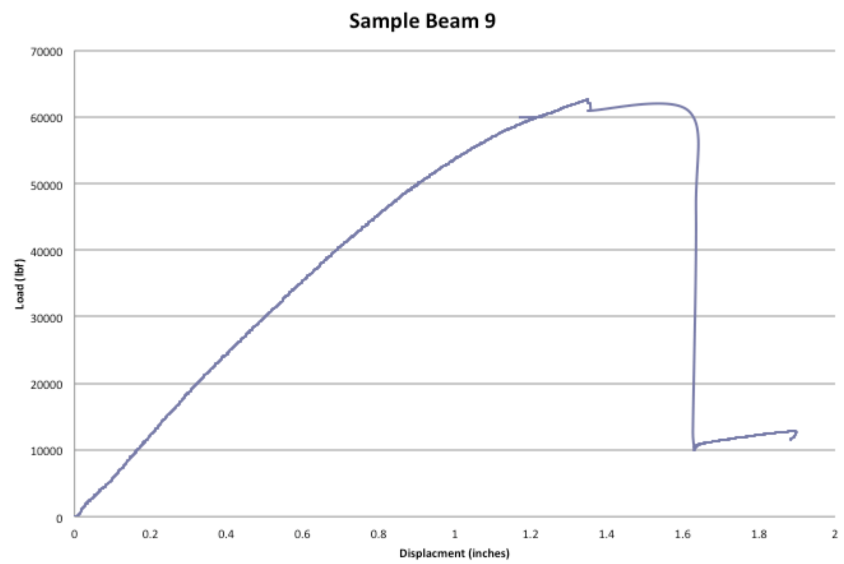




SAMPLE NO: 8 (10 1/16 x 10 1/8)

SAMPLE TYPE: Control

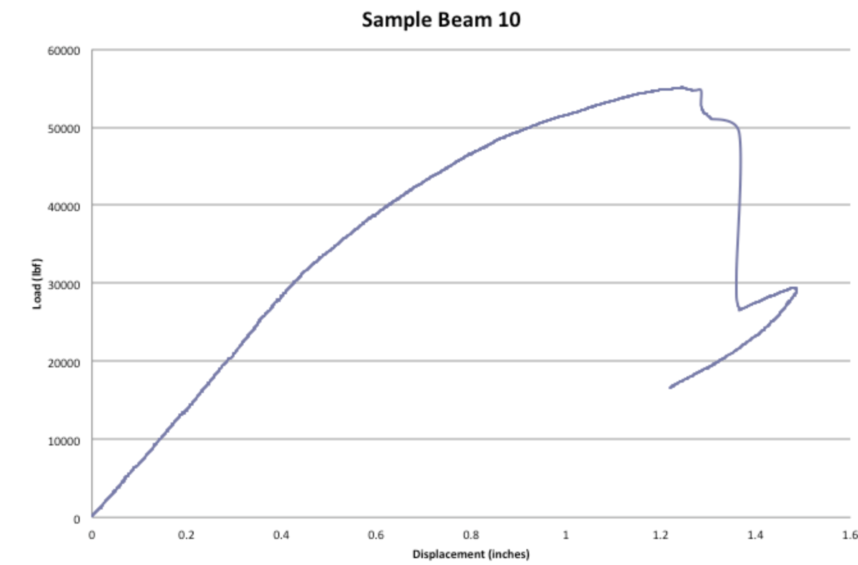
MODE OF FAILURE: Shear failure (sample left) initiated at about 35,000 lbf. Separation of heartwood at approximately 40,000 lbf



SAMPLE NO: 9 (10 x 10 1/16)

SAMPLE TYPE: LVL dutchman

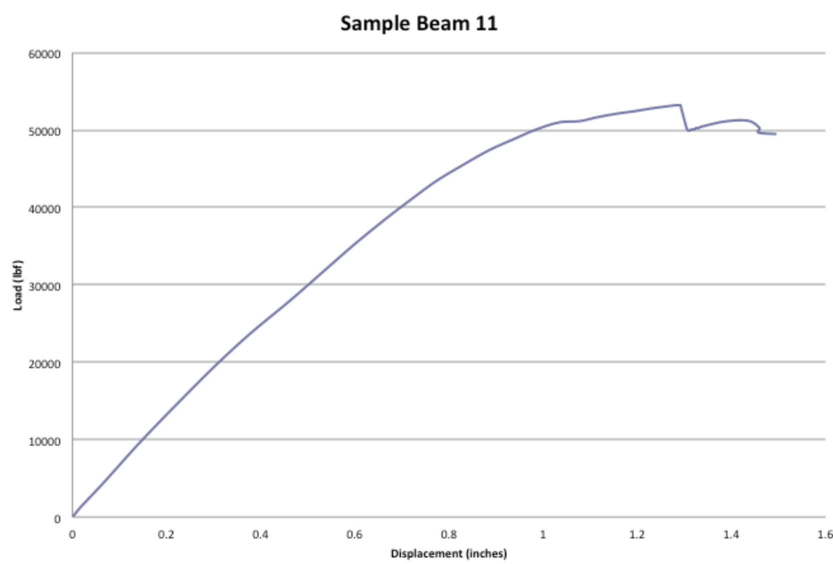
MODE OF FAILURE: Glue joints at top surface began to open @ 56,000 lbf, with abrupt failure in tension @ 62,000 lbf.



SAMPLE NO: 10 (10 x 10)

SAMPLE TYPE: LVL dutchman

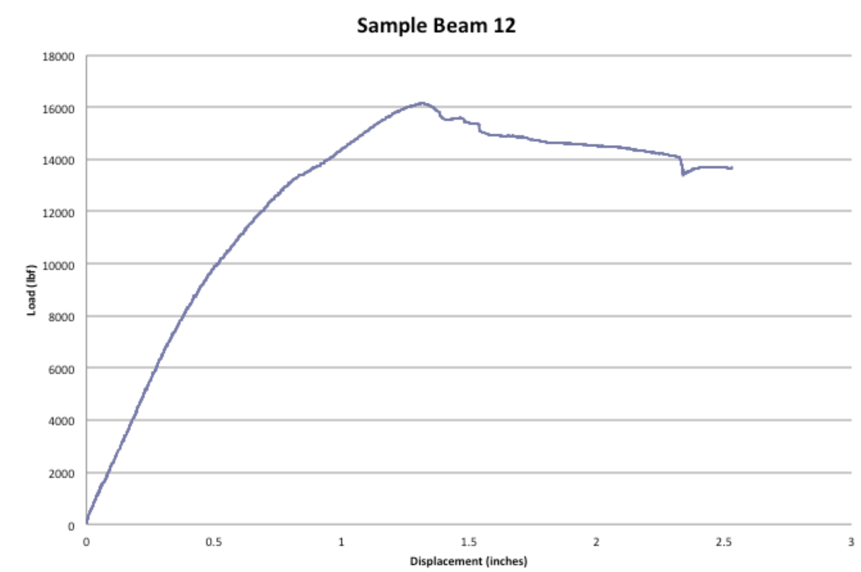
MODE OF FAILURE: Glue lines opened @ 48,000, with compression failure in cheeks below load cell initiating @ 53,500 lbf, failure in shear (sample right) @ 54,000 lbf



SAMPLE NO: 11 (9 7/8 x 9 15/16)

SAMPLE TYPE: LVL dutchman

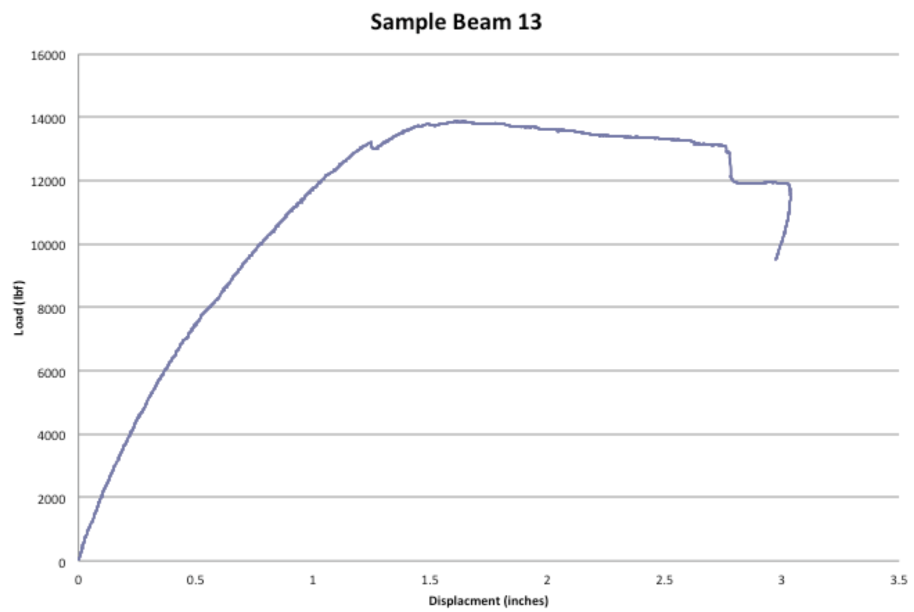
MODE OF FAILURE: Shear cracks appeared at ends of sample @ 50,000 lbf, with tension failure initiating @ 51,000 lbf, complete @ 52,500 lbf



SAMPLE NO: 12 (10 x 10)

SAMPLE TYPE: Scarf (2 keys)

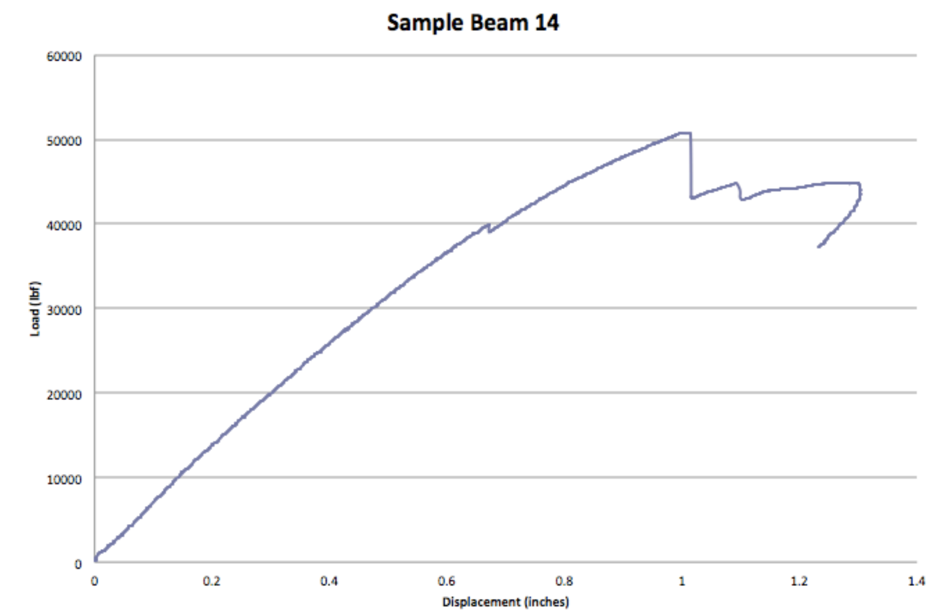
MODE OF FAILURE: Noticeable opening of joint on tension side at about 8,000 lbf, rolling of lower shear key by about 12,000 lbf. Compression failure around washers (top surface), shear crack opened in upper key and forward of it at about 15,000 lbf. Ceased to carry additional load at just over 15,000 lbf.



SAMPLE NO: 13 (10 x 10 1/16)

SAMPLE TYPE: Scarf (1 key)

MODE OF FAILURE: Opening on tension side noticeable by about 4,000 lbf. Beginning to see compression around bottom washers at 8,000 lbf. Some compression noticeable around key at 12,000 lbf. Overall deflection of about 2 3/8-inches at 13,000 lbf.



SAMPLE NO: 14 (10 1/16 x 10 1/16)

SAMPLE TYPE: Control

MODE OF FAILURE: Slope of grain steeper than other samples. Failure in tension initiating @ 38,000 lbf, ultimate failure @ 49,000 lbf

Appendix J: Modeling and Analysis

Through modeling and analysis of the principal roof frame elements, it was determined that factors of safety for each of the primary elements are satisfactory. This was the central goal and key accomplishment of the building investigation. In earlier engineering assessments of the Breeding Barn, it was thought that overstresses in principal trusses, purlins, and valley rafters required structural augmentation. Under the repair scenarios developed in these earlier studies, recommended treatments would have resulted in locating loads in new elements, bypassing the historic structural system. Through comprehensive archival and structural survey and in-depth analysis of the building, unnecessary and invasive additions to the structural system were avoided.

Modeling was focused on the principal truss in two configurations, the first consisting of the truss alone, requiring that lateral tensile forces be resisted by iron lower chord elements in the truss. The second configuration added the cross-brace tie to the truss as an additional path for the force to be resisted. By including the cross-brace ties in the analysis of the building cross section, the center tie bar of the truss becomes a zero force member.

Preliminary analysis of the principal truss was performed with a 30 psf snow load and 15 psf dead load. In a simple plane frame analysis of the truss only, the 1-inch diameter rods connecting the truss heel to the struts are stressed to an f_T= 36,057 psi. This is very high when compared to an average elastic limit of 33,150 psi and an average ultimate strength in tension of 47,330 psi.[1] Although the stresses for wrought iron components are high, all but one component are close or within range of the 25,000 psi to 30,000 psi elastic limit published in period handbooks.

Given the high stresses in some of the iron truss rod elements, a second computer model was constructed, substituting the very substantial cross-brace ties that connect every truss pair for the center iron tie in the truss. Load tests of the truss at gridline 11.0 were conducted, employing two loading scenarios consisting of 1000-lb. and 2000-lb. (force) unit loads suspended from purlins and panel points. Member forces derived from analyses using the same unit loads were compared to the results of the load test.

Vibration testing, supplemented by strain gauge measurements, was used to measure changes in strain in bottom chord elements. In both loading scenarios, measured strains were substantially lower than values based on the original model, tending to confirm modeling scenarios in which horizontal forces are managed by cross-brace ties (supplemented by aisle framing). The test added credence to the thesis that R. H. Robertson added the cross-brace tie rods to the basic truss configuration sometime during construction[2], thus reducing horizontal and vertical deflection and reducing the stresses in both the original wrought iron elements and timber top chord.

1 These are average values determined through strength-in-tension testing of four iron samples collected from the barn roof (see Appendix G: Iron Characterization and Testing)

2 For an argument concerning the development of the structural design based on the surviving drawings, please see Appendix A: Archival Drawings.

This appendix includes:

1. Document review 7/14/05 (DCF Engineering)
2. Truss analysis, load case #1 7/22/05 (DCF Engineering)
3. Truss analysis, load case #2 8/30/05 (DCF Engineering)
4. Moment diagram, truss 11.0 South 7/18/10 (DCF Engineering)
5. Repair detail, truss 11.0 South 7/18/10 (DCF Engineering)
6. Truss member properties, calculated axial loads (excerpted from Ernst, M. Assessment of the Breeding Barn roof structure using truss member resonant frequencies and computer modeling. Master's thesis, School of Engineering, University of Vermont. 2009

DCF Engineering, Inc.

July 14, 2005 05025

Douglas Porter
Graduate Program in Historic Preservation
University of Vermont
133 S. Prospect St.
Burlington, VT 05405

Re: Shelburne Farms
Breeding Barn
Shelburne, VT

Dear Doug:

Thank you for everything on last Wednesday. The Breeding Barn is certainly a great building on a wonderful site. I reviewed the following documents while traveling and arrived at several conclusions.

- Shelburne Museum Breeding Barn (CEA)
  Structural Evaluation
  February 5, 1990
  Civil Engineering Associates, Inc.
  Shelburne, Vermont
- Shelburne Farms
  Breeding Barn Complex
  Conservation Plan
  October 31, 2004
  Smith • Alvarez • Sienkiewycz, Architects
  Heritage Landscapes
  Mel Doherty, P.E.
  Mary Jo Llewellyn / Ann cousins
  Crothers Environmental Group
  Vermont
- The Breeding Barn
  Stabilization and Rehabilitation
  A plan for Shelburne Farms
  Made Possible by a Grant from
  The Vermont Housing and Conservation Board
  Prepared by the Office of Martin S. Tierney
- National Historic Landmark Nomination
  Designated a National Historic Landmark on
  January 3, 2001
  by the Secretary of the Interior
- Breeding Barn
  Tour Guide Manual 2004

PO Box 879 ■ Cary, NC 27512-0879 ■ (919)467-3853 ■ FAX (919)469-4901
EMAIL office@dcfengineeringinc.com

I focused on the CEA report because it appears to be what Mel Doherty relied on for his structural assessment in the 2004 SASA Conservation Plan.

Although the Civil Engineering Associates, Inc. report dated February 5, 1990 appears to be quite thorough and extensive, it is actually somewhat vague. For instance, the roof structure Sheets, S1 and S2, are drawn "not to scale" without basic dimensions shown. What is the overall size of the building, the span of the primary trusses, and the typical bay spacing? What is the roof pitch? Members should be called out, and hips, valleys, and ridges labeled. In short, the roof framing plan is drawn as if the drafter was unfamiliar with structural detailing for buildings. We are well aware that the drawings are meant to be merely a locator plan denoting areas of deterioration, but with some additional effort, these two sheets could have conveyed a considerable amount of useful information.

The basis for the conclusions reached in the report are not provided. What material design values were used in the analysis? Were allowable design values assumed from design values tabulated in The National Design Specification for Wood Construction for a particular grade of Southern Pine for beams and stringers? What value was assumed for the wrought iron components in tension?

Our procedure for evaluating a building such as this is to apply today's code mandated snow, live, and wind loads to various component systems, assuming that no deterioration has occurred. In this way, the original structure can be tested with specific design load criteria, against reasonable allowable design values with the amount of overstressing tabulated for various elements.

By performing a plane frame computer analysis, the stiffness in the various components can be included, resulting in accurate theoretical deflections. The computed dead and live load deflections can then be compared to today's code mandated limits for roof structures. Once this process is completed, then a review of the amount of overstress in particular elements can be compared against reasonable values which could be expected from dense clear growth Southern Pine harvested in the late 1880s. After the structural analysis is complete, then a condition analysis can be made on the basis of field observation, measurement, and testing. Through analysis and engineering judgement, the capacity of the various components can be tabulated accounting for deterioration.

The determination of an "overall safe live load capacity of the structure" reveals nothing about the various components. All of the components; trusses, rafters, purlins, sheathing, common rafters, valley and hip beams should be tabulated with the basis for their capacity individually noted. In the report, there is no discussion of the modifications to the basic values for timber design, such as Load Duration Factor and Size Factor. Load Duration Factor is an interesting subject which is central to timber design but largely ignored by structural engineers reviewing historic timber structures. Since wood has the ability to sustain substantially greater loads for short periods of time, allowable design values can be increased 15% for snow load, 60% for wind, and 100% for impact. This has implications for historic structures because the application of full design load of two months for snow load, and ten minutes for wind, acting on the structure is cumulative. The question becomes "what is the cumulative amount of time that this structure has been stressed to its full allowable design value for the various loading conditions, over its

Shelburne Farms Breeding Barns July 14, 2005 Page 3 of 3
05025

history?" It may be a very difficult question to answer without accurate weather data, but it should be discussed.

There are other "first impressions" that we should pass on. First, replacement-in-kind, with nominal increase in size of existing timbers, if necessary, should be the goal.

The scope of work should include the production of a set of measured drawings of at least the structural elements, based on what has now been produced including the original R.H. Robertson drawings. The existing condition of the roof structure should be updated using the Civil Engineering Associates, Inc. report as a guide. As soon as sufficient verification of sizes and dimensions allows, a structural analysis can proceed. A conditional analysis would lead into suggested repair strategies. Once agreement with the owner and approving agencies and grantor is received, then specific repairs can be designed and detailed.

Sincerely,
DCF Engineering, Inc.

David C. Fischetti

David C. Fischetti, P.E.
President

STATE OF VERMONT
DAVID C. FISCHETTI
No 5353
REGISTERED
PROFESSIONAL ENGINEER

DCF Engineering, Inc.

July 22, 2005 05025

Douglas Porter
Graduate Program in Historic Preservation
University of Vermont
133 S. Prospect St.
Burlington, VT 05405

Re: Shelburne Farms
Breeding Barn
Shelburne, VT

Dear Doug:

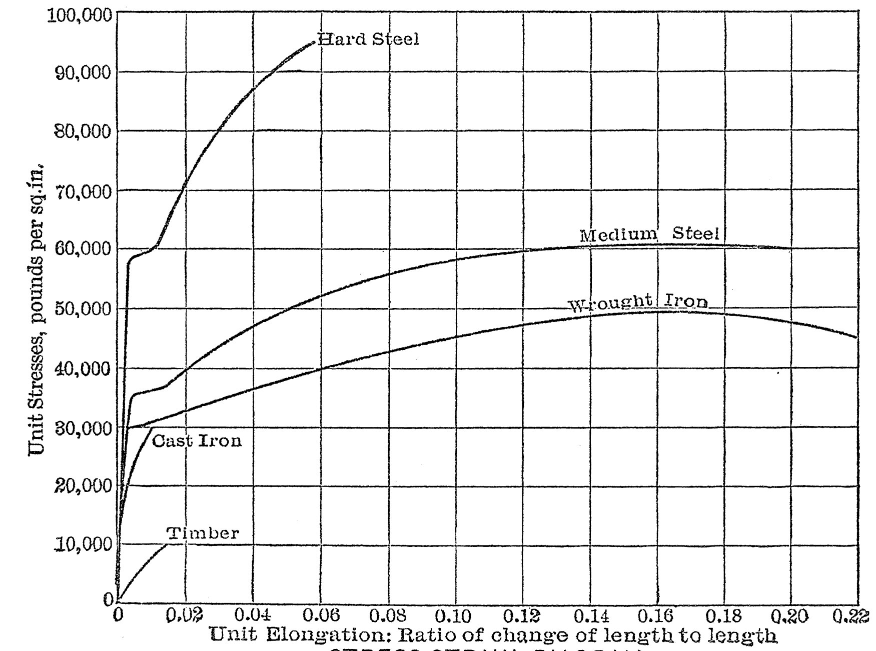
We performed a preliminary analysis of the primary truss with a 30 psf snow load and 15 psf dead load.

There are overstresses in the top chord as well as the rods which extend from the heel supports to the queenpost struts. In my opinion, the reason for the overstress in the 10" x 12" top chord may be because the original designer analyzed the truss using graphical means (force diagram and string polygon) first developed in the 1840's by Col. Stephen H. Long. This method of analysis only provides axial member forces. It is fairly accurate for trusses where purlin loads are applied to panel points. In this case, each side of the truss has reactions from purlins applied half way between panel points. Even today, with new structures, computer analyses will provide very large secondary bending forces that can not be determined from a graphical analysis.

The one inch diameter rods are stressed to a f_T= 36,057 psi. This is very high when compared to a tabulated elastic limit of 25,000 psi and a ultimate strength in tension of 50,000 psi. Furthermore, we measured these rods to be actually 7/8 of an inch in diameter during our brief visit on July 13, 2005. This would increase f_T to 47,072 psi which is well above the elastic limit for wrought iron.

By using ultimate published values for clear wood specimens reduced by a factor of 4.0, the top chord of the truss actually checks out (6% overstress). The static bending modulus of rupture and the maximum crushing strength in compression parallel to grain for clear straight-grained specimens of Loblolly Pine , divided by a factor of safety of 4.0 will yield values of Fb=3200 psi and Fc=1782 psi.

PO Box 879 ▪ Cary, NC 27512-0879 ▪ (919)467-3853 ▪ FAX (919)469-4901
EMAIL office@dcfengineeringinc.com



STRESS-STRAIN DIAGRAM

Although the stresses for wrought iron components are high, all but one component, are close or within range of the 25,000 psi to 30,000 psi elastic limit published in handbooks such as The Engineer's Manual by Ralph G. Hudson, S.B. , 1939, John Wiley & Sons, Inc.

Discarding the effects of deterioration for a moment, and ignoring connection design, we can say that as long as the stresses in the timber and wrought iron materials are within the elastic limit when reasonable design loads are applied, the Shelburne Farms Breeding Barn is not in danger of collapsing.

The overriding questions in the evaluation of this building, are as follows:
- What applied snow loads are reasonable to use in the Shelburne Farms area?
- What allowable design values should be used for the timber, steel and wrought iron?
- How has deterioration affected the capacity of the original design shown in R.H. Robertson's drawings?

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05025

- How can observation, measurement, testing and analysis be used to expose defects and deterioration critical to the safety of the structure?
- What solutions are available to ensure the continued service of the building with a reasonable level of intervention?

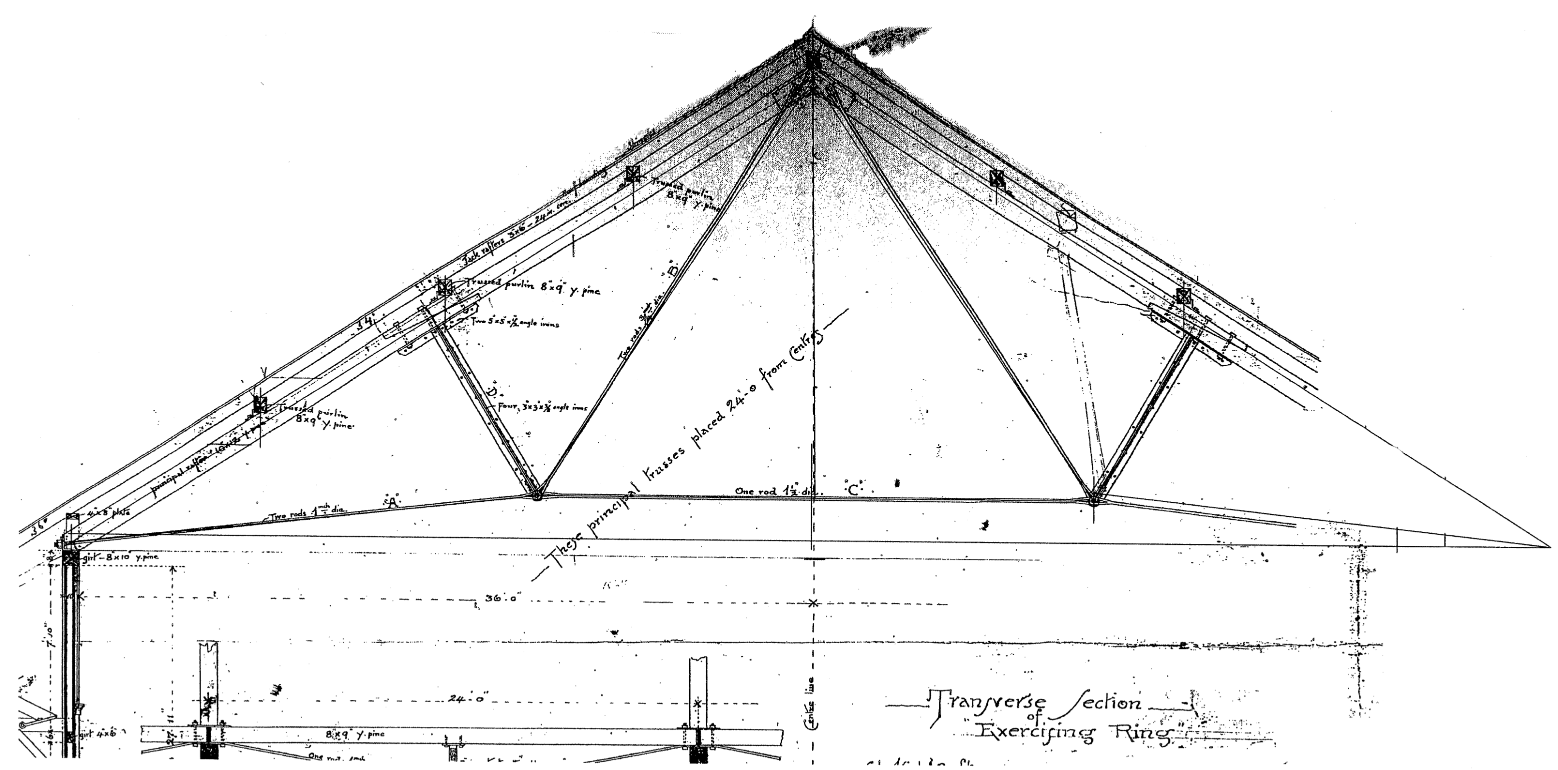
With this analysis it is hoped that we can move toward answering some of these questions.

Sincerely,
DCF Engineering, Inc.

David C. Fischetti

David C. Fischetti, P.E.
President

STATE OF VERMONT
DAVID C. FISCHETTI
No. 5353
REGISTERED
PROFESSIONAL ENGINEER



Project Name: Shelburne Farms Breeding Barn 05025

SECTION PROPERTIES

Member	Size			A	S	I
	b	x	d	In^2	In^3	In^4
		d (Dia.)		b x d	$\frac{b(d^2)}{6}$	$\frac{b(d^3)}{12}$
Top Chord 10" x 12"	10.0000	x	12.0000	120	240	1440
Strut (4) L 3"x 3"x 3/8"				8.44	4.56	13.69
Rods	(2) 1.0000			1.5708	0.1964	0.0982
Rod	1.5000			1.7671	0.3313	0.2485
Rods	(2) 0.7500			0.8836	0.0828	0.0311
Rods	(2) 0.8750			1.2026	0.1315	0.0575

MATERIAL PROPERTIES

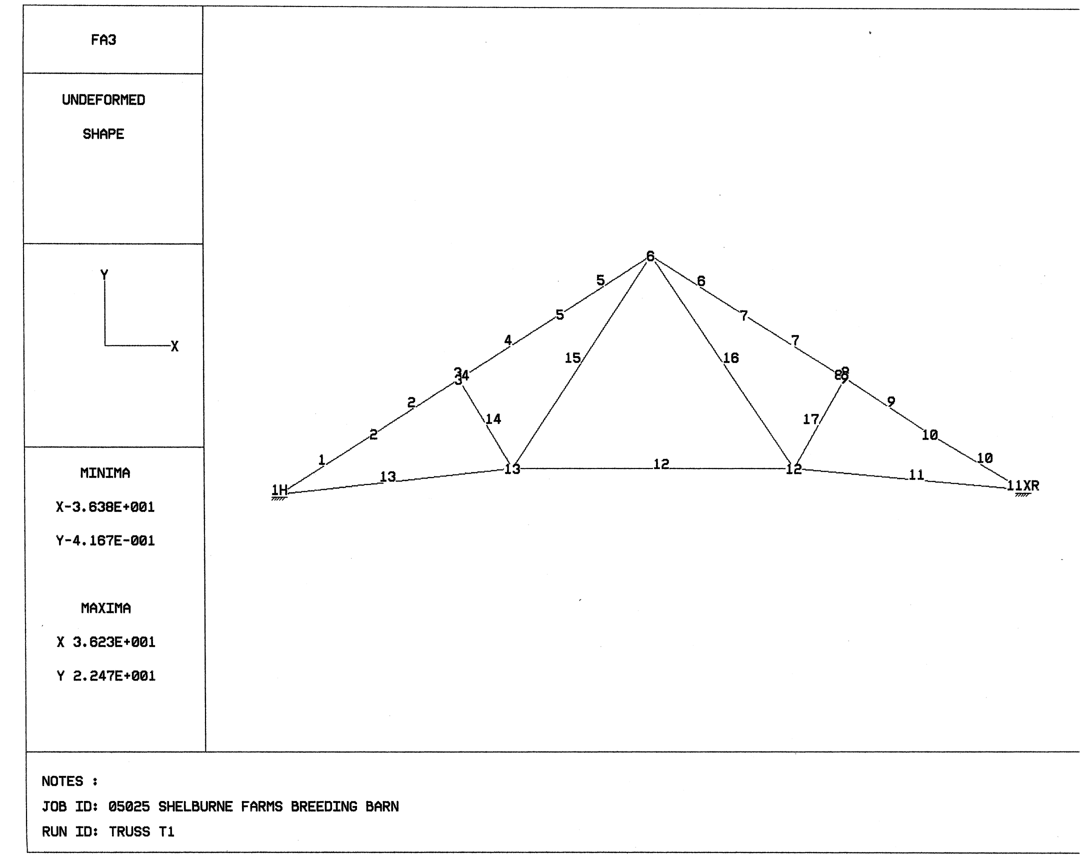
Southern Pine	E= 1,600 ksi
Wrought Iron	E= 28,000 ksi
Structural Steel	E= 29,000 ksi

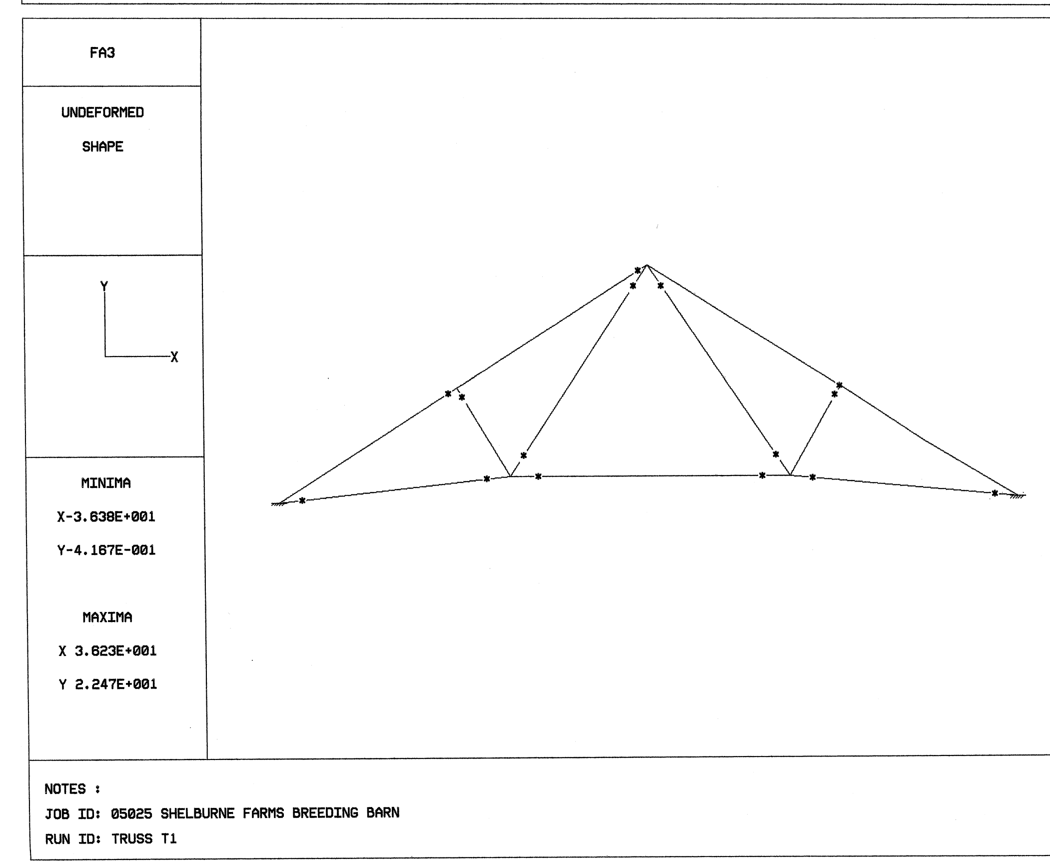
ALLOWABLE DESIGN VALUES

Southern Pine	Fb = 1,750 psi
	Fc = 1,100 psi
Wrought Iron	Ft = 14,000 psi
Structural Steel	Fc = 12,576 psi (Based on L=10 ft., r = 1.27)

MEMBER FORCES

Member	Force	Stress
(2) 1" Dia. Rods	T= 56609#	f_T = 36057 psi
1 1/2" Dia. Rod	T= 34611#	f_T = 19587 psi
(2) 3/4" Dia. Rods	T= 23261#	f_T = 26171 psi
L Struts	C= 16307#	f_c = 1932 psi





PROGRAM : General Frame Analysis v2.05 PAGE NO. 1
DCF ENGINEERING, Inc. TIME : Fri Jul 22 14:29:40 2005
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

N O D A L   I N F O R M A T I O N

NODE NO	NODAL COORDINATES X	Y	SUPPORT CONDITIONS CODE	PX STIFF	PY STIFF	M STIFF
Units :	Ft	Ft		K /In	K /In	K -In /Deg
1	-36.375	-0.417	H			
2	-27.149	5.390				
3	-18.797	10.651				
4	-18.150	11.057				
5	-8.870	16.900				
6	0.000	22.474				
7	9.089	16.838				
8	18.286	11.135				
9	18.870	10.770				
10	27.151	5.354				
11	36.229	0.000	XR			
12	13.917	2.036				
13	-13.583	2.073				

E L E M E N T   I N F O R M A T I O N

ELEM NO	NE NODE	PE NODE	ELEM LENGTH	BETA ANGLE	PROP TYPE	ELEM TYPE	NE HINGE	PE HINGE
		Units :	Ft	Deg				
1	1	2	10.901	32.19	1	BEAM		
2	2	3	9.871	32.21	1	BEAM		Y
3	3	4	0.764	32.11	1	BEAM		
4	4	5	10.966	32.20	1	BEAM		
5	5	6	10.476	32.15	1	BEAM		Y
6	6	7	10.695	-31.80	1	BEAM		
7	7	8	10.822	-31.80	1	BEAM		
8	8	9	0.688	-32.03	1	BEAM		Y
9	9	10	9.895	-33.19	1	BEAM		
10	10	11	10.539	-30.53	1	BEAM		
11	11	12	22.405	174.79	2	BEAM	Y	Y
12	12	13	27.500	179.92	3	BEAM	Y	Y
13	13	1	22.928	-173.77	2	BEAM	Y	Y
14	3	13	10.038	-58.71	5	BEAM	Y	
15	6	13	24.509	-123.66	4	BEAM	Y	Y
16	6	12	24.726	-55.75	4	BEAM	Y	Y
17	9	12	10.040	-119.56	5	BEAM	Y	

PROGRAM : General Frame Analysis v2.05 PAGE NO. 2
DCF ENGINEERING, Inc. TIME : Fri Jul 22 14:29:45 2005
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

P R O P E R T Y   I N F O R M A T I O N

PROP NO	SECTION NAME	MODULUS	AREA	I	DIST
	Units :	K /In 2	In2	In4	Ft
1	10"x12"	1.6e+003	120	1.44e+003	
2	(2) 1" DIA. RODS	2.8e+004	1.57	0.0982	
3	1 1/2" DIA. ROD	2.8e+004	1.77	0.248	
4	(2) 3/4" DIA. RODS	2.8e+004	0.883	0.031	
5	(4) L 3 x 3 x 3/8	2.9e+004	8.44	13.7	

N O D A L   L O A D   I N F O R M A T I O N

REC NO	LOAD CASE	LOAD TYPE	PX DX	PY DY	M BETA
	Units :		Lb Ft	Lb Ft	Ft-Lb Deg
Description : DEAD LOAD					
Node List : 2,4,5,6,7,8,10					
1	1	FORCE	0.00	-3240.00	0.00
Description : SNOW LOAD					
Node List : 2,4,5,6,7,8,10					
2	2	FORCE	0.00	-6480.00	0.00
Description : 1/2 UNBAL					
Node List : 2,4,5					
3	3	FORCE	0.00	-6480.00	0.00
Description : 1/2 UNBAL					
Node List : 7,8,10					
4	3	FORCE	0.00	-3240.00	0.00
Description : 1/2 UNBAL					
Node List : 6					
5	3	FORCE	0.00	-4860.00	0.00

PROGRAM : General Frame Analysis v2.05 PAGE NO. 3
DCF ENGINEERING, Inc. TIME : Fri Jul 22 14:29:45 2005
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

N O D A L   D I S P L A C E M E N T S

NODE NO	LOAD COMB	DX	DY	ROTATION
	Units :	In	In	Deg

LOAD COMBINATIONS:

COMB 1 : 1.00 X CASE 1

COMB 2 : 1.00 X CASE 1
+ 1.00 X CASE 2

COMB 3 : 1.00 X CASE 1
+ 1.00 X CASE 3

NODE NO	LOAD COMB	DX	DY	ROTATION
1	1	0.0000	0.0000	-0.3890
	2	0.0000	0.0000	-1.1669
	3	0.0000	0.0000	-1.1187
2	1	0.3471	-0.5809	-0.1109
	2	1.0412	-1.7426	-0.3327
	3	0.9869	-1.6469	-0.2848
3	1	0.2732	-0.4882	-0.3369
	2	0.8197	-1.4647	-1.0106
	3	0.7166	-1.2831	-0.9981
4	1	0.3009	-0.5342	-0.3327
	2	0.9027	-1.6027	-0.9982
	3	0.7988	-1.4192	-0.9857
5	1	0.5194	-0.9064	0.0426
	2	1.5581	-2.7193	0.1277
	3	1.4430	-2.5086	0.1407
6	1	0.2250	-0.4600	-0.3464
	2	0.6749	-1.3799	-1.0391
	3	0.5482	-1.1417	-0.6812
7	1	-0.0723	-0.9168	-0.0371
	2	-0.2169	-2.7505	-0.1113

PROGRAM : General Frame Analysis v2.05 PAGE NO. 4
DCF ENGINEERING, Inc. TIME : Fri Jul 22 14:29:45 2005
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

N O D A L   D I S P L A C E M E N T S

NODE NO	LOAD COMB	DX	DY	ROTATION
	3	-0.0368	-2.0309	-0.0628
8	1	0.1422	-0.5459	0.3281
	2	0.4265	-1.6377	0.9842
	3	0.4013	-1.2649	0.6671
9	1	0.1666	-0.5051	-0.2234
	2	0.4999	-1.5152	-0.6703
	3	0.4509	-1.1817	-0.4331
10	1	0.0310	-0.6888	0.1167
	2	0.0930	-2.0665	0.3502
	3	0.2030	-1.5052	0.2777
11	1	0.4199	0.0000	0.4791
	2	1.2598	0.0000	1.4373
	3	1.0505	0.0000	1.0348
12	1	0.2550	-0.5521	0.0476
	2	0.7650	-1.6562	0.1427
	3	0.6650	-1.2970	0.1157
13	1	0.1781	-0.5430	-0.0522
	2	0.5342	-1.6290	-0.1566
	3	0.4724	-1.4223	-0.1337

E L E M E N T   R E P O R T S
SIGN CONVENTION : BEAM DESIGNERS

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
	Units :		Lb	Lb	Lb-Ft	Lb-Ft /In	Ft

LOAD COMBINATIONS:

COMB 1 : 1.00 X CASE 1

COMB 2 : 1.00 X CASE 1
+ 1.00 X CASE 2

PROGRAM : General Frame Analysis v2.05 PAGE NO. 5
DCF ENGINEERING, Inc. TIME : Fri Jul 22 14:29:45 2005
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

E L E M E N T   R E P O R T S
SIGN CONVENTION : BEAM DESIGNERS

COMB 3 : 1.00 X CASE 1
+ 1.00 X CASE 3

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
1	1	1	-22986.7235	1306.8361	0.0000		
		2	-22986.7235	1306.8361	14246.1222	-0.0815	6.29
	2	1	-68960.1705	3920.5082	0.0000		
		2	-68960.1705	3920.5082	42738.3666	-0.2444	6.29
	3	1	-61636.2345	3919.2007	0.0000		
		2	-61636.2345	3919.2007	42724.1124	-0.2443	6.29
2	1	2	-21260.3494	-1443.2493	14246.1222		
		3	-21260.3494	-1443.2493	0.0000	-0.0668	4.17
	2	2	-63781.0483	-4329.7478	42738.3666		
		3	-63781.0483	-4329.7478	0.0000	-0.2003	4.17
	3	2	-56457.1124	-4328.3037	42724.1124		
		3	-56457.1124	-4328.3037	0.0000	-0.2003	4.17
3	1	3	-21339.3403	3964.8263	0.0000		
		4	-21339.3403	3964.8263	3028.4759		
	2	3	-64018.0210	11894.4788	0.0000		
		4	-64018.0210	11894.4788	9085.4277	-0.0003	0.44
	3	3	-56694.1424	11886.7781	0.0000		
		4	-56694.1424	11886.7781	9079.5457	-0.0003	0.44
4	1	4	-19619.0235	1190.5739	3028.4759		
		5	-19619.0235	1190.5739	16084.6293	-0.1091	6.08
	2	4	-58857.0706	3571.7217	9085.4277		
		5	-58857.0706	3571.7217	48253.8879	-0.3273	6.08
	3	4	-51533.1887	3575.1611	9079.5457		
		5	-51533.1887	3575.1611	48285.7232	-0.3275	6.09

PROGRAM : General Frame Analysis v2.05 PAGE NO. 6
DCF ENGINEERING, Inc. TIME : Fri Jul 22 14:29:45 2005
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

E L E M E N T   R E P O R T S
SIGN CONVENTION : BEAM DESIGNERS

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
5	1	5	-17894.0658	-1535.3881	16084.6293		
		6	-17894.0658	-1535.3881	0.0000	-0.0849	4.43
	2	5	-53682.1974	-4606.1643	48253.8879		
		6	-53682.1974	-4606.1643	0.0000	-0.2548	4.43
	3	5	-46358.3154	-4609.2032	48285.7232		
		6	-46358.3154	-4609.2032	0.0000	-0.2550	4.43
6	1	6	-17773.4612	1510.2850	0.0000		
		7	-17773.4612	1510.2850	16151.8111	-0.0889	6.17
	2	6	-53320.3837	4530.8551	0.0000		
		7	-53320.3837	4530.8551	48455.4334	-0.2666	6.17
	3	6	-42806.0953	3019.7351	0.0000		
		7	-42806.0953	3019.7351	32294.6928	-0.1777	6.17
7	1	7	-19480.8889	-1243.5278	16151.8111		
		8	-19480.8889	-1243.5278	2694.2609	-0.1049	4.79
	2	7	-58442.6667	-3730.5833	48455.4334		
		8	-58442.6667	-3730.5833	8082.7827	-0.3147	4.79
	3	7	-46220.9506	-2487.9747	32294.6928		
		8	-46220.9506	-2487.9747	5369.6459	-0.2096	4.79
8	1	8	-21203.7442	-3914.3754	2694.2609		
		9	-21203.7442	-3914.3754	0.0000		
	2	8	-63611.2326	-11743.1261	8082.7827		
		9	-63611.2326	-11743.1261	0.0000	-0.0002	0.29
	3	8	-49666.6095	-7801.3267	5369.6459		
		9	-49666.6095	-7801.3267	0.0000	-0.0001	0.29
9	1	9	-20934.9407	1940.5643	0.0000		
		10	-20934.9407	1940.5643	19201.5884	-0.0905	5.71

```
===============================================================================
PROGRAM : General Frame Analysis v2.05                              PAGE NO. 7
DCF ENGINEERING, Inc.                       TIME :  Fri Jul 22 14:29:45 2005
JOB : 05025 SHELBURNE FARMS BREEDING BARN                        JOB NO. : 165
RUN : TRUSS T1
===============================================================================
                          E L E M E N T   R E P O R T S
                                SIGN CONVENTION : BEAM DESIGNERS
```

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
	2	9	-62804.8220	5821.6928	0.0000		
		10	-62804.8220	5821.6928	57604.7653	-0.2714	5.71
	3	9	-49127.0206	4054.5834	0.0000		
		10	-49127.0206	4054.5834	40119.4864	-0.1890	5.71
10	1	10	-22648.2945	-1821.9154	19201.5884		
		11	-22648.2945	-1821.9154	0.0000	-0.1026	4.45
	2	10	-67944.8834	-5465.7463	57604.7653		
		11	-67944.8834	-5465.7463	0.0000	-0.3078	4.45
	3	10	-52553.9739	-3806.6805	40119.4864		
		11	-52553.9739	-3806.6805	0.0000	-0.2144	4.45
11	1	11	18659.8727	0.0000	0.0000		
		12	18659.8727	0.0000	0.0000		
	2	11	55979.6181	0.0000	0.0000		
		12	55979.6181	0.0000	0.0000		
	3	11	43513.8271	0.0000	0.0000		
		12	43513.8271	0.0000	0.0000		
12	1	12	11537.0677	0.0000	0.0000		
		13	11537.0677	0.0000	0.0000		
	2	12	34611.2030	0.0000	0.0000		
		13	34611.2030	0.0000	0.0000		
	3	12	28865.5195	0.0000	0.0000		
		13	28865.5195	0.0000	0.0000		
13	1	13	18869.7407	0.0000	0.0000		
		1	18869.7407	0.0000	0.0000		
	2	13	56609.2221	0.0000	0.0000		
		1	56609.2221	0.0000	0.0000		

```
===============================================================================
PROGRAM : General Frame Analysis v2.05                              PAGE NO. 8
DCF ENGINEERING, Inc.                       TIME :  Fri Jul 22 14:29:45 2005
JOB : 05025 SHELBURNE FARMS BREEDING BARN                        JOB NO. : 165
RUN : TRUSS T1
===============================================================================
                          E L E M E N T   R E P O R T S
                                SIGN CONVENTION : BEAM DESIGNERS
```

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
	3	13	50374.6136	0.0000	0.0000		
		1	50374.6136	0.0000	0.0000		
14	1	3	-5372.0885	0.0000	0.0000		
		13	-5372.0885	0.0000	0.0000		
	2	3	-16116.2654	0.0000	0.0000		
		13	-16116.2654	0.0000	0.0000		
	3	3	-16119.7053	0.0000	0.0000		
		13	-16119.7053	0.0000	0.0000		
15	1	6	7994.9650	0.0000	0.0000		
		13	7994.9650	0.0000	0.0000		
	2	6	23984.8950	0.0000	0.0000		
		13	23984.8950	0.0000	0.0000		
	3	6	23165.9642	0.0000	0.0000		
		13	23165.9642	0.0000	0.0000		
16	1	6	7753.5489	0.0000	0.0000		
		12	7753.5489	0.0000	0.0000		
	2	6	23260.6466	0.0000	0.0000		
		12	23260.6466	0.0000	0.0000		
	3	6	16177.8945	0.0000	0.0000		
		12	16177.8945	0.0000	0.0000		
17	1	9	-5435.5568	0.0000	0.0000		
		12	-5435.5568	0.0000	0.0000		
	2	9	-16306.6703	0.0000	0.0000		
		12	-16306.6703	0.0000	0.0000		
	3	9	-10870.1222	0.0000	0.0000		
		12	-10870.1222	0.0000	0.0000		

```
================================================================================
PROGRAM : General Frame Analysis v2.05                               PAGE NO. 9
DCF ENGINEERING, Inc.                         TIME :  Fri Jul 22 14:29:45 2005
JOB : 05025 SHELBURNE FARMS BREEDING BARN                       JOB NO. : 165
RUN : TRUSS T1
================================================================================
                       R  E  A  C  T  I  O  N  S
 NODE      LOAD
  NO       COMB             PX                PY             MOMENT
================================================================================
                   Units :  Lb                Lb             Lb-Ft

LOAD COMBINATIONS:

COMB  1 :  1.00 X CASE  1

COMB  2 :  1.00 X CASE  1
        +  1.00 X CASE  2

COMB  3 :  1.00 X CASE  1
        +  1.00 X CASE  3

     1         1            0.0000      11301.2449          0.0000
               2            0.0000      33903.7346          0.0000
               3            0.0000      30678.4142          0.0000

    11         1            0.0000      11378.7551          0.0000
               2            0.0000      34136.2654          0.0000
               3            0.0000      26021.5858          0.0000
```

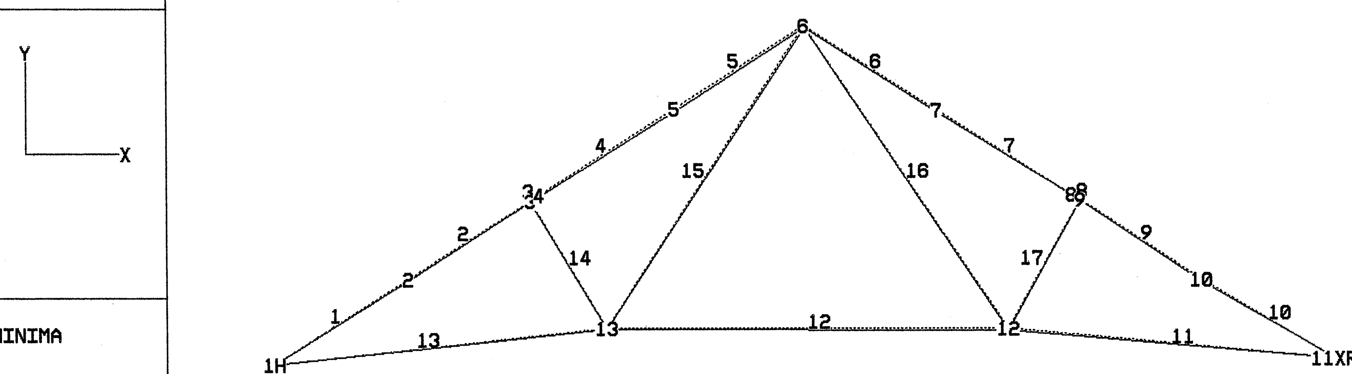
Truss Analysis: Load Case #2

FA3

DEFORMED
SHAPE
LOAD CASE #2

MINIMA
X-1.205E-002
Y-1.528E-001

MAXIMA
X 8.656E-002
Y 0.000E+000



NOTES :
JOB ID: 05025 SHELBURNE FARMS BREEDING BARN
RUN ID: TRUSS T1

Project Name: Shelburne Farms Breeding Barn 05025

Species or Species Combination	Species That May Be Included in Combination	Grading Rules Agency
Southern Pine	Loblolly Pine Longleaf Pine Shortleaf Pine Slash Pine	SPIB

Table 4–2.–Mechanical properties[1] of some commercially important woods grown in the United States

Common names of species	Specific gravity	Static Bending: Modulus of rupture *psi*	Static Bending: Modulus of elasticity *million psi*	Compression parallel to grain –maximum crushing strength *psi*
Loblolly----------	0.47	7,300	1.40	3,510
	0.51	12,800	1.79	7,130
Longleaf---------	0.54	8,500	1.59	4,320
	0.59	14,500	1.98	8,470
Shortleaf--------	0.47	7,400	1.39	3,530
	0.51	13,100	1.75	7,270
Slash------------	0.54	8,700	1.53	3,820
	0.59	16,300	1.98	8,140

[1] Results of test on small, clear straight-grained specimens. [Values in the first line for each species are from tests of green material; those in the second line are adjusted to 12 pct. Moisture content.] Specific gravity is based on weight when ovendry and volume when green or at 12 pct. moisture content.

DCF Engineering, Inc.
Calculations

Project Name: Shelburne Farms Breeding Barn
Project No. 05025

Notations Combined Bending and Axial Loading UNITY CHECK

F_c	=	tabulated compression design value parallel to grain, psi
F_c	=	$F_c C_D$
f_c	=	$\frac{P}{A}$
F_t	=	tabulated tension design value parallel to grain, psi
F_t	=	$F_t C_D$
f_t	=	$\frac{P}{A}$
F_b	=	tabulated bending design value parallel to grain, psi
F_b	=	$F_b C_d$
f_b	=	$\frac{M}{S}$
C_D	=	load duration factor
C_d	=	penetration depth factor for connections
P	=	total concentrated load or total axial load, lbs
A	=	area of cross section, in.^{2}
M	=	maximum bending moment, in.-lbs
S	=	section modulus, in.^{3}
	=	Manual Fill-in
	=	Auto Fill-in

DCF Engineering, Inc.
Calculations

Project Name: Shelburne Farms Breeding Barn Truss 10" x 12" (DL = 15 psf, LL = 30 psf)
Project No. 05025

Combined Bending and Axial Loading

Compression

$F_c = F_c C_D$ — F_c: 1100, C_D: 1.150 — F_c: 1265

$f_c = \frac{P}{A}$ — P: 67945, A: 120.000 — f_c: 566

Tension

$F_t = F_t C_D$ — F_t: 0, C_D: 0.000 — F_t: 0

$f_t = \frac{P}{A}$ — P: 0, A: 0.000 — f_t: 0

Bending

$F_b = F_b C_d$ — F_b: 1750, C_d: 1.150 — F_b: 2013

$f_b = \frac{M}{S}$ — M: 691260.00, S: 240.00 — f_b: 2880.25

UNITY CHECK

$$\frac{f_c \text{ or } f_t}{F_c \text{ or } F_t} + \frac{f_b}{F_b} = \frac{566}{1265} + \frac{2880.25}{2013} = 1.88 > 1.00 \text{ NG}$$

M in foot lbs.= 57605
691260

DCF Engineering, Inc.
Calculations

Project Name: Shelburne Farms Breeding Barn — Truss 10" x 12" (DL = 15 psf, LL = 30 psf)
Project No. 05025

Combined Bending and Axial Loading

Compression

$F_c' = F_c C_D$: F_c = 1300, C_D = 1.150 → F_c' = 1495

$f_c = \frac{P}{A}$: P = 67945, A = 120.000 → f_c = 566

Tension

$F_t' = F_t C_D$: F_t = 0, C_D = 0.000 → F_t' = 0

$f_t = \frac{P}{A}$: P = 0, A = 0.000 → f_t = 0

Bending

$F_b' = F_b C_d$: F_b = 2100, C_d = 1.150 → F_b' = 2415

$f_b = \frac{M}{S}$: M = 691260.00, S = 240.00 → f_b = 2880.25

UNITY CHECK $\frac{f_c \text{ or } f_t}{F_c' \text{ or } F_t'} + \frac{f_b}{F_b'} = \frac{566}{1495} + \frac{2880.25}{2415}$ = 1.57 > 1.00 NG

M in foot lbs.= 57605
691260

DCF Engineering, Inc.
Calculations

Project Name: Shelburne Farms Breeding Barn — Truss 10" x 12" (DL = 15 psf, LL = 30 psf)
Project No. 05025

Combined Bending and Axial Loading

Compression

$F_c' = F_c C_D$: F_c = 1782, C_D = 1.150 → F_c' = 2049

$f_c = \frac{P}{A}$: P = 67945, A = 120.000 → f_c = 566

Tension

$F_t' = F_t C_D$: F_t = 0, C_D = 0.000 → F_t' = 0

$f_t = \frac{P}{A}$: P = 0, A = 0.000 → f_t = 0

Bending

$F_b' = F_b C_d$: F_b = 3200, C_d = 1.150 → F_b' = 3680

$f_b = \frac{M}{S}$: M = 691260.00, S = 240.00 → f_b = 2880.25

UNITY CHECK $\frac{f_c \text{ or } f_t}{F_c' \text{ or } F_t'} + \frac{f_b}{F_b'} = \frac{566}{2049} + \frac{2880.25}{3680}$ = 1.06 > 1.00 Say OK

M in foot lbs.= 57605
691260

Project Name: Shelburne Farms Breeding Barn 05025

With Tie Bars

SECTION PROPERTIES

Member	Size			A	S	I
	b	x	d	In^2	In^3	In^4
	d (Dia.)			b x d	$b(d^2)$ / 6	$b(d^3)$ / 12
Top Chord 10" x 12"	10.0000	x	12.0000	120	240	1440
Strut (4) L 3"x 3"x 3/8"				8.44	4.56	13.69
Tie Bars (2) 3/4"x 3"				4.50	2.25	3.375
Rods	(2) 1.0000			1.5708	0.1964	0.0982
Rod	1.5000			1.7671	0.3313	0.2485
Rods	(2) 0.7500			0.8836	0.0828	0.0311
Rods	(2) 0.8750			1.2026	0.1315	0.0575

MATERIAL PROPERTIES

Southern Pine	E= 1,600 ksi
Wrought Iron	E= 28,000 ksi
Structural Steel	E= 29,000 ksi

ALLOWABLE DESIGN VALUES

Southern Pine	Fb = 1,750 psi
	Fc = 1,100 psi
Wrought Iron	Ft = 14,000 psi
Structural Steel	Fc = 12,576 psi (Based on L=10 ft., r = 1.27)

MEMBER FORCES

Member	Force	Stress
(2) 1" Dia. Rods	T= 19032#	f_T = 12116 psi
1 1/2" Dia. Rod	T= Deleted	f_T = 0 psi
(2) 3/4" Dia. Rods	T= 19246#	f_T = 21781 psi (high)
L Struts	C= 16308#	f_c = 1932 psi (low)
(2) Tie Bars	T= 31159#	f_T = 6924 psi (low)

DCF Engineering, Inc.
Calculations

Project Name: Shelburne Farms Breeding Barn
Project No. 05025

Notations Combined Bending and Axial Loading UNITY CHECK

F_c = tabulated compression design value parallel to grain, psi
F_c = $F_c C_D$

f_c = $\frac{P}{A}$

F_t = tabulated tension design value parallel to grain, psi
F_t = $F_t C_D$

f_t = $\frac{P}{A}$

F_b = tabulated bending design value parallel to grain, psi
F_b = $F_b C_d$

f_b = $\frac{M}{S}$

C_D = load duration factor
C_d = penetration depth factor for connections
P = total concentrated load or total axial load, lbs
A = area of cross section, $in.^2$
M = maximum bending moment, in.-lbs
S = section modulus, $in.^3$

= Manual Fill-in
= Auto Fill-in

DCF Engineering, Inc.
Calculations

Project Name: Shelburne Farms Breeding Barn — Truss 10" x 12" (DL = 15 psf, LL = 30 psf)
Project No. 05025 — With Tie Bars

Combined Bending and Axial Loading

Compression				
F_c =	$F_c C_D$	F_c = 1100, C_D = 1.150		F_c = 1265
f_c =	$\frac{P}{A}$	P = 61224, A = 120.000		f_c = 510
Tension				
F_t =	$F_t C_D$	F_t = 0, C_D = 0.000		F_t = 0
f_t =	$\frac{P}{A}$	P = 0, A = 0.000		f_t = 0
Bending				
F_b =	$F_b C_d$	F_b = 1750, C_d = 1.150		F_b = 2013
f_b =	$\frac{M}{S}$	M = 672180.00, S = 240.00		f_b = 2800.75

UNITY CHECK

$$\frac{f_c \text{ or } f_t}{F_c \text{ or } F_t} + \frac{f_b}{F_b} = \frac{510}{1265} + \frac{2800.75}{2013} = 1.79 > 1.00 \text{ NG}$$

M in foot lbs.= 56015
672180

DCF Engineering, Inc.
Calculations

Project Name: Shelburne Farms Breeding Barn — Truss 10" x 12" (DL = 15 psf, LL = 30 psf)
Project No. 05025 — With Tie Bars

Combined Bending and Axial Loading

Compression				
F_c =	$F_c C_D$	F_c = 1300, C_D = 1.150		F_c = 1495
f_c =	$\frac{P}{A}$	P = 61224, A = 120.000		f_c = 510
Tension				
F_t =	$F_t C_D$	F_t = 0, C_D = 0.000		F_t = 0
f_t =	$\frac{P}{A}$	P = 0, A = 0.000		f_t = 0
Bending				
F_b =	$F_b C_d$	F_b = 2100, C_d = 1.150		F_b = 2415
f_b =	$\frac{M}{S}$	M = 672180.00, S = 240.00		f_b = 2800.75

UNITY CHECK

$$\frac{f_c \text{ or } f_t}{F_c \text{ or } F_t} + \frac{f_b}{F_b} = \frac{510}{1495} + \frac{2800.75}{2415} = 1.50 > 1.00 \text{ NG}$$

M in foot lbs.= 56015
672180

DCF Engineering, Inc.
Calculations

Project Name: Shelburne Farms Breeding Barn — Truss 10" x 12" (DL = 15 psf, LL = 30 psf)
Project No. 05025 — With Tie Bars

Combined Bending and Axial Loading

Compression

$F_c' = F_c C_D$

F_c	C_D	F_c'
1782	1.150	2049

$f_c = \frac{P}{A}$

P	A	f_c
61224	120.000	510

Tension

$F_t' = F_t C_D$

F_t	C_D	F_t'
0	0.000	0

$f_t = \frac{P}{A}$

P	A	f_t
0	0.000	0

Bending

$F_b' = F_b C_d$

F_b	C_d	F_b'
3200	1.150	3680

$f_b = \frac{M}{S}$

M	S	f_b
672180.00	240.00	2800.75

UNITY CHECK

$$\frac{f_c \text{ or } f_t}{F_c' \text{ or } F_t'} + \frac{f_b}{F_b'} = \frac{510}{2049} + \frac{2800.75}{3680} = 1.01$$

1.01 > 1.00 Say OK

M in foot lbs.= 56015
672180

PROGRAM : General Frame Analysis v2.05 PAGE NO. 1
DCF ENGINEERING, Inc. TIME : Wed Oct 24 16:01:41 2007
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

N O D A L   I N F O R M A T I O N

NODE NO	NODAL COORDINATES X	Y	SUPPORT CONDITIONS CODE	PX STIFF	PY STIFF	M STIFF
Units :	Ft	Ft		K /In	K /In	K -In /Deg
1	-36.432	0.000	H			
2	-27.203	5.615				
3	-20.719	9.620				
4	-16.724	12.068				
5	-8.906	16.833				
6	0.000	22.307				
7	8.906	16.833				
8	16.724	12.068				
9	20.718	9.620				
10	27.203	5.615				
11	36.432	0.000	XR	750		
12	13.734	2.297				
13	-13.734	2.297				
14	-18.766	10.818				
15	18.766	10.818				

E L E M E N T   I N F O R M A T I O N

ELEM NO	NE NODE	PE NODE	ELEM LENGTH	BETA ANGLE	PROP TYPE	ELEM TYPE	NE HINGE	PE HINGE
Units :			Ft	Deg				
1	1	2	10.803	31.31	1	BEAM		
2	2	3	7.622	31.70	1	BEAM		
3	3	4	4.685	31.50	1	BEAM		
4	4	5	9.156	31.37	1	BEAM		
5	5	6	10.454	31.58	1	BEAM		Y
6	6	7	10.454	-31.58	1	BEAM		
7	7	8	9.156	-31.37	1	BEAM		
8	8	9	4.684	-31.51	1	BEAM		Y
9	9	10	7.623	-31.70	1	BEAM		
10	10	11	10.803	-31.31	1	BEAM		
11	11	12	22.814	174.22	2	BEAM	Y	Y
12	12	13	27.469	180.00	3	BEAM	Y	Y
13	13	1	22.814	-174.22	2	BEAM	Y	Y
14	14	13	9.895	-59.44	5	BEAM		
15	6	13	24.270	-124.46	4	BEAM	Y	Y
16	6	12	24.270	-55.54	4	BEAM	Y	Y
17	15	12	9.895	-120.56	5	BEAM		
18	1	11	72.865	0.00	6	BEAM	Y	Y

PROGRAM : General Frame Analysis v2.05 PAGE NO. 2
DCF ENGINEERING, Inc. TIME : Wed Oct 24 16:01:46 2007
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

E L E M E N T   I N F O R M A T I O N

ELEM NO	NE NODE	PE NODE	ELEM LENGTH	BETA ANGLE	PROP TYPE	ELEM TYPE	NE HINGE	PE HINGE
19	3	14	2.291	31.52	7	BEAM	Y	
20	14	4	2.394	31.48	7	BEAM		Y
21	15	9	2.290	-31.54	7	BEAM		Y
22	8	15	2.394	-31.48	7	BEAM	Y	

P R O P E R T Y   I N F O R M A T I O N

PROP NO	SECTION NAME	MODULUS	AREA	I	DIST
Units :		K /In 2	In2	In4	Ft
1	10"x12"	1.6e+003	120	1.44e+003	
2	(2) 1" DIA. RODS	2.8e+004	1.57	0.0982	
3	1 1/2" DIA. ROD	2.8e+004	1.77	0.248	
4	(2) 3/4" DIA. RODS	2.8e+004	0.883	0.031	
5	(4) L 3 x 3 x 3/8	2.9e+004	8.44	13.7	
6	(2) 3/4" x 3" BAR	2.8e+004	4.5	3.38	
7	BOLSTER ANGLES	2.8e+004	34.8	9.5	

N O D A L   L O A D   I N F O R M A T I O N

REC NO	LOAD CASE	LOAD TYPE	PX DX	PY DY	M BETA
Units :			Lb Ft	Lb Ft	Ft-Lb Deg
Description : DEAD LOAD					
Node List : 2,4,5,6,7,8,10					
1	1	FORCE	0.00	-3240.00	0.00
Description : SNOW LOAD					
Node List : 2,4,5,6,7,8,10					
2	2	FORCE	0.00	-6480.00	0.00
Description : 1/2 UNBAL					
Node List : 2,4,5					
3	3	FORCE	0.00	-6480.00	0.00

```
==============================================================================
PROGRAM : General Frame Analysis v2.05                             PAGE NO. 3
DCF ENGINEERING, Inc.                       TIME :  Wed Oct 24 16:01:46 2007
JOB : 05025 SHELBURNE FARMS BREEDING BARN                       JOB NO. : 165
RUN : TRUSS T1
==============================================================================
               N O D A L   L O A D   I N F O R M A T I O N
 REC  LOAD  LOAD             PX         PY         M
  NO  CASE  TYPE             DX         DY        BETA
==============================================================================

Description : 1/2 UNBAL
Node List   : 7,8,10
     4    3 FORCE            0.00   -3240.00      0.00

Description : 1/2 UNBAL
Node List   : 6
     5    3 FORCE            0.00   -4860.00      0.00

Description : UNIT LOAD
Node List   : 2,14,5,6,15,10,7
     6    4 FORCE            0.00   -1000.00      0.00

Description : UNIT LOAD
Node List   : 14,6,15
     7    5 FORCE            0.00   -1000.00      0.00
```

```
==============================================================================
PROGRAM : General Frame Analysis v2.05                             PAGE NO. 4
DCF ENGINEERING, Inc.                       TIME :  Wed Oct 24 16:01:46 2007
JOB : 05025 SHELBURNE FARMS BREEDING BARN                       JOB NO. : 165
RUN : TRUSS T1
==============================================================================
                 N O D A L   D I S P L A C E M E N T S
 NODE     LOAD
  NO      COMB          DX              DY              ROTATION
==============================================================================
             Units :  In              In              Deg

LOAD COMBINATIONS:

COMB  1 :  1.00 X CASE  1
        +  1.00 X CASE  2

COMB  2 :  2.00 X CASE  4


      1       1       0.0000          0.0000          -0.5341
              2       0.0000          0.0000          -0.1240

      2       1       0.4073         -0.7510          -0.0633
              2       0.0941         -0.1712          -0.0107

      3       1       0.3458         -0.7035          -0.0689
              2       0.0674         -0.1386           0.0198

      4       1       0.4625         -0.8997          -0.3524
              2       0.0724         -0.1478          -0.0412

      5       1       0.6877         -1.3273           0.0985
              2       0.1122         -0.2249           0.0067

      6       1       0.0253         -0.3094          -1.0453
              2       0.0052         -0.0630          -0.1681

      7       1      -0.8667         -1.7037          -0.1770
              2      -0.1354         -0.2801          -0.0182

      8       1      -0.5104         -1.0633           0.7392
              2      -0.0763         -0.1718           0.0979

      9       1      -0.1066         -0.3990          -0.5661
              2      -0.0293         -0.0940          -0.1129
```

PROGRAM : General Frame Analysis v2.05 PAGE NO. 5
DCF ENGINEERING, Inc. TIME : Wed Oct 24 16:01:46 2007
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

N O D A L   D I S P L A C E M E N T S

NODE NO	LOAD COMB	DX	DY	ROTATION
10	1	-0.4559	-0.9149	-0.0373
	2	-0.0981	-0.1952	-0.0041
11	1	0.0534	0.0000	0.7123
	2	0.0109	0.0000	0.1502
12	1	-0.0174	-0.6043	-0.2022
	2	-0.0047	-0.1260	-0.0125
13	1	0.0954	-0.6561	-0.0403
	2	0.0191	-0.1336	-0.0230
14	1	0.2617	-0.5689	-0.1908
	2	0.0531	-0.1158	-0.0094
15	1	-0.1786	-0.5187	0.6682
	2	-0.0379	-0.1085	0.0794
22	1	0.0000	0.0000	0.0000
	2	0.0000	0.0000	0.0000

E L E M E N T   R E P O R T S

SIGN CONVENTION : BEAM DESIGNERS

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
		Units :	Lb	Lb	Lb-Ft	Lb-Ft /In	Ft

LOAD COMBINATIONS:

COMB 1 : 1.00 X CASE 1
+ 1.00 X CASE 2

COMB 2 : 2.00 X CASE 4

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
1	1	1	-62652.4314	2252.9777	0.0000		
		2	-62652.4314	2252.9777	24338.6111	-0.1367	6.24

PROGRAM : General Frame Analysis v2.05 PAGE NO. 6
DCF ENGINEERING, Inc. TIME : Wed Oct 24 16:01:46 2007
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

E L E M E N T   R E P O R T S

SIGN CONVENTION : BEAM DESIGNERS

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
	2	1	-12750.3608	542.6723	0.0000		
		2	-12750.3608	542.6723	5862.4146	-0.0329	6.24
2	1	2	-57558.3396	-6440.9666	24338.6111		
		3	-57558.3396	-6440.9666	-24751.9798	0.0179	5.98
	2	2	-11702.7326	-1245.2517	5862.4146		
		3	-11702.7326	-1245.2517	-3628.4189	-0.0080	2.33
3	1	3	-10115.6285	3353.3503	-24751.9798		
		4	-10115.6285	3353.3503	-9041.3128	0.0350	2.16
	2	3	-1914.2995	-3.5925	-3628.4189		
		4	-1914.2995	-3.5925	-3645.2500	0.0075	2.34
4	1	4	-52956.3329	4978.9783	-9041.3128		
		5	-52956.3329	4978.9783	36544.8639	-0.1154	5.64
	2	4	-10711.7050	1115.1421	-3645.2500		
		5	-10711.7050	1115.1421	6564.6891	-0.0142	6.22
5	1	5	-47884.5566	-3495.7662	36544.8639		
		6	-47884.5566	-3495.7662	0.0000	-0.1922	4.42
	2	5	-9668.4626	-627.9574	6564.6891		
		6	-9668.4626	-627.9574	0.0000	-0.0345	4.42
6	1	6	-45765.1915	4437.5948	0.0000		
		7	-45765.1915	4437.5948	46390.7726	-0.2439	6.04
	2	6	-9357.9277	765.9566	0.0000		
		7	-9357.9277	765.9566	8007.3373	-0.0421	6.04
7	1	7	-50840.4283	-4029.4011	46390.7726		
		8	-50840.4283	-4029.4011	9498.6670	-0.2222	4.09
	2	7	-10401.6772	-976.0076	8007.3373		
		8	-10401.6772	-976.0076	-928.7238	-0.0290	3.72

PROGRAM : General Frame Analysis v2.05 PAGE NO. 7
DCF ENGINEERING, Inc. TIME : Wed Oct 24 16:01:46 2007
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

E L E M E N T   R E P O R T S

SIGN CONVENTION : BEAM DESIGNERS

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
8	1	8	-9818.9702	-2027.8410	9498.6670		
		9	-9818.9702	-2027.8410	0.0000	-0.0100	1.98
	2	8	-1865.3615	198.2704	-928.7238		
		9	-1865.3615	198.2704	0.0000	0.0010	1.98
9	1	9	-54775.2125	5083.7754	0.0000		
		10	-54775.2125	5083.7754	38751.3552	-0.1083	4.40
	2	9	-11294.9546	1046.1398	0.0000		
		10	-11294.9546	1046.1398	7974.2580	-0.0223	4.40
10	1	10	-59859.8213	-3587.1373	38751.3552		
		11	-59859.8213	-3587.1373	0.0000	-0.2176	4.57
	2	10	-12341.1705	-738.1615	7974.2580		
		11	-12341.1705	-738.1615	0.0000	-0.0448	4.57
11	1	11	1546.4148	0.0000	0.0000		
		12	1546.4148	0.0000	0.0000		
	2	11	444.3861	0.0000	0.0000		
		12	444.3861	0.0000	0.0000		
12	1	12	-16941.1413	0.0000	0.0000		
		13	-16941.1413	0.0000	0.0000		
	2	12	-3573.0586	0.0000	0.0000		
		13	-3573.0586	0.0000	0.0000		
13	1	13	4641.3547	0.0000	0.0000		
		1	4641.3547	0.0000	0.0000		
	2	13	897.8756	0.0000	0.0000		
		1	897.8756	0.0000	0.0000		

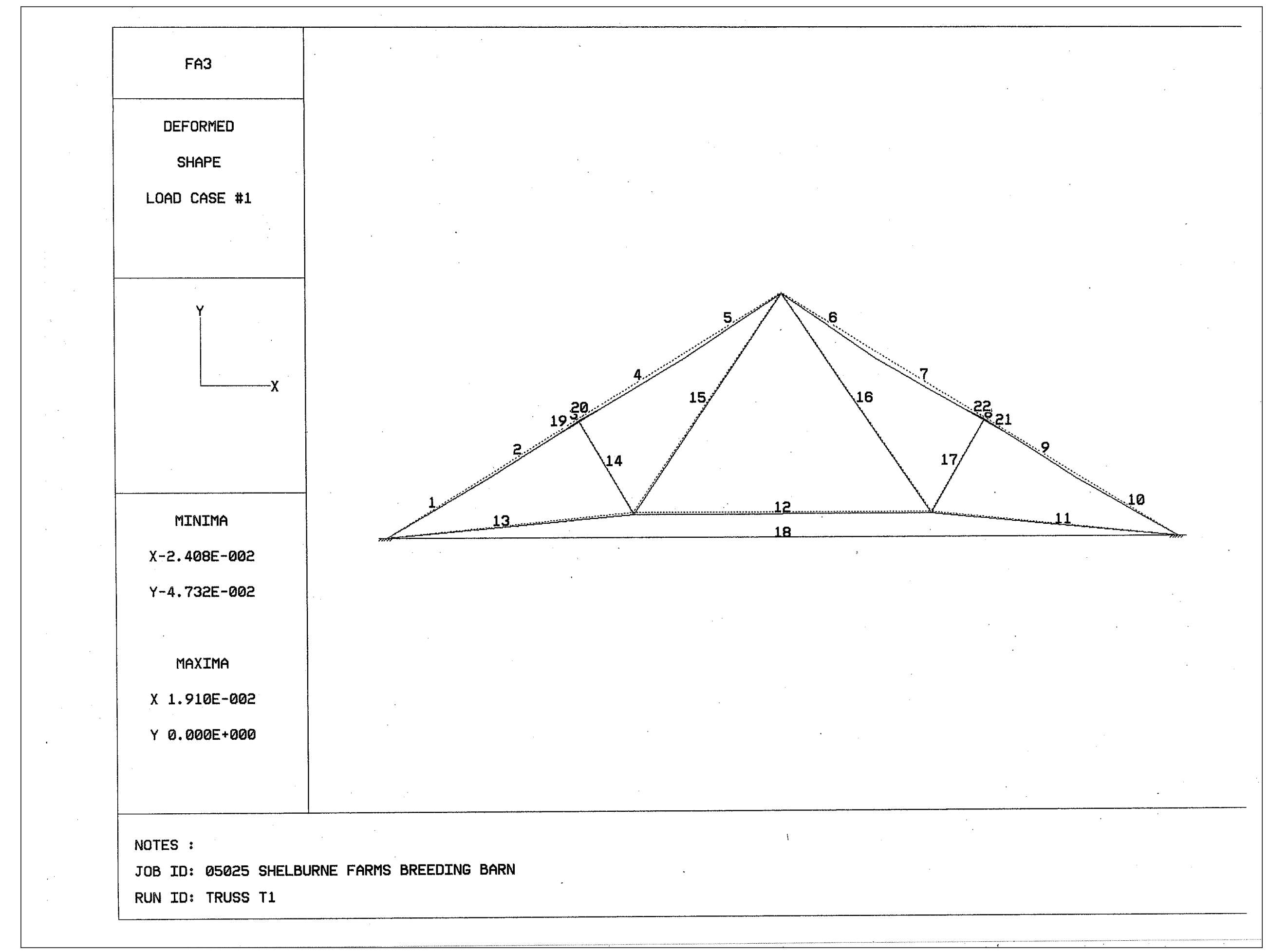
PROGRAM : General Frame Analysis v2.05 PAGE NO. 8
DCF ENGINEERING, Inc. TIME : Wed Oct 24 16:01:46 2007
JOB : 05025 SHELBURNE FARMS BREEDING BARN JOB NO. : 165
RUN : TRUSS T1

E L E M E N T   R E P O R T S

SIGN CONVENTION : BEAM DESIGNERS

ELEM NO	LOAD COMB	NODE NO	AXIAL	SHEAR	MOMENT	MAX MOM/DEFL	DIST
14	1	14	-19385.6545	-147.8481	1463.0015		
		13	-19385.6545	-147.8481	0.0000	-0.0400	4.18
	2	14	-3999.6396	13.3805	-132.4041		
		13	-3999.6396	13.3805	0.0000	0.0036	4.18
15	1	6	20904.5451	0.0000	0.0000		
		13	20904.5451	0.0000	0.0000		
	2	6	4278.6602	0.0000	0.0000		
		13	4278.6602	0.0000	0.0000		
16	1	6	18585.3321	0.0000	0.0000		
		12	18585.3321	0.0000	0.0000		
	2	6	3938.8430	0.0000	0.0000		
		12	3938.8430	0.0000	0.0000		
17	1	15	-17109.1210	855.4368	-8464.8056		
		12	-17109.1210	855.4368	0.0000	0.2314	4.18
	2	15	-3666.0714	90.3071	-893.6160		
		12	-3666.0714	90.3071	0.0000	0.0244	4.18
18	1	1	7693.8019	0.0000	0.0000		
		11	7693.8019	0.0000	0.0000		
	2	1	1566.1965	0.0000	0.0000		
		11	1566.1965	0.0000	0.0000		
19	1	3	-47461.3445	-9609.3390	0.0000		
		14	-47461.3445	-9609.3390	-22016.8473	0.0482	1.32
	2	3	-9792.2913	-1204.1064	0.0000		
		14	-9792.2913	-1204.1064	-2758.8399	0.0060	1.32
20	1	14	-47927.0285	9808.3079	-23479.8488		
		4	-47927.0285	9808.3079	0.0000	0.0561	1.01

```
================================================================================
PROGRAM : General Frame Analysis v2.05                              PAGE NO. 9
DCF ENGINEERING, Inc.                        TIME :  Wed Oct 24 16:01:46 2007
JOB : 05025 SHELBURNE FARMS BREEDING BARN                        JOB NO. : 165
RUN : TRUSS T1
================================================================================
                         E L E M E N T   R E P O R T S
 ELEM    LOAD  NODE              SIGN CONVENTION : BEAM DESIGNERS
  NO     COMB   NO       AXIAL        SHEAR       MOMENT     MAX  MOM/DEFL  DIST
================================================================================

          2     14    -8799.5553    1097.1489   -2626.4358
                 4    -8799.5553    1097.1489       0.0000        0.0063    1.01

  21      1     15   -44969.2803    6952.1642  -15922.2295
                 9   -44969.2803    6952.1642       0.0000        0.0348    0.97

          2     15    -9432.5991     815.0733   -1866.7258
                 9    -9432.5991     815.0733       0.0000        0.0041    0.97

  22      1      8   -46105.6833  -10187.2695       0.0000
                15   -46105.6833  -10187.2695  -24387.0351        0.0582    1.38

          2      8    -8538.0969   -1153.0859       0.0000
                15    -8538.0969   -1153.0859   -2760.3418        0.0066    1.38

================================================================================
                        R E A C T I O N S
 NODE      LOAD
  NO       COMB            PX                PY             MOMENT
================================================================================
                Units :  Lb                Lb               Lb-Ft

LOAD COMBINATIONS:

COMB  1 :  1.00 X CASE  1
        +  1.00 X CASE  2

COMB  2 :  2.00 X CASE  4

    1         1       40043.2712        34020.0000         0.0000
              2        8151.4489         7000.0000         0.0000


   11         1      -40043.2712        34020.0000         0.0000
              2       -8151.4489         7000.0000         0.0000
```

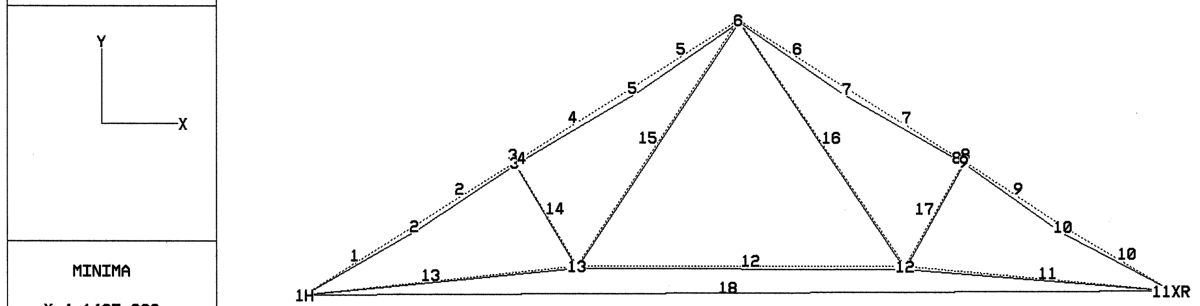


FA3

DEFORMED

SHAPE

LOAD CASE #2



MINIMA

X-4.146E-002

Y-1.080E-001

MAXIMA

X 5.904E-002

Y 0.000E+000

NOTES :

JOB ID: 05025 SHELBURNE FARMS BREEDING BARN

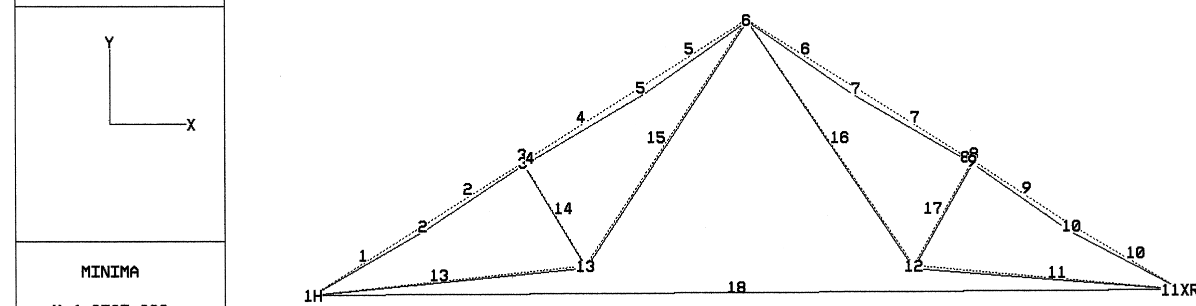
RUN ID: TRUSS T1

FA3

DEFORMED

SHAPE

LOAD CASE #2



MINIMA

X-4.373E-002

Y-1.087E-001

MAXIMA

X 5.934E-002

Y 0.000E+000

NOTES :

JOB ID: 05025 SHELBURNE FARMS BREEDING BARN

RUN ID: TRUSS T1

Load Scenario 2

STRAIN MEASUREMENT		
MEMBER	TEST	CALCULATION
11	5040 T	4300 T
12	140 T	0
13	3780 T	4719 T
14	no value recorded	3970 C
15	2928 T	4718 T
16	28 T	4407 T
17	no value recorded	3665 C
18	1260 T	1050 T

DCF Engineering, Inc.
PO Box 879 ▪ Cary, NC 27512-0879
(919) 467-3853 ▪ Fax: (919) 469-4901

05025
JULY 18, 2010

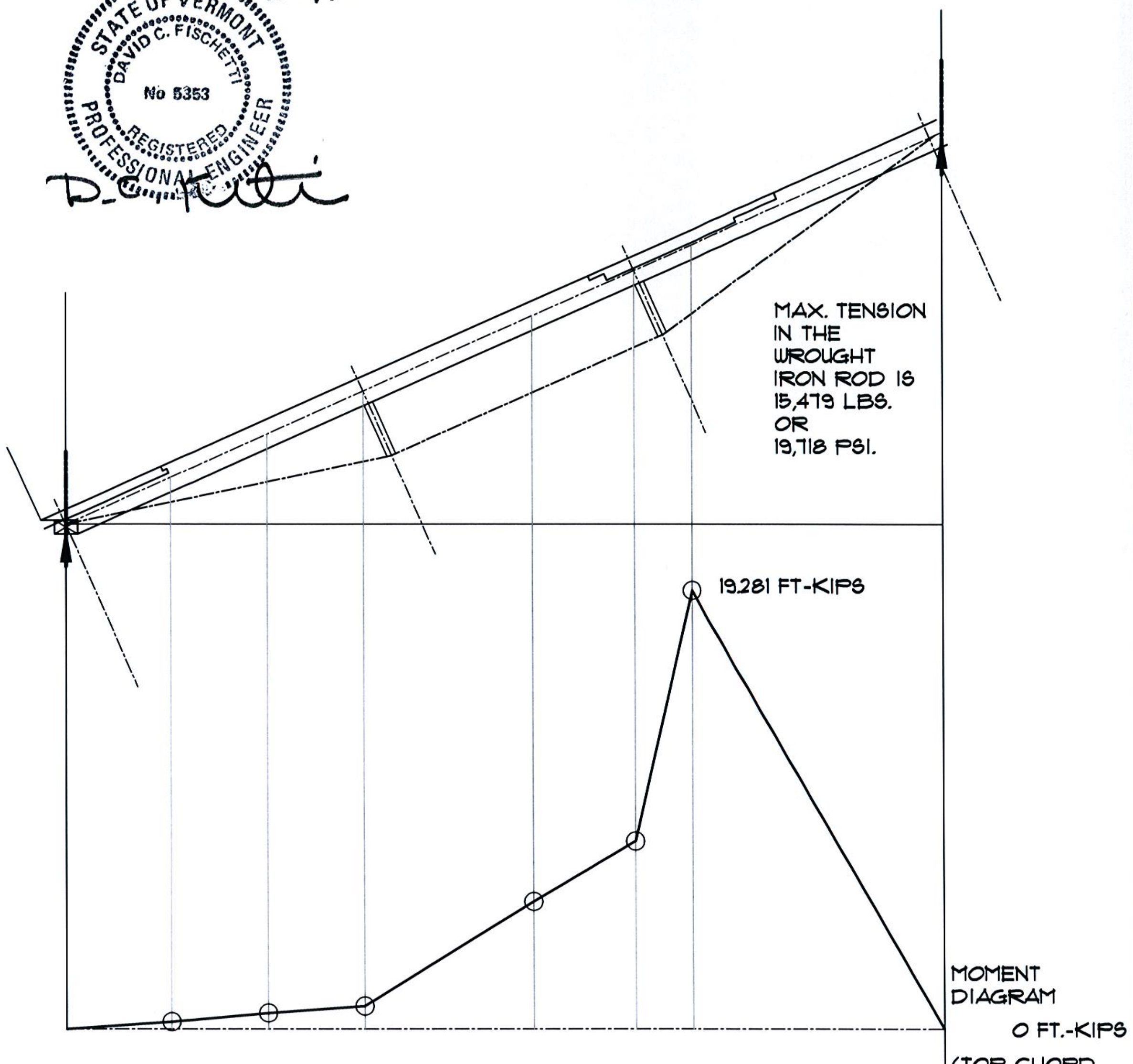
SHELBURNE FARMS RING BARN

QUEENPOST VALLEY MEMBER 11S
MOMENT DIAGRAM

STATE OF VERMONT
DAVID C. FISCHETTI
No 5353
REGISTERED
PROFESSIONAL ENGINEER

7/18/10

D.C. Fischetti



MOMENT DIAGRAM RAFTER 11S
NOT TO SCALE

DCF Engineering, Inc.
PO Box 879 ▪ Cary, NC 27512-0879
(919) 467-3853 ▪ Fax: (919) 469-4901

05025
JULY 18, 2010

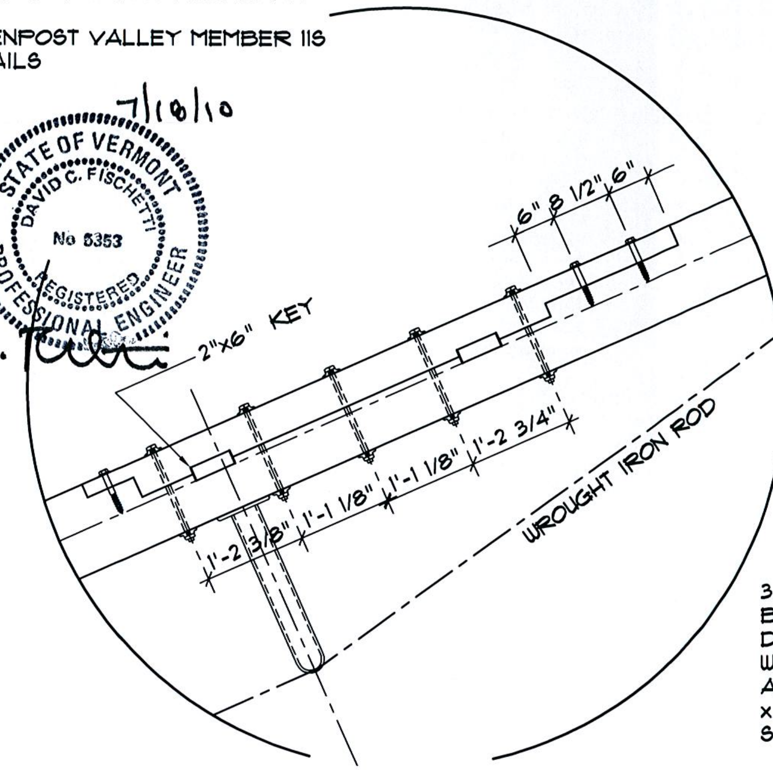
SHELBURNE FARMS RING BARN

QUEENPOST VALLEY MEMBER 11S
DETAILS

STATE OF VERMONT
DAVID C. FISCHETTI
No 5353
REGISTERED
PROFESSIONAL ENGINEER

7/18/10

D.C. Fischetti



3/4' DIA.
BOLTS W/ 3"
DIA. OGEE
WASHERS
AND 3/4" DIA.
x 6" LAG
SCREWS.

DETAIL @ RAFTER 11S
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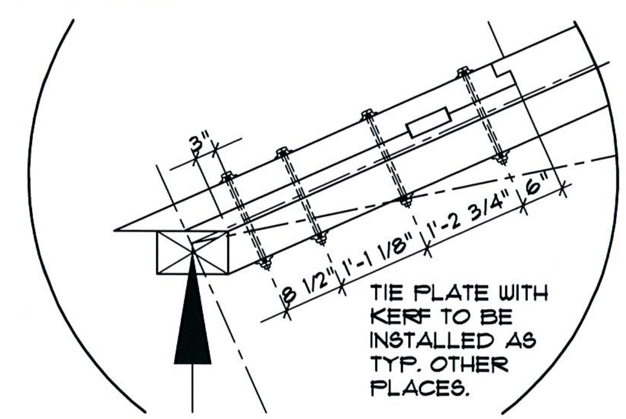

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Truss Member Properties: Calculated Axial Loads

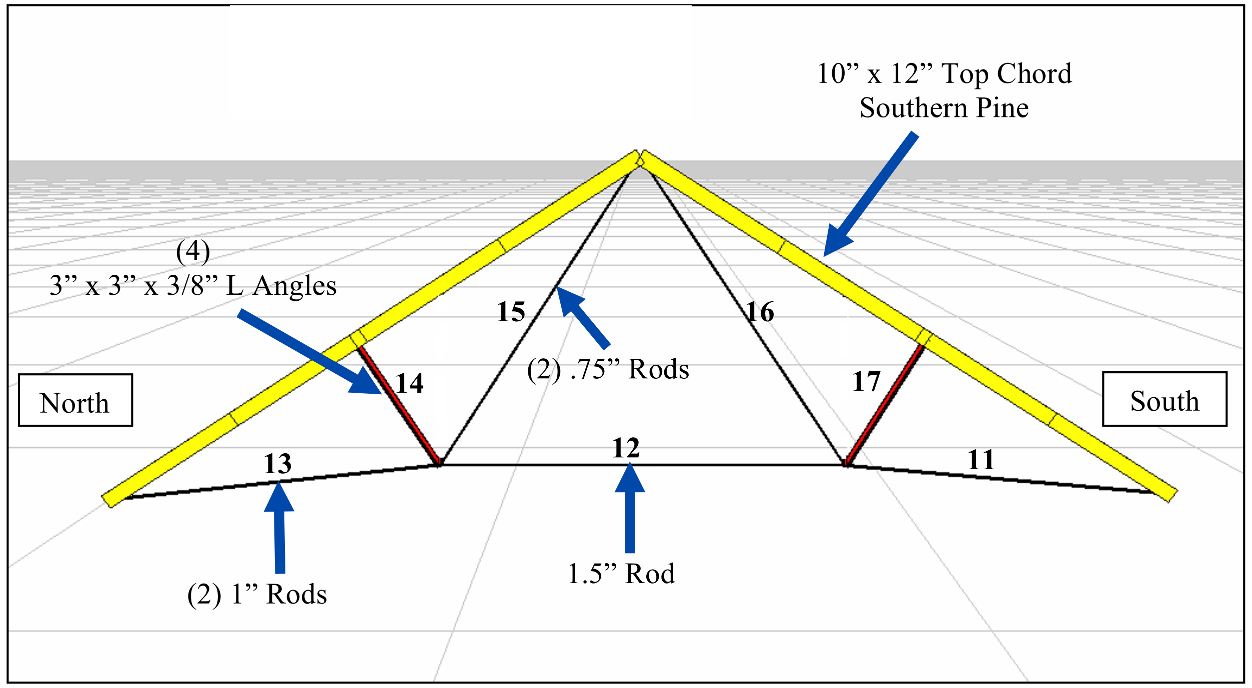

Figure 1.3: Layout of principal truss used throughout most of the Breeding Barn

Table 1.1: Specifications of members used in principal trusses

Member #	Member Type	Material	Size: Quantity in ( )
1 thru 10	Top Chord	Southern Pine	10" x 12"
11	Tensile Rod	Wrought Iron	(2) 1" Diam.
12	Tensile Rod	Wrought Iron	1.5" Diam.
13	Tensile Rod	Wrought Iron	(2) 1" Diam.
14	Compression Strut	Wrought Iron	(4) 3"x3"x3/4" Angle L
15	Tensile Rod	Wrought Iron	(2) 3/4" Diam.
16	Tensile Rod	Wrought Iron	(2) 3/4" Diam.
17	Compression Strut	Wrought Iron	(4) 3"x3"x3/4" Angle L

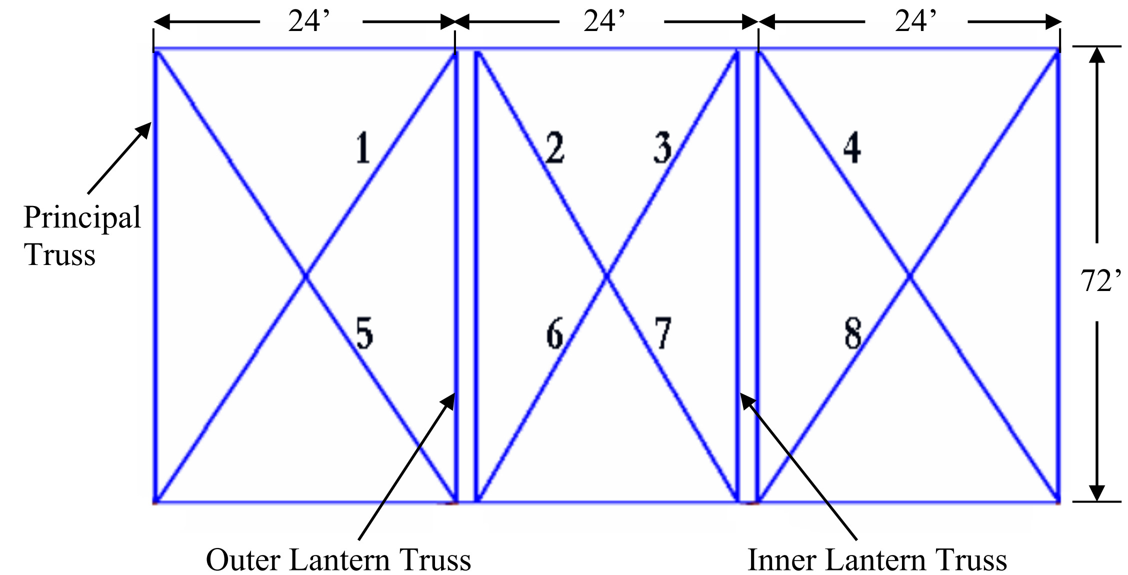

Figure 1.6: Layout and numbering scheme of the x-braces in the lantern area

Table 5.7: Axial loads predicted by 2-D and 3-D models for principal truss #11 compared to experimental loads measured during dead load conditions.

Member #	Principal Truss #11 Average Axial Loads (lbs)			
	Matlab 2D	Visual Analysis 2D	Visual Analysis 3D	Experimental
11 E,W	6,250	6,200	1,160	5,000
12	7,660	7,650	-1,820	-
13 E,W	6,320	6,300	1,175	4,750
15 E,W	2,810	2,910	2,235	4,500
16 E,W	2,720	2,805	2,265	3,750

Table 5.8: Axial loads predicted by 3-D models for principal truss #11 compared to experimental loads measured during dead load conditions.

Member #	X-Brace Member Axial Loads (lb)	
	Experimental	Visual Analysis 3D
1	9,000	5,540
2	10,500	7,500
3	10,000	7,520
4	8,500	5,570
5	9,500	5,540
6	9,400	7,500
7	11,500	7,520
8	8,300	5,570

Table 5.15: Factors of safety for principal truss #11 members predicted by adding the experimental dead loads to the 2-D snow model results

Member #	Principal Truss Axial Load (lbs)				Yield FOS
	Experimental	VA 2D	Superposition 2D	Yield	
11 E,W	5,000	16,450	21,450	24,350	**1.14**
12	-	20,400	-	54,780	**-**
13 E,W	4,750	16,700	21,450	24,350	**1.14**
15 E,W	4,500	7,400	11,900	13,695	**1.15**
16 E,W	3,750	7,100	10,850	13,695	**1.26**

Table 5.16: Factors of safety for principal truss #11 members predicted by adding the experimental dead loads to the 3-D snow model results

Member #	Principal Truss Axial Load (lbs)				Yield FOS
	Experimental	VA 3D	Superposition 3D	Yield	
11 E,W	5,000	3,845	8,845	24,350	**2.75**
12	-	-3,200	-	54,780	**-**
13 E,W	4,750	3,910	8,660	24,350	**2.81**
15 E,W	4,500	5,700	10,200	13,695	**1.34**
16 E,W	3,750	5,750	9,500	13,695	**1.44**

Table 5.17: Factors of safety for principal truss #11 members predicted by adding the experimental dead loads to the 1-side loaded 3-D snow model results

Member #	Principal Truss Axial Load (lbs)				Yield FOS (1 sided loading)
	Experimental	VA 3D (1-sided loading)	Superposition 3D (1-sided loading)	Yield	
11 E,W	5,000	4,965	9,965	24,350	**2.44**
12	-	-1,100	-	54,780	**-**
13 E,W	4,750	-600	4,150	24,350	**5.87**
15 E,W	4,500	-85	4,415	13,695	**3.10**
16 E,W	3,750	5,900	9,650	13,695	**1.42**

Table 5.18: Factors of safety for the x-brace members predicted by adding the experimental dead loads to the 3-D snow model results

Member #	X-Brace Member Axial Loads (lb)				Yield FOS
	Experimental	VA 3D	Superposition 3D	Yield	
1	9,000	10,800	19,800	69,750	**3.52**
2	10,500	10,200	20,700	69,750	**3.37**
3	10,000	10,200	20,200	69,750	**3.45**
4	8,500	10,700	19,200	69,750	**3.63**
5	9,500	10,800	20,300	69,750	**3.44**
6	9,400	10,200	19,600	69,750	**3.56**
7	11,500	10,200	21,700	69,750	**3.21**
8	8,300	10,700	19,000	69,750	**3.67**

Appendix K: Publications

Structural repair of the Breeding Barn at Shelburne Farms has been a model project in terms of the building investigation, the structural design work, and the repair techniques employed. The use of nondestructive evaluation and testing (resistance drilling of decayed timber, measurement of axial loads in iron truss elements, load testing), materials testing (metallurgical analysis, strength-in-tension tests, and repair mockups and testing), modeling and analysis (replication of the original design process using graphical analyses, laser scanning to measure deflections, and computer modeling) and a conservative approach to repairs (segmental infill using laminated veneer lumber dutchman) make this project invaluable to any discussion of repairing and extending the service life of a historic timber structure using best practices.

The following papers discussing the project's investigation and treatment methods have been published and are included here:

Porter, D., Fischetti, D. "On acceptable levels of safety in the Breeding Barn at Shelburne Farms", *Proceedings, V International Conference on the Structural Analysis of Historical Constructions,* New Delhi, India, 5-7 November, 2006.

Porter, D. and Anthony, R., "Development of an *in situ* repair strategy for the timber roof of the Breeding Barn at Shelburne Farms", SHATIS '11, *International Conference on Structural Health of Timber Buildings,* Lisbon, Portugal, 16-17 June, 2011 (paper to appear in conference proceedings).

Fischetti, D., Porter, D., Anthony, R. "Assessment and repair of the Breeding Barn at Shelburne Farms", *Proceedings, 16th ICOMOS International Wood Committee Conference and Symposium,* Collegio degli Ingegneri della Toscana, Florence, Italy.

Structural Analysis of Historical Constructions, New Delhi 2006 1
P.B. Lourenço, P. Roca, C. Modena, S. Agrawal (Eds.), ISBN 972-8692-27-7

On acceptable levels of safety in the Breeding Barn at Shelburne Farms

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ABSTRACT: The Breeding Barn, a National Landmark, is a monumental example of the estate architecture that appeared in North America near the end of the nineteenth century. The building features an enormous riding ring spanned by composite trusses of wood and iron. Engineers have called attention to overstresses in roof-frame members. A multidisciplinary design team conducted survey work in October 2005, employing a combination of non-destructive and quasi-nondestructive technologies, in order to determine rational design values for structural elements, reduce factors of safety to reasonable levels, and quantify overstresses through modeling and analysis. Results of resistance drilling of deteriorated wooden elements, strength testing of iron samples, and data produced by 3-D laser scanning will be used to complete the analysis.

1 HISTORICAL BACKGROUND

Shelburne Farms, originally the agricultural estate of William Seward and Lila Vanderbilt Webb, is a 1400-acre National Historic Landmark District located on the eastern edge of Lake Champlain in Vermont, USA. The property is owned and operated by a nonprofit organization devoted to the cultivation of a conservation ethic through education and the stewardship of natural and agricultural resources.

The Webbs developed the estate between 1886 and 1902, as part of a grand experiment to develop innovative new approaches to land use and farming. Early in the process of acquiring land for the farm, W. Seward and Lila Webb consulted with celebrated landscape architect Frederick Law Olmsted, Sr. (1822-1903), to develop a landscape design for what would ultimately be a 3800-acre estate. In his c.1887 design, Olmsted proposed a plan dividing the property into farmland, forest, and parkland, combining the pastoral and picturesque in the tradition of the great “ornamental farms” of nineteenth-century Europe.

The estate architecture was designed by New York architect Robert Henderson Robertson (1849-1919), a prominent nineteenth-century designer of monumental architecture. Today Robertson is best known for his Park Row Building (1896-1899), which at 391 feet from curb to lantern tops was the tallest building in the world when it was constructed (Landau, 1996. p. 252). The buildings at Shelburne Farms represent Robertson’s most significant estate commission. In general, the buildings combine the Queen Anne and Shingle styles and are characterized by extraordinary workmanship and design. The buildings feature gabled and hipped roofs with multiple towers, dormers, and ventilators, wide overhanging eaves supported on elaborate brackets and rafter-ends, multi-textured wall surfaces covered in shingles, clapboards, and pseudo half-timbering, and foundation stonework of estate-quarried red Monkton quartzite.

Robertson worked at Shelburne Farms for twenty years, and sixteen of his buildings survive, constructed between 1886 and 1905. The buildings are arranged on the estate in clusters or groupings according to function and consistent with Olmsted’s division of the landscape into farmland, forest, and parkland. The groupings are anchored by four enormous buildings,

Figure 1: The Breeding Barn at Shelburne Farms is approximately 418 feet long; the lantern at the center of the building springs from purlin timbers approximately 55 feet above the barn floor.

centerpieces around which life and work on the model estate revolved. They include Shelburne House (1888, with significant renovations by 1900), a Tudor Revival mansion which served as the Webb’s country residence; the Farm Barn (1888-1890), which was the agricultural headquarters of the estate; the Breeding Barn (1891), which served as the center of Dr. Webb’s horse-breeding efforts; and the Coach Barn (1902), the transportation center of the estate and one of Robertson’s last major efforts.

The Breeding Barn is the principal building of the Southern Acres portion of the Farm, and was built in part to fulfill Seward Webb’s dream of breeding a line of strong and elegant draft horses especially suited to Vermont. The building was originally called the Ring Barn, named for the riding ring that occupies the largest interior space. Construction of the barn was begun in 1889 and completed in 1891. At the time, the barn was described in *Frank Leslie’s Popular Monthly* (September 1892) as “probably the largest and best-appointed building of the kind, not only in the United States, but in the world. Those who have seen it call it one of the wonders of America.”

The main block of the building is approximately 32.6 m wide by 127.4 m long, with a two-story annex centered on the rear facade. The building is largely timber-framed, supported on a redstone foundation, and clad in wooden shingles. Building elevations are dominated by the complex-sloped two acre hipped roof, with multiple dormers and enormous central lantern. The walls are punctuated by scores of multi-pane windows that admit light and ventilate the interior space. A gable-roofed arched entry is centered on the front façade.

At the center of the building, an unbroken cathedral-like space measuring approximately 21.9 m wide and 114.3 m long once housed the riding ring. Surrounded by stables, the ring was lit by glazing in the gables of six large dormers (arranged in pairs at the center and at each end of the ring) and the lantern, supported on wooden purlins 16.8 m above the floor. This central space is surrounded by framed aisles on all four sides that once housed stalls at ground level and loft space above. The annex originally housed grooming operations, a tack room, and machinery for processing oats. The level of interior finish is very high throughout most of the building (with the exception of the loft space), with wood-paneled walls, cased window and door openings, and neat chamfers on exposed frame elements.

To support the roof over the riding ring, Robertson designed a composite truss of timber and iron. Each truss has wooden (Southern yellow pine) top chords trussed with wrought iron tension members and struts; a raised bottom chord of wrought iron completes the truss form. At the lower end, principal rafters are captured in cast iron housings that also receive the ends of tension members. Housings are fastened to timber plates at principal post locations. Principal rafters support a deck of purlins and common rafters. Aisle roofs are comprised of a deck of common rafters on timber purlins supported by king- and queen-rod trusses. The firm of Post & McCord, one of the largest iron and steel fabricators in New York City (the firm was the principal steel contractor involved in the construction of the Empire State Building), supplied the iron truss elements.

Figure 2: The Breeding Barn interior exhibits an extraordinary level of interior finish. The use of iron rather than wood for truss elements except top chords helps to covey the impression of great volume in the riding ring space.

There have been at least two major structural interventions in the riding ring roof frame. As originally designed by Robertson, principal trusses were paired to support the lantern at the center of the ring; a single truss was installed to support inboard dormer framing for major dormer pairs located at the east and west ends of the ring. Sometime subsequent to original construction, but early in the history of the building, a second truss was added at inboard dormer locations to support dormer framing not carried on the end walls. A second intervention resulted in alteration of many of the valley rafters associated with the major dormer pairs. Originally, valley rafters at each of the large dormers were trussed with wrought iron tension members and cast iron struts. Sometime in the twentieth century trusses on many of the valley rafters were removed and replaced with steel channels bolted on either side of the timbers.

2 CONDITIONS ASSESSMENT

After decades of disuse and deferred maintenance, the Breeding Barn was in an advanced state of deterioration. An engineering assessment of the building in 1990 identified several areas requiring repair and called attention to potential overstresses in many of the principal roof frame elements (CEA, 1990). The assessment called for repairs of deteriorated elements but stopped short of recommending augmentation of overstressed elements because of the impacts augmentation would have on historical integrity. In 1995, stabilization measures were implemented that ultimately included partial replacement of foundation stonework with foundations of poured concrete, repair and/or replacement of some of the deteriorated structural timbers, replacement of the roof covering with standing seam copper, and installation of a fire-suppression system.

With the help of a Getty Planning Grant, Shelburne Farms was able to complete a conservation assessment for the Southern Acres buildings and landscape in 2004. The A&E team responsible for assessment of the Breeding Barn identified several areas of deterioration

unaddressed in the earlier stabilization project, and once again called attention to several potential structural deficiencies in roof frame members. Specifically, the assessment recommended repair / replacement of decayed posts and valley rafters, and design of structural reinforcement of principal rafters, wrought iron bottom chords in the principal trusses, valley rafters, and trussed purlins (Smith, 2004).

Because of the importance of the resource, it was determined that any intervention should be as conservative of historic fabric as possible, that the historic structural system should be preserved to the fullest extent possible, and that traditional repairs are preferable to introducing new technologies so long as public safety goals are met. In order to design an intervention program that meets structural goals and guarantees public safety while having the smallest possible impact on surviving fabric, the multi-disciplinary design team determined that the focus of their work would be:

- Accurate and painstaking examination of surviving fabric to discover the nature and condition of materials and connections;
- Characterization of timber and metal elements, using non-destructive and quasi non-destructive testing techniques to the fullest extent possible;
- Rational selection of design values based on the conditions survey, materials testing, and review of the original construction documents and original design methodologies;
- Reduction of factors of safety through exhaustive knowledge of the building;
- Identification of overstresses through careful modeling and analysis.
- Development of a HABS-level documentation package, to be contributed to the Library of Congress upon completion of the project.

Initial examination of the building was organized as a training workshop in partnership with the University of Vermont. Professional team members included an architectural conservator, a structural engineer specialized in the analysis of historic timber buildings, and three timber framers associated with the truss research group of the Timber Framers Guild. Student trainees were selected from the Civil Engineering and Historic Preservation programs at the University. Trainees were paired with professional team members to form sub-teams. Each sub-team was assigned a portion of the building to survey. Survey data, including information about element dimensions, species, quality, and condition, was recorded on survey forms; survey forms included drawings of each of the principal structural elements so that deterioration and damage could be graphically represented. Team members accessed roof frame elements in the riding ring using 20 m lifts. This initial survey of the building was completed in three days.

The survey indicated that more detailed examination was necessary to determine the quality and condition of several of the iron structural elements, and to determine the extent of the deterioration in several of the timber elements. The team was most concerned with the capacity of unbraced rafters in each of the major dormers and with the condition of several of the valley rafters. Valley rafters exhibited varying levels of biodeterioration, and in some cases decay had resulted in dislodging of the timbers from their original positions at rafter apexes. A metallurgist was brought in to characterize the wrought iron used in struts and tension elements. Samples were obtained from fabric that had been previously demolished. The samples are large enough

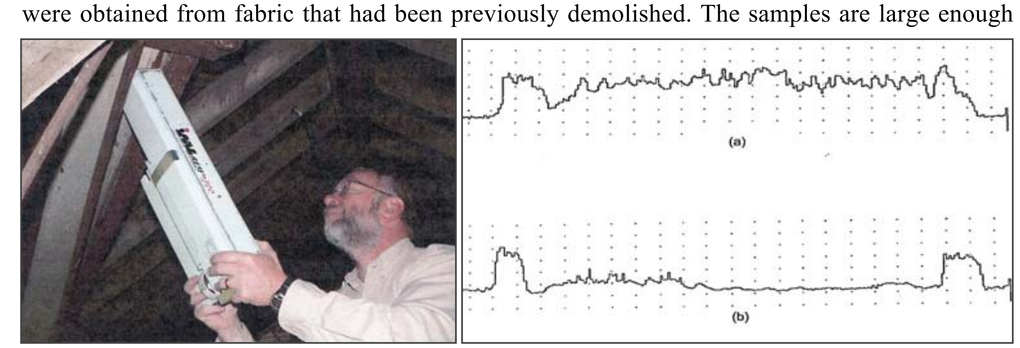

Figure 3: Resistance drilling was done to characterize the extent of deterioration in decayed frame members, left. At right, the top graph depicts sound timber (a), while the graph at bottom depicts timber with internal deterioration (Anthony, 2006).

for conducting tension tests (ASTM A 370-97a) of the iron, scheduled for May 2006; small portions of each sample were retained for metallographic characterization. Analysis indicated a low-carbon material; the closest SAE-AISI designation is 1005.

In order to quantify the extent of deterioration in decayed timber elements, a wood scientist assisted with a detailed evaluation of decayed timbers identified by the survey team. Quantification entailed resistance drilling of decayed timbers, using the IML-RESI System. Resistance drilling is a quasi-nondestructive technique for determining the relative density of wood, identifying discontinuities and quantifying the extent of section loss in the process. The process was exceptionally useful in evaluating valley rafter timbers, where installation of reinforcing steel channels (sometime in the 20th century) on either side of each timber prevented direct examination in most cases. Of the twelve timbers examined, four were found to have substantial section loss due to decay. Results of resistance drilling tests were expressed graphically and in tabular form, indicating the extent of section loss at each of the drill sites. By pinpointing areas of loss, resistance drilling will permit detailed design of timber repairs prior to dismantling the affected portions of the building, helping to reduce the number of inappropriate decisions made in the field.

3 ANALYSIS

The procedure for evaluating a building such as this is to apply today's code mandated snow, live, and wind loads to various component systems, assuming that no deterioration has occurred. In this way, the original structure can be tested with specific design load criteria, against reasonable allowable design values with the amount of overstressing tabulated for various elements.

By performing a plane frame computer analysis, the stiffness in the various components can be included, resulting in accurate theoretical deflections. The computed dead and live load deflections can then be compared to today's code mandated limits for roof structures. Once this process is completed, then a review of the amount of overstress in particular elements can be compared against reasonable values which could be expected from dense clear growth Southern Pine harvested in the late 1880s. After the structural analysis is complete, then a condition

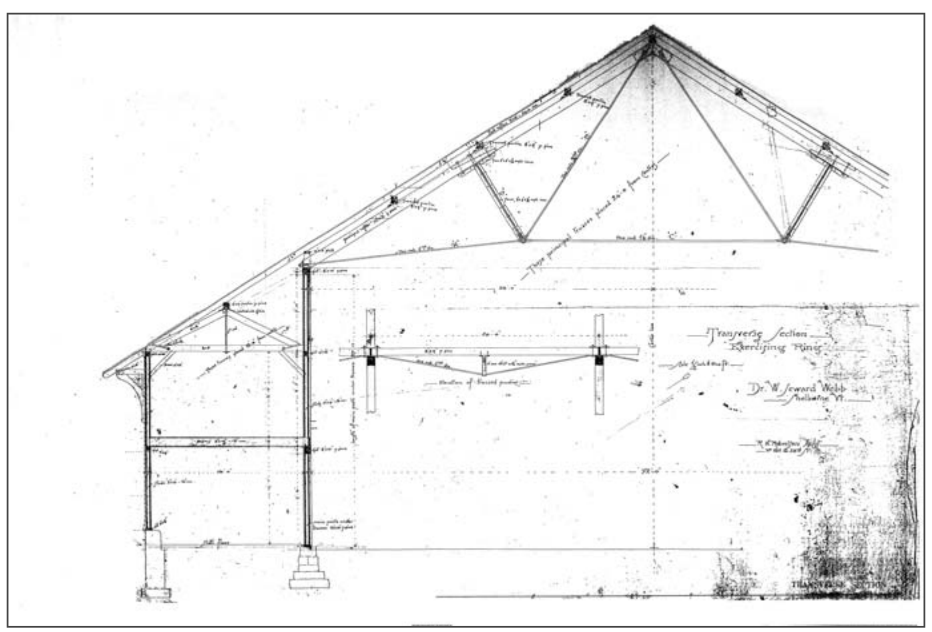

Figure 4: A portion of the transverse section drawn by Robertson and illustrating the principal truss (Collection, Shelburne Farms Archive).

analysis can be made on the basis of field observation, measurement, and testing. Through analysis and engineering judgment, the capacity of the various components can be tabulated accounting for deterioration.

The determination of an "overall safe live load capacity of the structure" reveals nothing about the various components. All of the components, including trusses, rafters, purlins, sheathing, common rafters, valley and hip beams, should be tabulated with the basis for their capacity individually noted. There should be a discussion of the modifications to the basic values for timber design, such as Load Duration Factor and Size Factor. Load Duration Factor is an interesting subject which is central to timber design but largely ignored by structural engineers reviewing historic timber structures. Since wood has the ability to sustain substantially greater loads for short periods of time, allowable design values can be increased 15% for snow load, 60% for wind, and 100% for impact. This has implications for historic structures because the application of full design load of two months for snow load, and ten minutes for wind, acting on the structure is cumulative. The question becomes "what is the cumulative amount of time that this structure has been stressed to its full allowable design value for the various loading conditions, over its history?" It may be a very difficult question to answer without accurate weather data.

The production of a set of measured drawings of the structural elements, based on the original R.H. Robertson drawings and data collected by 3-D laser scanning of the building (scheduled for June 2006), will establish the original configuration of the building "as built". As soon as sufficient verification of sizes and dimensions allows, a structural analysis can proceed. A conditional analysis would lead to suggested repair strategies. Once agreement with the owner and approving agencies and grantor is received, then specific repairs can be designed and detailed.

The structural elements of primary interest in the Breeding Barn are the principal trusses, purlins and valley rafters. The principal trusses known as cambered fan trusses, or trusses with raised bottom chords, are composite in configuration with timber top chords and wrought iron bottom chords. Cast iron fittings and steel pins and steel or wrought iron struts complete the assembly. The purlins, hip rafters, and dormer beams are trussed with wrought iron rods with cast iron posts in the queenpost fashion.

The designers will have to conduct a close review of the original plans to determine the impact on the analysis of various elements. For example, the struts in the principal trusses in the R.H. Robertson drawings are called out as "four 3" x 3" x 3/8" angle irons." The survey indicated that the sizes of actual members are different from those shown in the plans. Although there is a wealth of information in the partial set of original drawings which remain, the effort to obtain a complete set of documents from other sources should continue. As HABS-level drawings are developed, differences between the actual building "as built" and the original drawings will be documented.

The preliminary analysis of the primary truss was performed with a 1436.1 Pa snow load and 718.1 Pa dead load. The analysis indicated that there are overstresses in the top chord as well as the rods which extend from the heel supports to the queenpost struts. It is possible that the overstress in the 25.4 cm x 30.5 cm top chord is a result of the original designer analyzing the truss using graphical means (force diagram and string polygon) first developed in the 1840's by Col. Stephen H. Long. This method of analysis provides only axial member forces. It is fairly accurate for trusses where purlin loads are applied to panel points. In this case, each side of the truss has reactions from purlins applied half way between panel points. Even today, with new structures, computer analyses will provide very large bending forces that can not be determined from a graphical analysis.

The 2.5-cm diameter rods are stressed to a f_T= 248669.0 KPa. This is very high when compared to a tabulated elastic limit of 172413.8 KPa and ultimate strength in tension of 344827.6 KPa. Furthermore, the rods were measured to be actually 2.2-cm in diameter in lieu of 2.5-cm diameter, as shown in the existing drawings. This would increase f_T to 324,634.5 KPa, which is well above the elastic limit for wrought iron.

There is a system of tie bar x-bracing in the horizontal plane below the bottom of the trusses. An interesting thing happens when the two 1.9-cm by 7.6-cm tie bars are asked to carry the load. The middle bottom chord truss tie consisting of a 3.8-cm diameter rod goes into compression. Since it is so slender, this member will buckle rather than transmit compression.

For that reason, the program was re-run with the 3.8-cm tie rod deleted. With the tie bars included, the center rod deleted, the stresses are lower in the remaining wrought iron rods.

By using ultimate published values for clear wood specimens reduced by a factor of 4.0, the top chord of the truss actually checks out (6% overstress). The static bending modulus of rupture and the maximum crushing strength in compression parallel to grain for clear straight-grained specimens of Loblolly Pine , divided by a factor of safety of 4.0 will yield values of Fb=22,069.0 KPa and Fc=12,289.7 KPa.

The design team is collecting metallurgical information to establish the nature of the metal elements of the composite components. The design team will have to differentiate between cast-iron, wrought iron and rolled or forged steel elements which are present, and determine through testing and research the allowable design values for each. Although the stresses for wrought iron components are high, all but one component are close or within range of the 172,413.8 KPa to 206,896.6 KPa elastic limit published in period handbooks (Hudson, 1939).

4 CONCLUSION

Completion of project design is scheduled for February 2007. A key issue in the structural analysis is to determine the true stresses in the components of the principal trusses and solve the apparent weakness in the truss at the dormer where the top chord is unbraced by purlins. Discarding the effects of deterioration for a moment, and ignoring connection design, we can say that as long as the stresses in the timber and wrought iron materials are within the elastic limit when reasonable design loads are applied, the Breeding Barn at Shelburne Farms is not in danger of collapsing.

The overriding questions in the evaluation of this building are as follows:

- What applied snow loads are reasonable to use in the Shelburne Farms area?
- What allowable design values should be used for the timber and wrought iron?
- How has deterioration affected the capacity of the original design shown in R.H. Robertson's drawings?
- How can observation, measurement, testing and analysis be used to expose defects and deterioration critical to the safety of the structure?
- What solutions are available to ensure the continued service of the building with a level of intervention?

3-D laser scanning of the building is scheduled for June 2006. Scanning will be conducted as a partnership between Shelburne Farms and Texas Tech University. Data recovered by scanning will be used to create a point-cloud model of the barn in its current condition, and will allow the

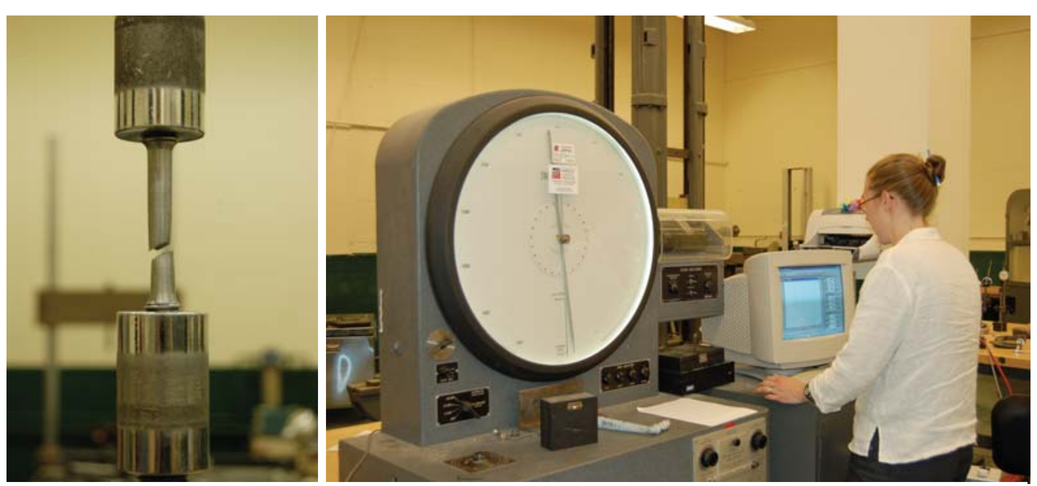

Figure 5: Tension tests (ASTM A 370-97a) were conducted on iron samples, yielding average peak stress values of approximately 319,820.7 KPa.

design team to quantify deflections in structural elements. Data collected in this manner will be archived as part of project documentation, and will provide a benchmark against which future structural movements can be measured. Scans will provide detailed information concerning as-built dimensions of individual elements, and will also be used to generate a set of HABS-level drawings of the building to be used for designing repairs. Upon conclusion of the project, drawings and accompanying large-format photography will be donated to the Library of Congress.

Depending on the results of a thorough analysis, the design team has several tools at its disposal for quantifying actual stresses in frame members and minimizing interventions while achieving safety goals. They include:

- Site-specific determination of snow- and wind-loads based on historical weather data and other observations and measurements;
- Load-testing of the frame;
- Programmatic management of risks, including closure of the building during snow season, and alarms operated by instrumentation installed on structural elements and alerting stewards to weather-related overstresses.

The Shelburne Farms Breeding Barn, despite pockets of deterioration and several design flaws, has not experienced a failure or partial failure in any of the elements which constitute the vast timber framed and wrought iron structure. With an ongoing program of observation, measurement, testing, and analysis, the building will be recorded in detail and a repair strategy formulated which will ensure the retention of the maximum amount of historic fabric through the construction effort and provide an acceptable level of safety in the restored building.

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Development of an *in situ* repair strategy for the timber roof of the Breeding Barn at Shelburne Farms

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Abstract The Breeding Barn (1891) is an enormous timber-framed structure on Shelburne Farms, a former Vanderbilt estate in Vermont, U.S.A. The structural repair of the barn, which took place in 2009-10, posed several interesting challenges. This paper presents information on the history of the Shelburne site and the Breeding Barn with particular focus on the assessment, testing, and repair decisions for the roof of the Breeding Barn. Discussion of the assessment includes a comparison of field measurements of deterioration obtained by resistance drilling during the assessment phase with the extent of actual deterioration found during the repair work. A testing program was designed to compare capacities of repaired timbers to solid control timbers to allow for evaluating dimensions and placement of structural dutchman repairs and scarfed connections to optimize the strength of the repairs implemented under field conditions.

Keywords *in situ* repairs, mechanical testing, resistance drilling

1. INTRODUCTION

Shelburne Farms, originally the agricultural estate of William Seward and Lila Vanderbilt Webb, is a 566-hectare National Historic Landmark District located on the eastern edge of Lake Champlain just south of Burlington, Vermont. The property is owned and operated by a nonprofit organization devoted to the cultivation of a conservation ethic through education and the stewardship of natural and agricultural resources.

The Webbs developed the estate between 1886 and 1902. Shelburne Farms quickly became one of the most ambitious model farm operations in the U.S., and Seward Webb one of the staunchest proponents of the role of science in agriculture. Early in the process of acquiring land for the estate, the Webbs consulted with celebrated landscape architect Frederick Law Olmsted, Sr. (1822-1903) to develop a landscape design for their growing estate. In his c.1887 design, Olmsted proposed a plan dividing the estate into farmland, forest, and parkland, combining the pastoral and picturesque in the tradition of the great "ornamental farms" of nineteenth-century Europe.

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The estate architecture was designed by New York architect Robert Henderson Robertson (1849-1919), a prominent New York architect. Robertson was an early designer of skyscrapers and is best known for his Park Row Building (1896-1899), which at 27 stories was the tallest building in New York at the time of its construction. The buildings at Shelburne Farms represent Robertson's most significant estate commission. In general, the buildings combine the Queen Anne and Shingle styles and are characterized by extraordinary design and craftsmanship.

The Breeding Barn was the architectural centerpiece of Webb's horse operation, which he intended to be one of the largest in the country, employing the most advanced concepts in animal husbandry (Figure 1). By 1891, breeding stock numbered about 219 horses, most of them English hackneys (Donnis 2010). Construction of the barn was begun in 1890 and completed in 1891 (Figure 2). At the time, the barn was said to be "probably the largest and best-appointed building of the kind, not only in the United States, but in the world. Those who have seen it call it one of the wonders of America" (Leslie 1892).

Figure 1 – The Breeding Barn at Shelburne Farms, a National Historic Landmark, is dominated by its complex-sloped 8094 m^2 hipped roof with multiple dormers and an enormous central lantern.

Figure 2 – Construction was begun in August of 1890 (left), and by December 1891 the building was nearly complete. The barn, designed by architect Robert Henderson Robertson, was the center of Seward Webb's horse breeding operation, one of the largest in the country (right).

The timber-framed building has a main block that is approximately 32.6 m wide by 127.4 m long, with a two-story annex centered on the rear facade. Building elevations are dominated by the complex-sloped 8094 m^2 hipped roof with multiple dormers and an enormous central lantern. The main interior feature is the riding ring, approximately 22 m wide and 114 m long, constructed for daily exercise of the horses (Figure 2). The roof system over the riding ring consists of a series of composite principal trusses of iron and timber, supporting a deck of timber purlins and common rafters. Trusses are supported on timber columns around the perimeter of the riding ring and bear most of the roof weight.

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The riding ring is surrounded on four sides by aisles that contained stalls on the lower level, and included space for hay storage and grain processing operations above (Figure 3).

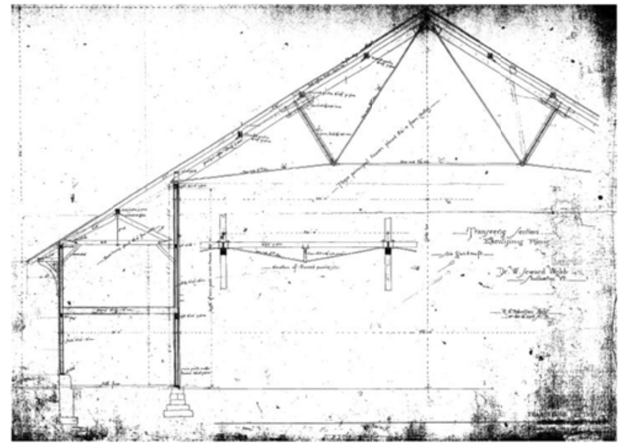

Figure 3 – Robertson's transverse section of the riding ring, showing the composite truss, trussed purlins, and aisle framing. Note the tapered scarf in each top chord member, located over the struts.

For the riding ring roof, Robertson selected a truss form commonly used in railroad construction during the latter half of the 19th century that originated with Camille Polonceau (Unwin 1869) (Figure 4). Designed for the construction of the Paris-Versailles railroad, the truss form featured an economical use of timber, room below the raised center chord, lightness, ease of assembly, and could be adjusted by tightening nuts at the heel connections (Polonceau 1839). By the time Robertson designed the barn, graphical analyses of the truss form were common in technical books on roof and bridge trusses. The truss form used in the Breeding Barn had numerous advantages over other forms, including the relatively lightweight construction for the span, the amount of light reaching the barn floor, and the incredible volume the truss form helped to add to the room. The barn trusses have southern yellow pine (*Pinus spp.*) top chords with wrought iron tension members, struts, and bottom chords. The firm of Post & McCord, one of the largest iron and steel fabricators in New York City, supplied the iron used in the construction of the Breeding Barn trusses and roof frame.

In 2001, Shelburne Farms was designated a National Historic Landmark District. The property is significant for the architecture of Robert Henderson Robertson, the landscape architecture of Frederick Law Olmsted Sr, and its associations with the Webb and Vanderbilt families.

2. BUILDING INVESTIGATION

After decades of disuse and deferred maintenance, the Breeding Barn was in an advanced state of deterioration. An engineering assessment of the building in 1990 called attention to overstress in the truss elements and identified several areas of deterioration, including decay in most of the valley members in the large dormers at the lantern and at either end of the riding ring, and in jack rafters and plate timbers in their vicinity (CEA 1990). The assessment called for repairs of deteriorated elements but stopped short of recommending augmentation of overstressed elements because of the impacts augmentation would have on historical integrity. Beginning in 1997, emergency stabilization measures were implemented that included repair or replacement of some of the structural iron and timber elements in the roof frame, replacement of the roof covering with standing seam copper, and installation of a fire suppression system.

In 2005 a project team was assembled to conduct detailed structural assessment of the building and prepare plans for its augmentation and repair. Goals of the structural investigation included determining as-built conditions and subsequent changes to the building structure and fabric, as well as current levels of deterioration. Specifically, investigators hoped to establish reasonable design values for structural ironwork, quantify section losses in decayed valley members, and address overstress in the truss elements.

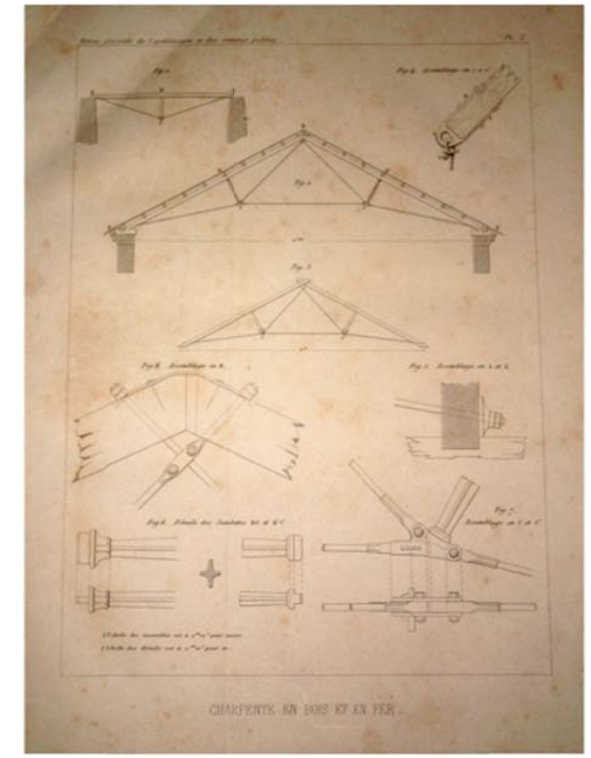

Figure 4 – Designed for the Paris-Versailles Railroad in 1837, Polonceau published a monograph on the truss form under the title "Notice sur nouveau système de charpente en bois et fer," (Revue Générale de l'Architecture et des Travaux Publics). This plate appeared in the original publication and details the adjustable connections at the ridge and rafter ends.

With the exception of angled struts and bolsters installed on top chords, dimensions of installed iron elements are the same as those specified in original construction drawings. Visual inspection confirmed that forge welds in tension elements were generally in good condition, and that heel connections were intact and in good condition. Samples were obtained from fabric that had been previously demolished and were used for metallographic characterization and for conducting tensile strength tests (Figure 5). Oriented oxide inclusions exhibited in the microstructure of the samples indicates that the lower chords and most web members are constructed of wrought iron. Chemical analysis indicates a low-carbon material; the closest SAE-AISI alloy designation is 1005. Strength-in-tension tests (ASTM A 370-97a) indicated average yield strength of about 228 MPa and a modulus of elasticity of 207 GPa, which correspond fairly closely to design values in period code and design manuals (Hudson 1939).

Deterioration of timber elements in the roof frame were the result of failure of the roof covering, particularly around the main dormer valleys. Valley members were affected, as well as columns supporting them and proximate aisle framing. Given the decay patterns, it is likely that water entering the building along main dormer valleys followed rafters to plates and column joinery below. Steel channels were installed on valley members in an earlier repair campaign, a strengthening measure that was perhaps a response to their decay. Addition of the steel channels necessitated shortening of purlins terminating at the valleys. Most of the purlins were reattached using bolted plates, except at dormer locations. Here, trusses located at the centers of each major dormer pair were left without bracing. The steel channels bolted to either side of each timber prevented direct examination of those surfaces.

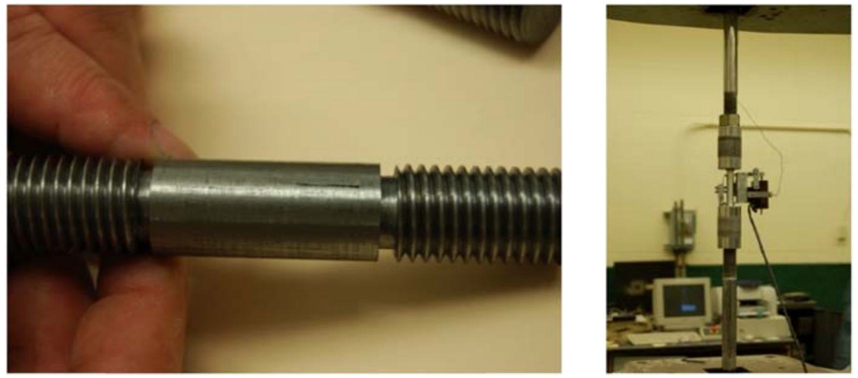

Figure 5 – Test coupons were made from iron samples collected from the barn (left), and were used to conduct strength-in-tension tests (right).

To quantify the extent of deterioration, a systematic survey was conducted using a resistance drill (IML-RESI System). Resistance drilling is a quasi-nondestructive technique for determining the relative density of wood, identifying discontinuities and quantifying the extent of section loss in the process (Figures 6, 7). The drill measures and records the torque encountered by the motor as a small-diameter needle advances into the wood; the needle does not remove material in the manner of a drill bit; rather, only a small amount of wood fiber is displaced as the needle is pushed through the wood. In most cases the drill sites are difficult to locate once the needle has been removed. The process was especially useful in evaluating valley members, where installation of reinforcing steel channels prevented direct examination. Timbers were drilled in the radial and transverse directions in order to characterize decay patterns and quantify section loss.

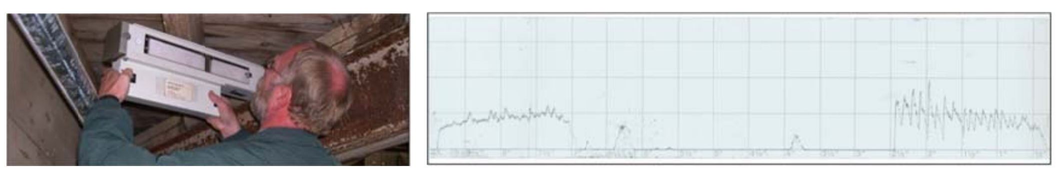

Figure 6 – The investigator is using a resistance drill to locate and measure voids in a valley member (left). Note the steel channels that prevent drilling except at the top and bottom of the member. The sample resistograph strip (right) indicates a void approximately 10.5 cm in width near the middle of the member.

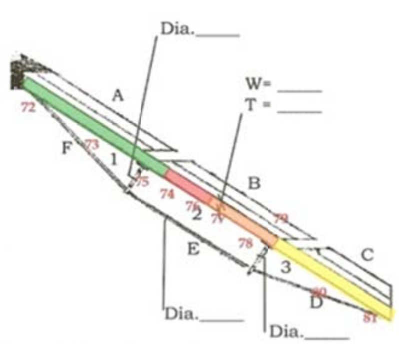

Figure 7 – Graphic presentation of resistance drill results for the valley member at B11.0. The color coding indicates various levels of damage requiring a specific level of repair as established by the project team.

Of the twelve timbers examined, five were found to have substantial section losses due to decay. Most of the losses appeared as decay channels located in the upper half of the timber section and probably the result of water leaking through the roof and finding its way into drying checks. Results of resistance drilling tests were expressed graphically and in tabular form, indicating the extent of section loss at each of the drill sites. Color-coding of the graphics allowed for easy identification of problem areas in each timber (Figure 7). By locating areas of material loss, resistance drilling permitted detailed design of timber repairs prior to dismantling the affected portions of the building, helping to reduce the number of inappropriate decisions made in the field.

The preliminary analysis of the riding ring truss was performed with a 146.5 kg/m^2 snow load and 73.2 kg/m dead load. Using published values for clear wood ultimate strength reduced by a safety factor of 4.0 (not unusual for the period of design), the top chord of the truss is overstressed slightly by approximately six percent. However, the 2.54 cm diameter iron rods at the center of the bottom chord have a tensile stress of 249 MPa. This is very high when compared to a published elastic limit of 172 MPa and ultimate tensile strength of 345 MPa. The calculations suggest that a primary issue to be resolved in the analysis of the Breeding Barn roof was to determine the path(s) of the horizontal forces in the roof. This was significant since no failure or distress was observed in the bottom chords, even though they appeared to be under considerable stress as modeled.

To determine load paths and quantify stresses in lower chord elements under load, load tests of two of the trusses were conducted. The tests employed two loading scenarios consisting of 4448 N and 8896 N (force) unit loads suspended from purlins and panel points (Figure 8).

Figure 8 – Two trusses were load tested. Loads were applied at purlins, and changes in strain were recorded. Results were compared to computer models of the truss.

Strain gauges were used to measure changes in strain in bottom chord elements. In both loading scenarios, measured strains were substantially lower than values based on the original model, lending credence to the thesis that R. H. Robertson added cross-brace ties to the basic truss configuration sometime during construction, thus reducing horizontal and vertical deflection and reducing the stresses in both the original wrought iron elements and timber top chord. The addition of 1.91 cm thick by 7.62 cm high cross-brace ties to the building, apparently during construction, provides another path for the tensile force to be resisted (Figure 9). By including the ties in the analysis of the building cross section, the center tie bar of the truss becomes a zero force member (Fischetti, et al 2007).

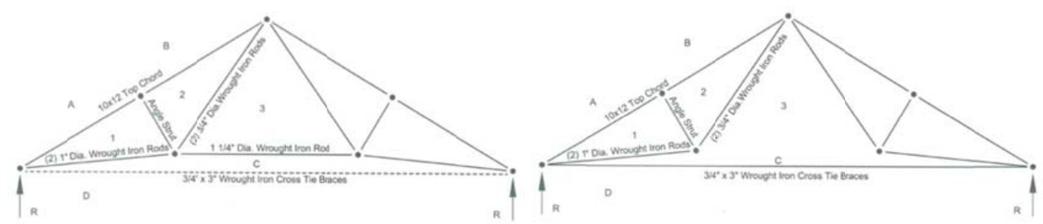

Figure 9 – Analysis of the primary truss indicated overstresses in bottom chord elements, particularly in the iron rod at the center (left). Adding the cross-brace tie, which Robertson apparently did sometime during construction, reduced stresses in iron and timber elements (right).

3. DEVELOPMENT OF A REPAIR STRATEGY AND IMPLEMENTATION OF REPAIRS

Analysis of the overall building structure indicates that it was originally very well designed, so that repair efforts were focused on conservation of deteriorated elements and reinstatement of missing elements. It was essential that the roof truss repairs be coordinated with other structural repairs to the building to ensure long-term performance of the truss repairs. Other building repairs included restoration of foundation stonework, repair of woodwork in the aisles (including roof frames, walls, and floor systems) and riding ring, and reinstatement of missing iron trusswork. Repair of the timber frame, including the roof trusses, began in October 2009 with cribbing of the northwest aisle, where concrete counter-walls poured around sills in the 1960s resulted in decay of perimeter wall woodwork. New column bases were scarfed into decayed posts, studs were sistered, and sills were replaced in kind. Sills were placed on new stone stem walls, and the wall, approximately 61 m in length, was leveled to the extent possible.

Of the 70 columns surrounding the riding ring, 60 of them were significantly decayed at the bases, partly due to installation of concrete floors in the mid-20th century, and damaged by agricultural machinery. Repairs typically included the scarfing of new column bases to replace decayed material, and the addition of sheet lead damp proofing, a limestone plinth block (matching the historic limestone piers), and white oak shims nested across the entire width of each column base. The scarf form used for most of the repairs replicated an historic form found in the building (Figure 10). By jacking three to four columns at a time, framers were able to bring aisle girts and plate timbers to a nearly level position.

Figure 10 – Columns were repaired by scarfing in new timber to replace decayed portions, using a nosed scarf form found elsewhere in the barn. New column bases were placed on limestone plinths and white oak shims, elevating them above a mid-20th century concrete floor (left). In four cases, column splices were made at the aisle level, requiring renewal of girt joinery and, in this case, replacement of the girts on either side (right).

Four of the column repairs, associated with roof leaks occurring at dormer valleys, extended into the aisle level and required installation of free tenons to support aisle floor girts. Additional repairs were made to aisle roof, wall, and floor frames at each of these locations. Roof and wall frame repairs typically consisted of scarfing in new rafter and tie beam ends, and replacement of decayed tenons with free tenons. Floor system repairs typically consisted of replacement in kind of decayed ledgers, joists, bridging, and decking.

Crews erected scaffolding and structural staging for roof frame repairs beginning in March 2010. Universal scaffolding was installed directly below each of the four valley members in the west dormer pair, X-shaped in plan, and with scaffold decks descending from the apex at the center to plate timbers on the north and south. Overall height of this construction was approximately 10.67 m at the plates to 16.76 m at the center of the dormer pair, and provided framers with a platform from which to work. Sixteen towers of structural staging were added to this construction to support purlins at purlin-valley connections (Fig 11).

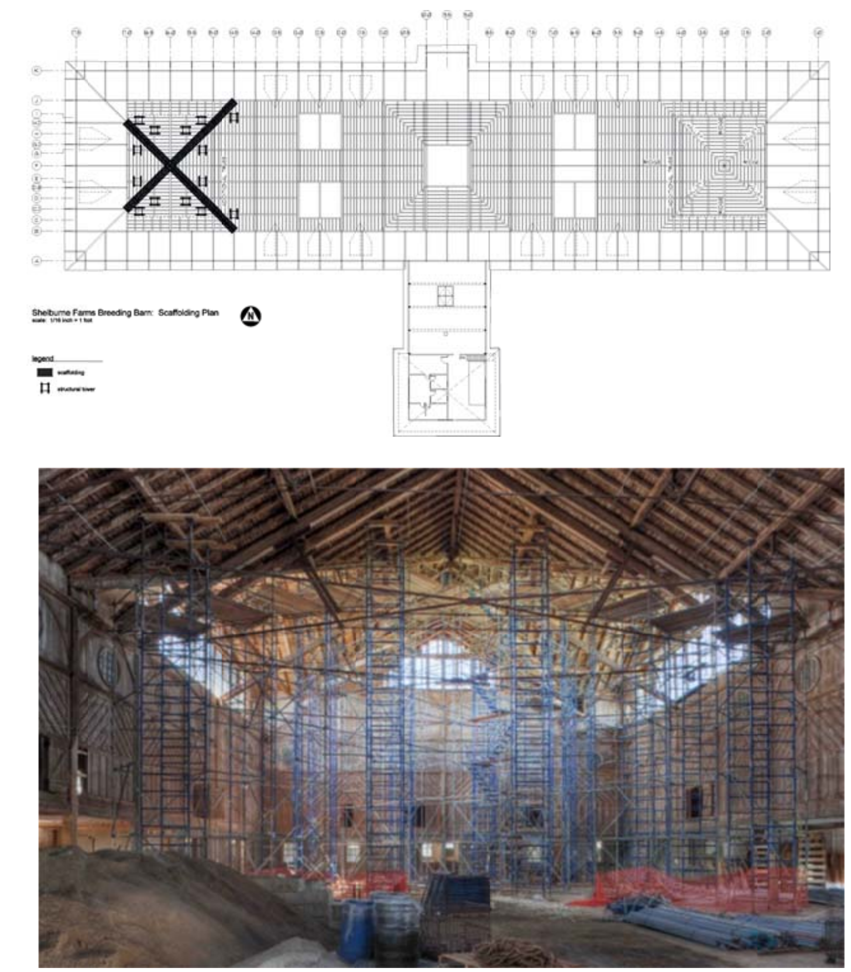

Figure 11 – Scaffolding used for accessing the roof was X-shaped in plan (top) and surrounded by structural staging that supported purlin loads (bottom).

Once roof loads were transferred to the structural staging, steel channels bolted on either side of each valley member were removed. These channels were added to valley members c.1930, apparently in an attempt to address deterioration of the timbers, but connections between the channels and timbers were poorly designed and the channels promoted decay of the wooden elements.

Repair strategies for valley members included: 1) scarfing new timber into historic members in areas where the bending moment is low or where scarf joints receive support from other members of the frame; and 2) installing laminated veneer lumber dutchman repairs in decay voids using a gap-filling epoxy adhesive.

A modest testing program was undertaken to select adhesives, determine the relative strength of dutchman repairs, and compare the performance of different scarf profiles. Candidate adhesives were selected based on temperature requirements, gap-filling properties, pot life, clamp time, and curing time. Each candidate epoxy was used to edge-glue ten panels under ambient temperature and humidity conditions. These were evaluated informally for ease of mixing and application, curing, and strength. Of the evaluated adhesives, the repair team selected West System 105 epoxy resin with West System 206 hardener as having better bulking and curing properties given ambient conditions in the barn.

To compare the strength of repaired timber to undeteriorated solid timbers, the team conducted a series of bending tests. The test protocol was established to replicate the field conditions under which the repairs would be made. As such, strict adherence to an established testing standard would likely have limited the project team's ability to determine the optimum repair strategy for the truss timbers. All tests were conducted using a three-point bending test (Figure 12).

The tested repairs included nosed, keyed, and bolted scarf joints, and laminated veneer lumber (LVL) dutchman repairs installed in slots cut into simulated deteriorated timbers. The ultimate bending strength, modulus of elasticity, and mode of failure was recorded for each test. Results were compared to solid timber control specimens. Based on the results (as well as the ability to implement the repair *in situ*), the LVL dutchman repair provided the optimum repair strategy for long unsupported spans. Scarf joints were acceptable where they receive support from other parts of the frame, or where bending moments are low.

Figure 12 – Full size samples were tested to compare the performance of repaired timber to undeteriorated timber.

For valley members intersecting the plates at grid coordinates B14.5 and J14.5 (see Figure 11 for grid coordinates), decay in the lower third of the length of the members (near their intersections with trusses at gridline 15.0) was repaired by scarfing in new timber (Figure 13); scarf joints were located near intersections with trusses and were reinforced with bolts and structural washers. For valley members intersecting the plates at grid coordinates B11.0 and B8.0, scarf repairs were made within 2 m of the plate timbers, where the bending moments are low. The valley member intersecting the plate at grid coordinate J11.0 was replaced in an earlier repair campaign by three butt-joined timbers bolted to the steel channels. This assembly was replaced with a scarfed member (installing a single full-length timber was not feasible without removing a portion of the roof) with the scarf joint located near the lower queen strut, where the bending moment is low. At either gable end, principal rafters were repaired by scarfing in new timber to replace decayed sections; scarf joints receive support from gable wall framing. Scarf joints were modeled on joints cut in each top chord element and are typically tapered, at least 1.5 m in length, and reinforced with bolts and structural washers.

Figure 13 – New timber was scarfed into the valley member at J14.5 where both timbers receive support from truss rafters. The length of the replacement timber is approximately 6.4 m.

Valley members intersecting the plates at grid coordinates B17.0, J17.0, B11.0, and B8.0 were characterized by long decay channels in the upper half of the timber section, apparently the result of water from roof leaks accessing drying checks on the upper surface of each timber. These were repaired by removing decayed material to leave a long dado that was then filled with an engineered lumber dutchman adhered with a gap-filling epoxy (Figure 14). In some instances (as at B11.0 and B8.0) it was possible to drop the timber to the scaffold deck for treatment; in others (as at B17.0 and J17.0), cutting and assembly were done *in situ*.

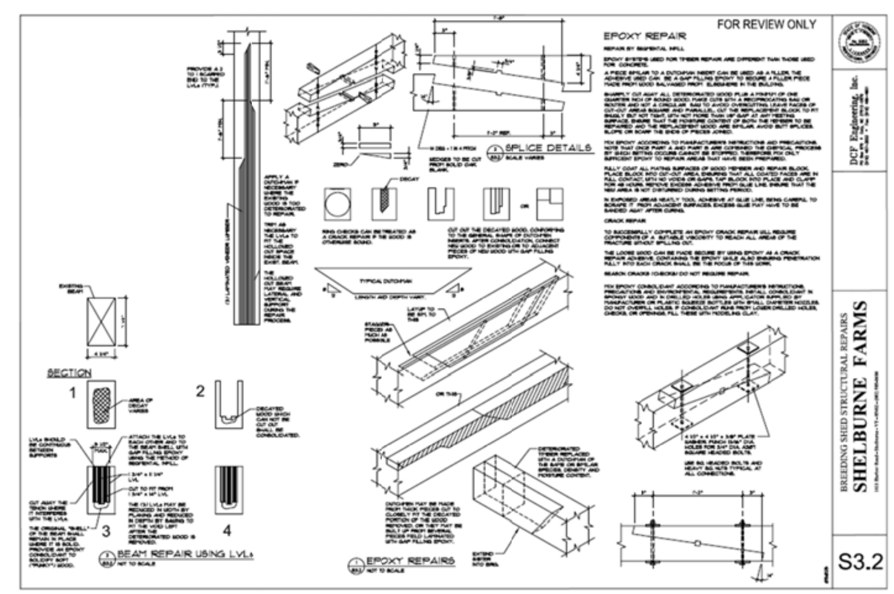

Figure 14 – To repair long decay channels, deteriorated wood was removed from the timbers; engineered lumber dutchman repairs were installed in the voids using gap-filling epoxy to adhere the pieces.

4. OBSERVATIONS AND SUMMARY

Resistance drilling was focused on valley members, where decay was extensive and timbers could not be visually inspected effectively because of the framing on top surfaces and steel channels bolted to the sides. In most cases, drilling results corresponded well to actual conditions and the drill survey proved to be a powerful tool in anticipating the extent and types of the repairs. For example, the results of the drill survey at J14.5 indicated a timber that was essentially sound over two-thirds of its length, with significant decay occurring near the intersection with the truss at gridline 15.0 and continuing through to its intersection with the plate. In the figure, the graphic is color-coded to indicate varying levels of deterioration along the length of the member. The section drawing approximates the size and shape of the decay channel at a particular drill site, and is based on drill results, the factors contributing to decay at this location, and the investigator's experience and judgment. The photo, taken while repairs were underway, indicates the close correspondence between the graphic and actual conditions at that particular drill site (Figure 15).

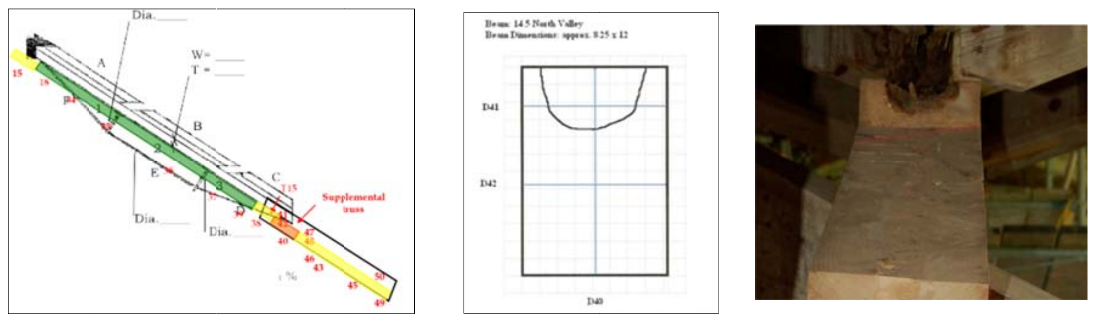

Figure 15 – The color-coded graphic indicates the portion of the valley member at J14.5 most severely affected by decay (left), and the section drawing roughly indicates the size and shape of the decay channel near the intersection of the valley with the rafter at gridline 15.0 (middle). The photo (right), taken during construction, demonstrates a close correspondence between the section drawing based on drill results and actual conditions.

In two cases, however, framers encountered significant damage that did not appear in the assessment documentation. In both instances, there were large areas of decay that appeared to originate at the surfaces to which steel channels were attached. These did not appear in the drill survey results in part because the steel channels limited access to those areas. While the presence of decay along the interface between the timbers and steel reinforcements was expected, the extent of the decay in these two instances came as a surprise. For the valley member at B11.0 (Figure 16), decay along both surfaces was so severe that the repair specification had to be changed. This limitation of resistance drilling should be considered when conducting a condition assessment where access to the full timber is restricted.

On completion of woodwork repairs, missing truss work was reinstated and purlins were reconnected to valley members. All of the valley members at the lantern location (at the center of the barn) required new truss work. Truss rods and pipe struts were fabricated in steel (the originals are of wrought iron) to match the profiles and dimensions of the originals. The iron bridles that connected purlins to valleys were lost when the steel channels were installed, and purlins appear to have been shortened to make room for the steel. Ghosts left on the woodwork allowed fabricators to replicate original bridle profiles, and extra-deep replacement bridles (to engage the shortened purlins) were fabricated of 4.76 mm steel plate (Figure 17). Blocks were installed at purlin ends to provide bracing for the valleys, and roof loads were returned to valley members.

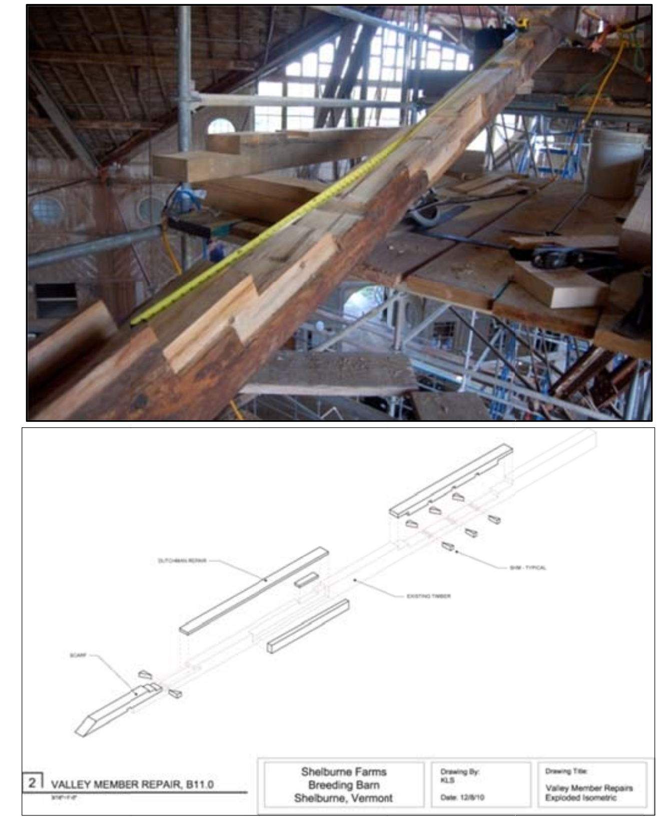

Figure 16 – The valley member at B11.0 with decayed material removed in the upper half of the span (top). Note the shear block mortises cut in the member and in the timber dutchman on the scaffolding behind the valley timber. The extent and location of the damage necessitated a change in the types of repairs employed (bottom).

Figure 17 – Purlins are supported on custom-made bridles that replicate original ironwork. Where it was necessary to extend purlins, loose tenons were installed and blocks added to brace the valley timber.

The structural investigation of the Breeding Barn was conducted over the course of three years, at a cost of just under 20 percent of repair costs. The time and effort spent on materials characterization, load testing, modeling, and analysis were offset in this case by vastly reduced impacts on historical integrity and significance. Resistance drilling proved to be an effective way to anticipate the extent of repairs in the valley members, provided designers with the lead time necessary to design repairs that were conservative of original material while meeting public safety requirements, and helped to prevent expensive delays in construction. The modest testing program that was focused on repair performance gave designers the data necessary for proper detailing of valley member *in situ* repairs, and gave craftspeople an opportunity to select repair materials best suited to conditions in the building, as well as streamlined the repair process. The assessment of the accuracy of field inspection results and the evaluation of the efficiency of various repair options that needed to be conducted *in situ* through a mechanical testing program make this project invaluable to any discussion of repairing and extending the service life of a historic timber structure using best practices.

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Assessment and Analysis of the Breeding Barn at Shelburne Farms

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ABSTRACT: Shelburne Farms, originally the agricultural estate of William Seward and Lila Vanderbilt Webb, is a 566-hectare National Historic Landmark District located on the eastern edge of Lake Champlain in Vermont, U.S.A. Significant for its landscape, designed by Frederick Law Olmsted, Sr., and buildings designed by New York architect Robert Henderson Robertson, it is dominated by four monumental Victorian buildings. The Breeding Barn (1891), center of Dr. Webb's horse-breeding efforts, consists of a main block 32.6 meters wide by 127.4 meters long, with a two- story annex (Figure 1). The riding ring is framed with composite trusses consisting of timber top chords and wrought iron ties and braces. Several previous engineering evaluations pronounced the roof structure inadequate with respect to current building codes in spite of reinforcing and a history of adequate service. The purpose of this paper is to present a more rigorous approach based on measurement, observation, condition assessment, testing, and analysis. The goal of the project was to use this comprehensive approach to more accurately predict the likelihood of mechanical failure in an element or connection in the roof structure.

Beginning in 2005, a project team conducted an extensive investigation which included:

1. Accurate and detailed examination of surviving fabric to discover the nature and condition of materials and connections, using laser scanning, resistance drilling, load testing, and metallurgical analysis.
2. Careful and rigorous analysis to better understand the results of the load tests and how they represent the actual performance of the roof structure.

As engineers, in reviewing the designs of trusses analyzed prior to the late 1960s, we must account for the variations in the results due to design methodologies in use at that time. In the Breeding Barn truss, the computer model must account for various details in the truss assembly in order to explain its record of service through 116 harsh Vermont winters. A simplistic approach does the original design and designers a disservice by pronouncing the structure inadequate and in need of significant reinforcing. On the other hand, a thoughtful approach which considers the subtle complexities in the truss, as well as the boundary conditions, is what constitutes the value of a second opinion.

HISTORY AND CONSTRUCTION CHRONOLOGY

Shelburne Farms, originally the agricultural estate of William Seward and Lila Vanderbilt Webb, is a 566-hectare National Historic Landmark District located on the eastern edge of Lake Champlain just south of Burlington, Vermont. The property is owned and operated by a nonprofit organization devoted to the cultivation of a conservation ethic through education and the stewardship of natural and agricultural resources.

Figure 1. *The Breeding Barn at Shelburne Farms*

The Webbs developed the estate between 1886 and 1902, as part of a grand experiment to develop innovative new approaches to land use and farming. Early in the process of acquiring the land, Webb consulted with celebrated landscape architect Frederick Law Olmsted, Sr. (1822-1903), to develop a landscape design for the growing estate. In his c.1887 design, Olmsted proposed a plan dividing the estate into farmland, forest, and parkland, combining the pastoral and picturesque in the tradition of the great "ornamental farms" of nineteenth-century Europe.

The estate architecture was designed by New York architect Robert Henderson Robertson (1849-1919), a prominent nineteenth-century designer of monumental architecture. Robertson was an early designer of skyscrapers and today he is best known for his Park Row Building (1896-1899), which at 27 stories was the tallest building in New York at the time of its construction. The buildings at Shelburne Farms represent Robertson's most significant estate commission. The Breeding Barn is the principal building of the Southern Acres portion of the Farm, and construction was begun in 1889 and completed in 1891 (Figure 2). At the time, the barn was said to be "probably the largest and best-appointed building of the kind, not only in the United States, but in the world. Those who have seen it call it one of the wonders of America". (Frank Leslie's Popular Monthly, September 1892).

The main block of the building is approximately 32.6 meters wide by 127.4 meters long, with a two-story annex centered on the rear facade. The building is timber-framed,

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supported on foundation stonework of Monkton quartzite, and clad in wooden shingles.

Figure 2. *The barn under construction, c1890.*

Building elevations are dominated by the complex-sloped 0.8-hectare hipped roof, with multiple dormers and enormous central tower. The walls are clad in wood shingles punctuated by scores of multi-pane windows that admit light and ventilate the interior space. A gable-roofed arched entry is centered on the front façade.

At the center of the building, an unbroken cathedral-like space measuring approximately 22 meters wide and 110 meters long once housed the riding ring. Surrounded by stables, the ring was lit by gable windows of eight large dormers and lantern glazing in the tower, supported on timber purlins 16.8 meters above the floor. The riding ring is surrounded by framed aisles on all four sides that once housed stalls at ground level and loft space above. The annex, added sometime after initial construction of the main block, originally housed grooming operations, a tack room, and machinery for processing oats. The level of interior finish is very high throughout most of the building (with the exception of the loft space), with wood-paneled walls, cased window and door openings, and neat chamfers on exposed frame elements.

The framing of the aisles and annex is fairly typical of heavy timber framing of the day, but Robertson borrowed from contemporary railroad construction in iron in designing the beautiful and highly efficient roof structure over the riding ring (Unwin, 1869). Here, a series of fourteen principal trusses of timber and iron were designed to support the roof expanse. Bay width was apparently determined by spacing of the stalls at ground level. Each truss has timber (southern pine) top chords trussed with wrought iron tension members and struts; a raised center element of wrought iron completes the truss form. At the lower end, principal rafters are captured in cast iron shoes that also receive the ends of the tension members. Shoes are seated on timber plates at principal post locations. With the ironwork painted white and receding from view in the limewashed riding ring, Robertson was able to convey the impression of a classically-framed timber building with an enormous open volume below the heavy timber rafters. Principal rafters support a roof of purlins and common rafters. Originally, purlins and valley rafters at each of the large dormers were trussed with wrought iron tension members and iron pipe struts.

There have been several major structural interventions in the Riding Ring roof frame; their chronology is only partially understood. As originally designed by Robertson, fourteen principal trusses divided the Riding Ring into fifteen bays, and the earliest set of drawings included no provision for cross-bracing of the long walls (Figure 3). At some point it was realized that lower chord elements as specified were too small to prevent deflection of the trusses and bending of side wall columns, and Robertson's office issued a new framing plan calling for installation of cross-brace ties between every principal rafter pair. These ties were to terminate in cast iron connections fastened to plate timbers behind truss heel connections. At the same time, trusses were doubled under the tower to support additional loads associated with that structure. The dimensions of iron tensile elements in the tower trusses were increased and additional struts were added to support top chords at tower purlins. At the doubled trusses, cross-brace ties terminate at "double-wide" cast shoes spiked to the timber plates.

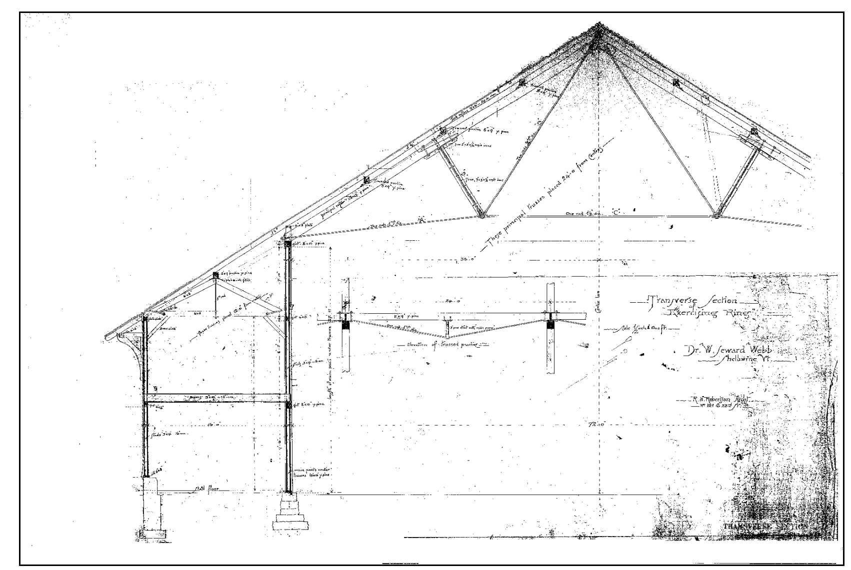

Figure 3. *Robertson's earliest drawings of the roof frame omitted the ironcross-tie braces at plate level.*

Sometime subsequent to original construction but early in the history of the building, additional trusses were installed to support inboard dormer framing for major dormer pairs located at the east and west ends of the ring. Top-bottom chord connections are still made at cast-iron housings, though these differ in design from the original elements. In the newer trusses, the raised center elements of the lower chords are equipped with turnbuckles, unlike their counterparts in the original trusses. Because of the proximity of cross-brace ties to the original trusses, the newer trusses were installed between columns and required additional bracing in riding ring walls. The level of craft displayed by these new trusses is roughly equivalent to that of the original construction.

An intervention which probably took place several decades later resulted in alteration of many of the valley rafters associated with the major dormer pairs. Originally, valley rafters at each of the large dormers were trussed with wrought iron tension members and paired iron pipe struts. Sometime in the twentieth century trusses on many of the valley rafters were removed and replaced with steel channels bolted on either side of the timbers. Addition of the steel channels necessitated cutting of purlins terminating at the valley rafters. In most cases, cutting appears to have been done crudely, with hatchets and chisels. Following installation of the steel channels most of the purlins were tied to valley rafters using bolted bent plates, except at dormer locations. Here purlins were never replaced, leaving trusses located at the centers of each major dormer pair unbraced.

PRESERVATION PLANNING

After decades of disuse and deferred maintenance, the Breeding Barn was in an advanced state of deterioration. An engineering assessment of the building in 1990 identified several areas requiring repair, including decay in all of the valley rafters in the large dormers at the tower and either end of the riding ring, jack rafters and plate timbers in their vicinity, and ridge timbers in the smaller dormers (CEA, 1990). The assessment called for repairs of deteriorated elements but stopped short of recommending augmentation of overstressed elements because of the impacts augmentation would have on historic integrity. Beginning in 1995, repairs were implemented that ultimately included stabilization of some of the foundation stonework with reinforced concrete, repair and/or replacement of some of the deteriorated structural timbers, replacement of the roof covering with standing seam copper, and installation of a fire-suppression system.

With the help of a Getty Planning Grant, Shelburne Farms was able to complete a conservation assessment for the Southern Acres buildings and landscape in 2004. A project team was assembled to conduct detailed structural assessment of the building and prepare plans for its augmentation and repair. Because of the significance and integrity of the resource, it was determined that any intervention should be as conservative of historic fabric as possible, that the historic structural system should be preserved to the fullest extent possible, and that traditional repairs are preferable to introducing new technologies so long as public safety goals are met. In order to design an intervention program that meets structural goals and guarantees public safety while having the smallest possible impact on surviving fabric, the multi-disciplinary design team determined that the focus of their work would be:

- Accurate and painstaking examination of surviving fabric to discover the nature and condition of materials and connections;
- Characterization of timber and metal elements, using non-destructive and quasi non-destructive testing techniques to the fullest extent possible;
- Rational selection of design values based on the conditions survey, materials testing, and review of the original construction documents and original design methodologies;
- Reduction of factors of safety through exhaustive knowledge of the building;
- Identification of overstresses through careful modeling and analysis.
- Development of a HABS-level documentation package, to be contributed to the Library of Congress upon completion of the project.

The Breeding Barn is in a jurisdiction subject to the *2005 Vermont Fire and Building Code* which has adopted the *ICC International Building Code, 2003 Edition.* This code allows for performance-based compliance exceptions in the case of historic buildings. The code has been used in establishing required live loads for the building. In managing the historic landscape and buildings of the estate and adapting them to new uses, Shelburne Farms is broadly guided by the *Secretary of the Interior's Standards for Rehabilitation*. Because of the significance and integrity of the Breeding Barn, and its importance in the history of the development of structural form, the project team has been additionally guided by the *ICOMOS Principles for the Preservation of Historic Timber Structures*, and the *ISCARSAH Principles* and *Guidelines*.

Figure 4. *Barn interior.c1900 showing the roof frame of the riding ring, including iron cross-brace ties.*

CONDITIONS ASSESSMENT AND MATERIAL TESTING

Initial examination of the building was organized as a training workshop in partnership with the University of Vermont. Professional team members included an architectural conservator, a structural engineer specialized in the analysis of historic timber buildings, and three timber framers associated with the truss research group of the Timber Framers Guild. Student trainees were selected from the Civil Engineering and Historic Preservation programs at the University. Trainees were paired with professional team members to form sub-teams. Each sub-team was assigned a portion of the building to survey. Survey data, including information about element dimensions, species, quality, and condition, was recorded on survey forms; survey forms included drawings of each of the principal structural elements so that deterioration and damage could be graphically represented.

The survey indicated that more detailed examination of the principal roof frame members was necessary to determine the quality and condition of several of the iron structural elements, and to determine the extent of the deterioration in several of the timber elements. The team was most concerned with unbraced rafters in each of the major dormers, with the condition and surviving capacity of several of the valley rafters, with the absence of a positive connection between valley rafters and other timber elements at the apex of the roof, and with the capacity of iron tension elements.

The wrought iron used in struts and tension elements was characterized with respect to metallurgical and mechanical properties. Samples were obtained from fabric that had been previously demolished. Strength-in-tension tests (ASTM E8) indicated an average yield strength of about 227,527 kPa and a MOE of 206,842,710 kPa. Small portions of each sample were retained for metallographic characterization. Analysis indicated a low-carbon material; the closest SAE-AISI alloy designation is 1005.

In order to quantify the extent of deterioration in decayed timber elements, a wood scientist assisted with a detailed evaluation of decayed timbers identified by the survey team. Quantification entailed resistance drilling of decayed timbers, using the IML-RESI System. Resistance drilling is a quasi-nondestructive technique for determining the relative density of wood, identifying discontinuities and quantifying the extent of section loss in the process. The process was exceptionally useful in evaluating valley rafter timbers, where installation of reinforcing steel channels on either side of each timber prevented direct examination in most cases.

Prior to this investigation, the extent of deterioration in the valley rafters at the Barn was not known. The wood investigation focused on resistance drilling, but included a combination of visual observations, moisture content measurements and probing to identify and quantify deterioration of the timbers in the 12 valley rafters. The likely causes of deterioration were identified for the purpose of establishing effective remedial treatments and repairs, and addressing long-term maintenance needs.

The timbers that make up the valley rafters were found to be generally in good to excellent condition. There were some exceptions. Each of the rafters was subjected to resistance drilling along its length to generate a schematic of the location and approximate extent of deterioration. Some of the rafters have deterioration on the upper face of the timber that penetrates to various depths, a condition called channelizing. Two of the valley rafters have severe deterioration of the heel where they bear on the interior wall.

Using the grid numbering system implemented by the survey team, the resistance drilling results were summarized graphically to illustrate the location and extent of deterioration in each rafter. Schematics of each valley rafter are color-coded to provide the reader with a visualization of the deterioration found. Conditions identified in red were priorities for further engineering analysis and possible repair. Areas colored green indicate no deterioration found. Areas in yellow exhibited minor channelizing (approximate depth of two inches or less) at the top of the rafter or minor deterioration elsewhere in the cross section. Orange areas indicate either local failure or deeper channelizing.

An example of one of the valley rafters is shown in Figure 5 and the corresponding resistance drilling findings are shown in Figure 6. Approximate resistance drilling test locations are marked by the drilling number on the schematic. As shown in Figure 6, this rafter has a varying extent of channelizing along the lower length of the rafter. Resistance drilling and probing revealed minor channelizing between the queen posts that progressively increased to the heel of the rafter. The upper length of the timber was found to be in good condition and is indicated as such by the green color.

Figure 5. *Valley Rafter 17 South viewed from the apex.*

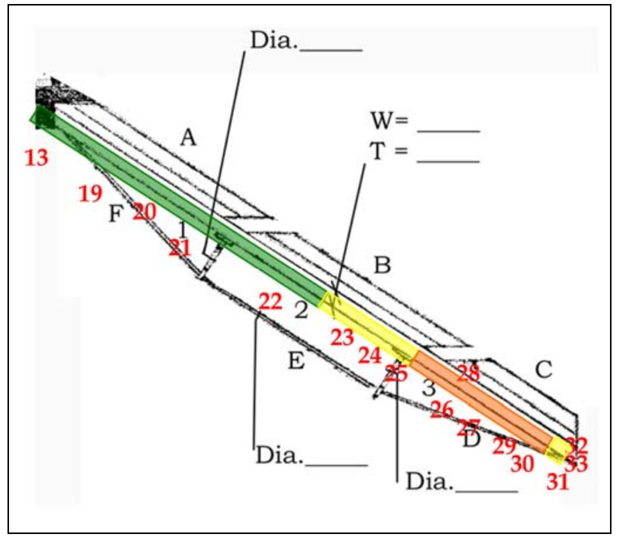

Figure 6. *Resistance drilling results for Valley Rafter 17 South. Numbers in red indicate drilling locations.*

Figure 7 is a diagram of the drilling results from drillings 31, 32, and 33 (which are shown on the schematic in Figure 6). The diagram is an approximation of the width and depth of the channel due to decay as indicated by the three drillings. A decay pocket of this depth was referred to as deep channelization**.**

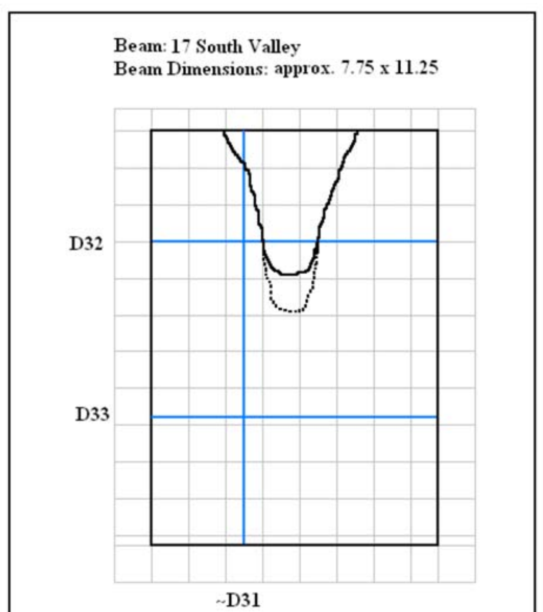

Figure 7. *Diagram showing channelization pattern in Valley Rafter 17 South. Dotted line indicates likely pattern of the decay pocket, since only three drillings were conducted.*

Of the twelve timbers examined, four were found to have substantial section loss due to decay. Because the results of resistance drilling tests were expressed graphically and in tabular form, indicating the extent of section loss at each of the drill sites, this pinpointed areas of loss. Characterization of section loss based on resistance drilling will permit detailed design of timber repairs prior to dismantling the affected portions of the building, helping to reduce the number of inappropriate decisions made in the field.

STRUCTURAL ANALYSIS

The procedure for evaluating a building such as this is to apply today's code mandated snow, live, and wind loads to various component systems, assuming that no deterioration has occurred. In this way, the original structure can be tested with specific design load criteria, against reasonable allowable design values with the amount of overstress tabulated for various elements.

By performing a plane frame computer analysis, the stiffness in the various components can be included, resulting in accurate theoretical deflections. The computed dead and live load deflections can then be compared to today's code mandated limits for roof structures. Once this process is completed, then a review of the amount of overstress in particular elements can be compared against reasonable values which could be expected from dense clear growth southern pine harvested in the late 1880s. After the structural analysis is complete, then a condition analysis can be made on the basis of field observation, measurement, and testing. Through analysis and engineering judgment, the capacity of the various components can be tabulated accounting for deterioration.

The production of a set of measured drawings of the structural elements, based on the original R.H. Robertson drawings and data collected by 3-D laser scanning of the building established the original configuration of the building "as built". The structural elements of primary interest in the Breeding Barn are the principal trusses, purlins and valley rafters. The team reviewed the original plans to determine the impact on the analysis of various elements. For example, the struts in the principal trusses in the R.H. Robertson drawings are called out as "four 3" x 3" x 3/8" angle irons." The survey indicated that the sizes of actual members are different from those shown in the plans. Although there is a wealth of information in the partial set of original drawings which remain, the effort to obtain a complete set of documents from other sources should continue. As Historic American Building Survey (HABS)-level drawings are developed, differences between the actual building "as built" and the original drawings will be documented.

The preliminary analysis of the primary truss was performed with a 146.5 kg/m² snow load and 73.2 kg/m² dead load. The analysis indicated that there are overstresses in the top chord as well as the rods which extend from the heel supports to the queenpost struts. It is possible that the overstress in the 0.254 m x 0.305 m top chord is a result of the original designer analyzing the truss using graphical means (force diagram and string polygon) first developed in the United States by Col. Stephen H. Long in the 1840's. This method of

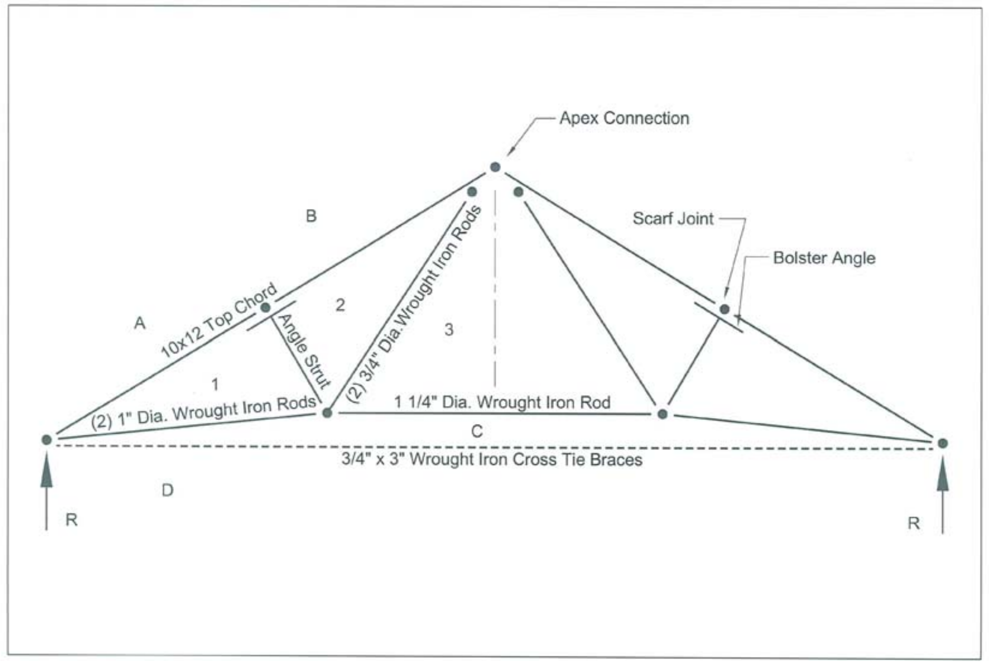

Figure 8. *This image shows the location of the bolster angels and the connection arrangement at the apex.*

analysis provides only axial member forces. It is fairly accurate for trusses where purlin loads are applied to panel points. In this case, the top chord on each side of the truss has reactions from purlins applied midway between panel points. Even today, with new structures, computer analyses will provide often critical bending forces that can not be determined from a graphical analysis.

To analyze the truss as component in its simplest form, certain assumptions are required for graphical analysis, the method of joints, and the methods of moments and shears. Primary axial stresses are obtained on the basis of simplifying assumptions, producing an ideal truss with members having only axial forces. The following assumptions are made to allow the truss to be analyzed:

1. The truss members are connected together with frictionless pins.
2. Truss members are straight and the axis of the members intersect at joints.
3. Deformations under load do not excessively change the basic truss geometry.
4. Loads and reactions are applied only at joints (Figure 9).

Apparently, the truss in the riding ring evolved from a simple truss analyzed by graphical means, to one with purlins located between joints (Figure 10), to a system combining a truss with a horizontal tie (Figure 11). The wrought iron angles used to bolster the spliced top chord is an interesting detail. These may have been added as an afterthought, sometime during design, to reinforce the top chord acting as a two-span beam supporting purlins at the midpoint of both spans. Although it certainly was possible to obtain southern pine in twelve-meter lengths to produce a two-span continuous top chord, the designers chose to splice the top chord directly above the bolster angles.

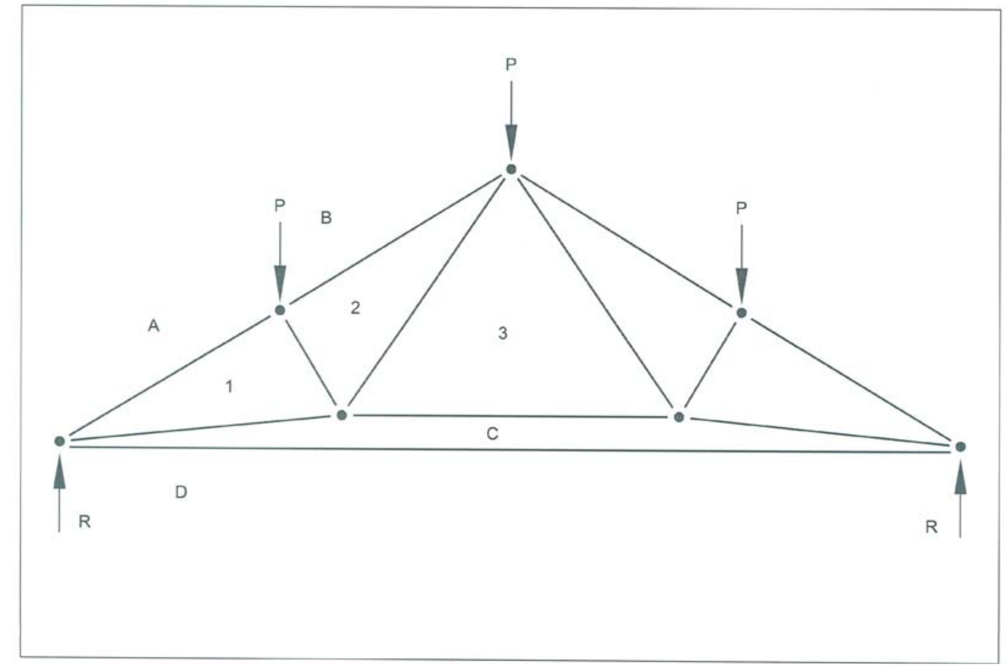

Figure 9. *This shows the placement of the unit loads for the load test with the loads located at joints.*

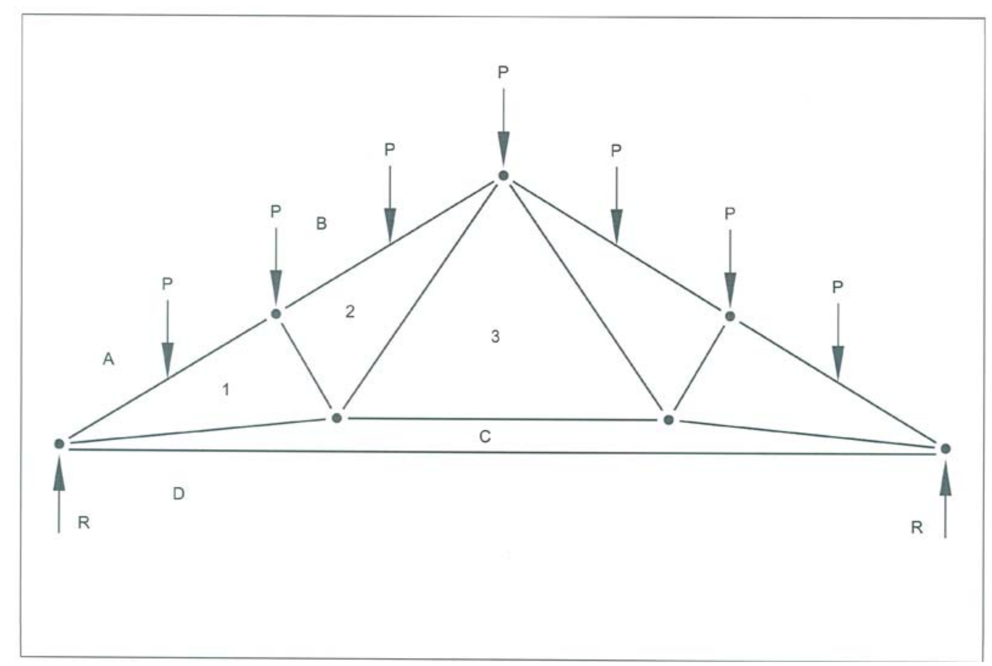

Figure 10. *This shows the placement of the unit loads for the load test with the loads located at purlins.*

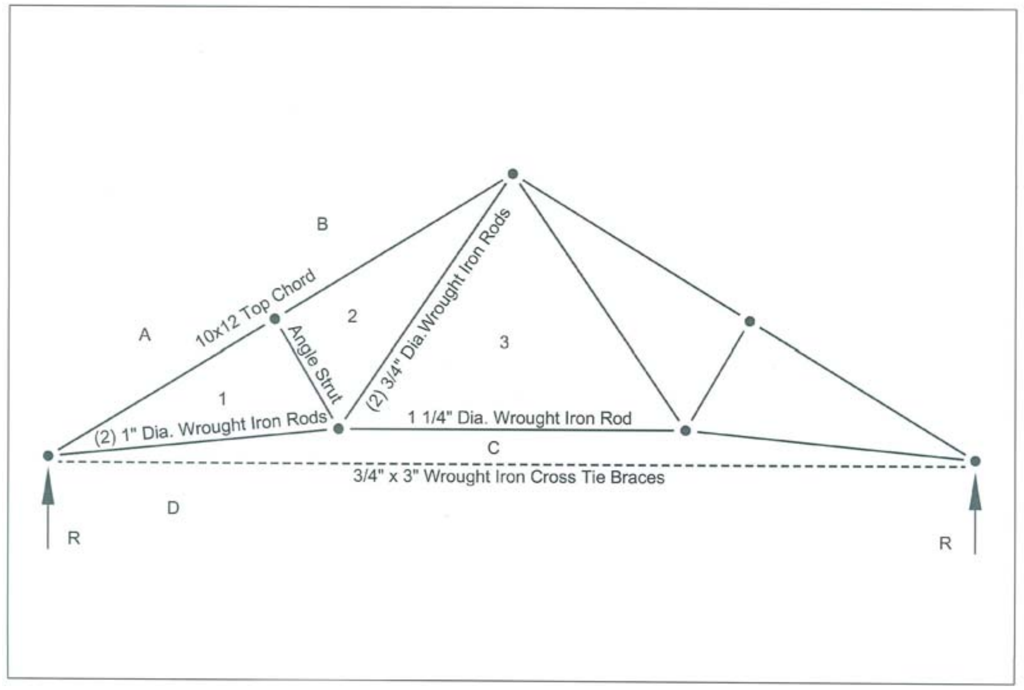

Figure 11. *Analysis of the truss with the cross braces included, causes member C3 to revert to compression.*

The computer analysis has tremendous advantages over traditional methods of analysis. Stiffness and continuity of various truss members can be accounted for as well as slight variations in truss geometry where the centroids of members do not converge at a single joint.

In simple plane frame analysis of the truss only, the 2.54 cm diameter rods are stressed to a f_T= 248,604 kPa. This is very high when compared to a tabulated elastic limit of 172,369 kPa and ultimate strength in tension of 344,738 kPa. Furthermore, the rods were measured to be actually 2.22 cm in diameter in lieu of 2.54 cm diameter, as shown in the existing drawings. This would increase f_T to 324,550 kPa which is well above the elastic limit for wrought iron.

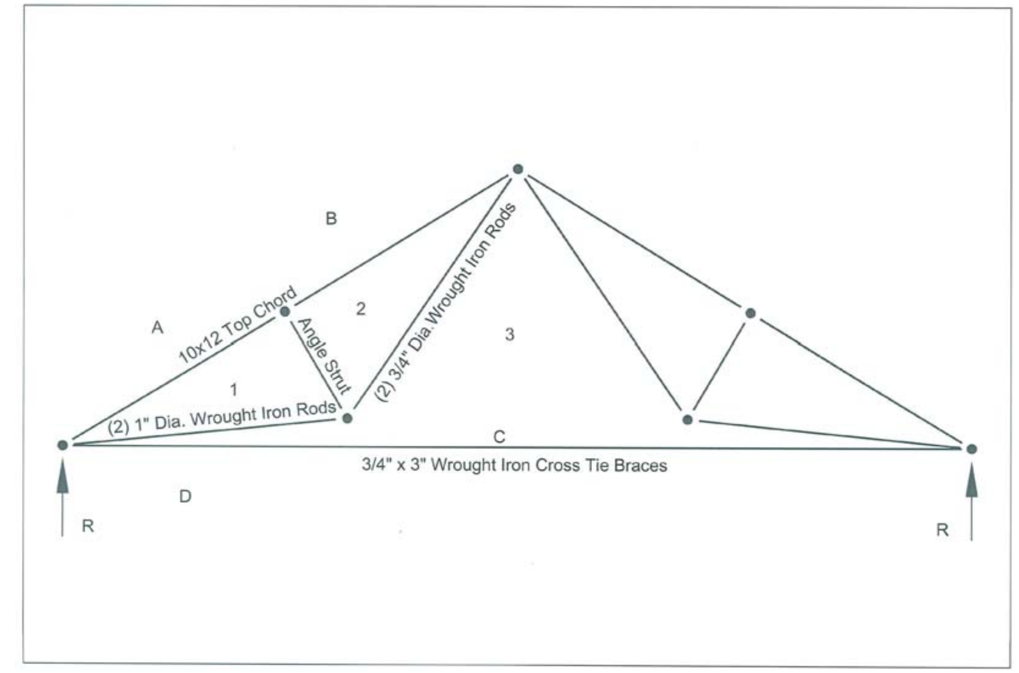

Figure 12. *Testing and analysis indicates that the roof system acts as an A-frame with trussed rafters.*

By using ultimate published values for clear wood specimens reduced by a factor of 4.0, the top chord of the truss almost checks out with 6% overstress. The static bending modulus of rupture and the maximum crushing strength in compression parallel to grain for clear straight-grained specimens of loblolly pine, divided by a factor of safety of 4.0 will yield values of Fb=22,063 kPa and Fc=12,286 kPa.

The design team has characterized the metal truss elements and has determined the allowable design values for each element through testing. Although the stresses for wrought iron components are high, all but one component are close or within range of the 172,369 kPa to 206,843 kPa elastic limit published in period handbooks (Hudson, 1939).

The primary issue in analyzing the trusses of the Breeding Barn is to determine the path of the horizontal tensile force in the trusses. This was also the goal of load tests. Member forces derived from analyses using the same unit load tests were compared to the results of the load tests.

Load tests consisting of 4448 N and 8896 N (force) unit loads suspended from purlins and panel points generally matched the results of the computer analysis and added credence to the thesis that R. H. Robertson added the cross-brace tie rods to the basic truss configuration sometime during construction, thus reducing horizontal and vertical deflection and reducing the stresses in both the original wrought iron elements and timber top chord.

The addition of 1.91cm thick by 7.62cm high cross-brace tie rods to the building, apparently during construction, provides another path for the tensile force to be resisted. By including the tie rods in the analysis of the building cross section, the center tie bar of the truss becomes a zero force member (Figure 11). Analysis of the roof truss by the computer using the appropriate section properties and material stiffness shows that immediate horizontal deflection under dead load only would have been a total of five centimeters. This deflection of the trusses would have manifested itself in bending of the 25.4cm x 25.4cm post from a point at the horizontal chord of the roof trussed above the side aisle to the 20.32cm x 25.4cm girt (sill) at the heel of the truss, a distance of almost two and a half meters. Certainly, a horizontal movement of two and a half centimeters would have been observed in the post for a distance of only two and a half meters in height. If the annex had already been built or partially framed, it would have provided some restraint, pushing most of the deflection towards the posts along the north side of the building which certainly would have been observed by workmen.

The answer was to provide additional ties to limit the movement which is natural to a truss with a raised bottom chord. In providing these ties R. H. Robertson transformed the building cross section into a tied A-frame with trussed rafters (Figure 12).

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