Acknowledgements of the team contributing to this Report:

**SmithGroupJJR / Architecture and MEP**
Troy Thompson, AIA LEED AP  
Principal in Charge  
Marcus Wilkes, AIA, LEED AP  
Project Manager

**Thornton Tomasetti / Structural Engineering**
Mark J. Tamaro, P.E., LEED AP  
Principal  
Henry Clouting, CEng, MIstructE  
Senior Engineer  
Jennifer Himottu  
Engineer

**Smithsonian Institution**
Richard Stamm  
Curator, Smithsonian Institution Castle Collection
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Executive Summary

Location of project scope

(A) Main Hall Roof Trusses

(B) West Range Cloister

(C) South Tower Corbel

Fifth Floor Plan

Second Floor Plan
Executive Summary

This narrative presents the final submission of the Roof Structural Survey and Repair Report for the Smithsonian Institution Building (SIB), a.k.a. ‘The Castle’. The scope of this project was to follow up on critical recommendations highlighted in the 2009 Existing Conditions Report for the SIB as related to structural (roof) issues, and provide recommendations for repairs if necessary. The SmithGroupJJR team was asked to investigate three specific conditions (noted on plans at left):

(A) Analyze the Main Hall Roof trusses with the ultimate goal of determining their structural capacity.

(B) Investigate corroded beams of the West Range (Cloister) roof and determine if remedial action is necessary.

(C) Investigate a beam/bracket detail and wall cracking seen on the 5th floor South Tower and determine if remedial action is necessary.

The recommendations for action included in this report are intended to be temporary. While these solutions address the immediate structural concerns, they are not designed to be permanent solutions. The premise of this assumption being that a future building-wide renovation would address structural deficiencies in a manner more sensitive to the historic fabric of the building and in the context of a full roof replacement and/or envelope performance upgrade. A detailed Cost Budget for each recommendation is provided in the Appendix of this report.

Task A – Main Hall Roof - Summary

The existing Main Hall Roof trusses were surveyed to confirm member sizes, configuration and connection details. Modifications to the roof trusses during previous renovations (primarily 1968) were documented including framing alterations for mechanical dunnage platforms suspended from trusses in two areas over the 4th floor office spaces. Preliminary calculations were undertaken in August of 2011 using a material strength assumption taken from published literature and section manuals of the period (ca. 1866). This initial effort indicated a probable overstress of some truss members under modern loading criteria. The design team then recommended a more accurate evaluation of the truss member strength via material testing. A research effort was undertaken to determine if records of previous metallurgical tests could be found. Ultimately, no records of such tests were found within Smithsonian Archives or Records, and a new set of material samples were taken in the Fall of 2013. With this new laboratory test data, the structural capacity of the trusses was determined and evaluated for adequacy to meet loading criteria required by modern building code.

The Main Hall roof trusses were found to be overstressed by a maximum of 38% when analyzed with full ASCE 7-05 snow loads and therefore do not meet the requirements of modern building code under this criteria. This code applies a factor of safety on material strength and the degree of overstress, in these truss members, is such that it has reduced the effective factor of safety, but is not high enough to have exceeded the material strength. Overstress of truss members is slightly greater at locations of suspended steel dunnage supporting air handlers than typical roof trusses. Load cases and assumptions for this analysis are discussed in detail in Chapter 3 of this report.

Upon understanding the overstress of the Main Hall roof trusses under full code compliant loading, an analysis was undertaken to determine the maximum uniform depth of snow which would cause the allowable stress to be exceeded. The result of this analysis indicates that the trusses can support a uniform depth of approximately 6 inches of snow before they begin to become overstressed.

Recommendations for temporary measures to avoid snow accumulation as well as recommendations to reduce the likelihood and severity of any potential structural distress or failures due to a snow event are presented in Chapter 4 of this report. These options include adding heat to the Main Hall attic via electric fin tubes suspended under the roof, adding bracing from the 4th floor to support mechanical platforms, and removing dunnage steel platforms.
Executive Summary

Task B – Cloister Roof - Summary

The West Range (Cloister) roof beams were surveyed to determine member sizes, configuration, and extent and severity of corrosion. A structural analysis was undertaken to determine the load carrying capacity of the existing Cloister roof. Material testing of the Cloister roof framing was not undertaken as a part of this project. Material strength for these members was assumed from similar size members published in literature and section manuals of the period (ca. 1866). The corrosion levels, while unsightly, do not appear to have reduced the section properties of the members in a significant way. Utilizing this data, and under loading criteria required by modern building code, the Cloister roof framing members to not appear to be overstressed. The recommended action for the Cloister roof framing is to clean the loose material from the framing members and repaint all beams as a temporary measure until the deteriorating roof can be replaced.

Task C – South Tower - Summary

The corbel/bracket detail supporting the main roof beam of the South Tower was surveyed to determine member sizes, configuration and connection details. A minimal destructive probe of the lath and plaster was conducted to reveal the bearing connection with the wall, which revealed that the beam sits fully on the bracket and is not seated in the masonry wall. The initial wall cracks observed in 2009 appear to be primarily cosmetic in nature and limited to the plaster finish. In addition, the cracking did not worsen as a result of the August 2011 earthquake. However due to the unusual nature of the beam/wall connection the recommended action is to add a steel bracket to each end of the main beam so as to more positively seat the beam in the masonry wall.
Reference Documents

The following documents have served as reference material for this Concept Report:

- “SIB Master Plan Existing Conditions Report”, Polshek Tobey + Davis, July 8, 2002
- “SIB Master Plan Existing Conditions Report-Appendix”, Polshek Tobey + Davis, July 8, 2002
- “SIB Master Plan Space Allocation Report”, Polshek Tobey + Davis, July 8, 2002
- “SIB Master Plan Preliminary Concept Design”, Polshek Tobey + Davis, July 8, 2002
- “SIB Master Plan Preliminary Concept Design Cost Estimate”, Polshek Tobey + Davis, September 10, 2002
- “Modify Windows & Skylights for Blast Mitigation: SIB”, Beyer, Blinder, Belle, April 18, 2006
- “Book One- Structural Preplanning Study-Phase 1-SIB”, Harry Weese & Associates, December 9, 1988
- Building Evaluation Study, Smithsonian Institution Building Prepared by RTKL, OFEO Project No. 933202 (Also refer to OSHEM Review Comments Memorandum dated 1/19/96)
- Office of Safety Health and Environmental Management (OSHEM) evaluation and technical review (METR) reports
- Current OSHEM Fire Protection Master Plan
- All Hazards Risk Assessment Report, July 2005
- “Facilities Assessment Report-the Castle, Smithsonian Institution Building-Appendices”, March 1998
- CAD Architectural and Site plan drawings
- SI Guidelines for Accessible Design (SGAD)
- SI space guidelines and definitions
- Historic documentation
- Cadastral information
- Current 5–10 year capital improvement plans
- Current 5 year major maintenance plan
- Current strategic plans at the Smithsonian-wide level
- Indoor Air Quality Evaluation, Smithsonian Institution Building Prepared by Law Engineering, OFEO Project No. 953202
- SI-Wide Perimeter Security Plans
- Basement Corridor Improvement Study, Smithsonian Institution Building, Prepared by Architrave, P.C., OFEO Project No. 913203
- Asbestos Studies by Law Engineering, see “Hazard Materials” section for additional information.
- Survey by AHHP “SIB Floor Plan Chronology”
Chapter 2
Research and Surveys
Two historic record drawings were referenced in the preparation of this report (reduced versions can be seen below).

The first is what appears to be a rendering of the typical Main Hall roof truss at a shop drawing level of quality. This drawing dimensions all major truss members, spacing, and connections. A half-truss typical cross section (cut north-south) provides overall dimensions. Information provided appears identical to as-built field measurements taken of the truss. The wood framing for the plaster ceiling suspended from the truss remains intact, but appears to be higher in elevation than as indicated on this drawings. The elevation of the plaster ceiling wood framing does not affect the analysis in any significant way.

The second drawing appears to be an architectural cross section (cut north-south) of the West Range and Cloister roof. Of note in the beam member of the Cloister roof can be seen the size of the framing written as: 6in. of 40lbs 8’ 7” from centers, giving the depth, weight, and spacing of the major framing which appears identical to the existing condition in dimension.
Research and Surveys
Site Survey Information and HSR Summary

Site visits were undertaken on July 19th, July 26th, and August 9th, 2011. During this time, the Main Hall Roof trusses were surveyed, and truss member sizes were recorded. Connection details, were accessible, were recorded as well. The presence of restraint or otherwise to truss compression members was noted and documented as was the suspended ceiling assembly below. Member sizes and the layout of beams were recorded for the West Range (Cloister) area and the South Tower 5th floor (roof) framing. Detailed results and dimensions from the site survey are shown in Appendix A.

The Main Hall roof trusses were fabricated in late summer of 1866, and installed by May 31, 1867. They replaced the original 1857 Main Hall roof framing that was destroyed in the 1865 fire. Per historic records, all iron trusses for the Main Hall Roof were manufactured by Phoenix Iron CO., of Phoenixville, PA. The suspended plaster ceiling was removed during a 1968 renovation, but the wood framing for the three major coffers remain. Two cross gables were added in 1968 joining the ridge of the South Tower and North tower to the Main Hall roof, replicating those which were present prior to the 1865 fire. These cross gables introduced new steel at the ridge and valleys between trusses #13 and #14. The 1968 renovation also added significant mechanical equipment, dunnage, and a plywood platform to the attic space created by the void of the Main Hall trusses, all adding deadload to the historic trusses. Further insulation was added in the same period adding potential snow load.

The South Tower was severely damaged during the 1865 fire, and the top 30 feet reconstructed in 1867. Therefore much of the structure visible at the fifth floor is believed to date to the 1867 reconstruction period. The floor beams, as designed by Aldolf Cluss in 1867, were supported by a pair of iron columns on each floor. These columns were removed from the Regents’ Room in 1899 and from the Children’s Room in 1900 to give more space to the rooms. Although no documentation has been found, it is possible that the columns on the other floors including the fifth floor of the South Tower were also removed at that time for the same reasons. Therefore the beam and bracket investigated on the fifth floor could date to either period.

Iron beams and truss members from both reconstruction periods (1867 and 1887) are assumed to be constructed from wrought iron based on historic documents recording construction progress during that era. The AISC historical sections book does not cover sections rolled prior to 1873 but it does give values for the allowable stresses for wrought iron. These values range from 10 ksi to 14 ksi. 10 ksi was used for the analysis of the beams for the Cloister and South Tower as this is the most conservative value in the absence of material test data. As part of this project, a Material Test was performed at the Main Hall roof trusses, therefore, the actual values were used for that system evaluation.
During the initial site survey and measurement of the Main Hall trusses (June 2011) a cut was discovered in a cross brace of Truss #14 (see photo at right). The size and shape of the cut was indicative of a material coupon or mill test used to determine material strength via laboratory (metallurgical) testing. Such a test is a typical step in determining the actual allowable stress of existing steel prior to making alterations or increasing loads on a structure. This type of test is destructive and often irreversible.

In the absence of this type of test, and often where preservation of historic fabric is of greatest concern, the allowable stress can be assumed based on of the period the structure was erected. However this method often results in a very conservative figure for the allowable stress, and is not nearly as accurate as material coupon.

A preliminary analysis of the Main Hall trusses (August 2011) was based on an assumption for wrought iron derived from published literature and text materials of the period (1866). This initial analysis suggested an overstress of members by almost 200%, and it became apparent that the actual strength of the truss material would be required to complete a meaningful analysis of the Main Hall roof structure.

Rather than risk further destruction of historic fabric, the design team recommended a research effort be undertaken to determine if the observed cut was, in fact, a previous material test and if any material test data was available in Smithsonian records.
Research and Surveys
Smithsonian Archive Research Findings

In June of 2013, SmithGroupJJR began a research effort to determine if any records of a previous test had occurred.

A search of the online records (http://siarchives.si.edu/) and microfiche images extracted from the SI internal archives revealed the following:

• The 1988 Pre-planning Structural Inspection report (Phase I – SI project #8632112) by Harry Weese Associates (HWA) indicated that a testing program be initiated with Phase II or the project.
• A meeting minutes dated 12/14/1988 indicating a postponement of the testing proposed by HWA.
• A letter from Tadjer Cohen Edleson Associates to SI dated 2/12/1990 asking if the roof trusses should be analyzed by a metallurgist. It mentions the very tests SGJJR would recommend, and by way of asking the question, suggests they had not been carried out to that date or that the information was not available at that time.
• A letter from HWA to SI indicating canceling of Phase II of the Pre-planning Structural study.
• No mention of material testing of the roof trusses occur in the 1995 RTKL planning study.

These results lead us to believe a material test had not been done during the period of 1988 to the present.

After exhausting these online and electronic databases, SGJJR began a search of the Smithsonian Archives for evidence of material testing at the Main Hall roof trusses. The only other major project known to have involved the roof trusses since the 1865 fire, was the 1968 renovation designed by Chatelain, Gauger and Nolan (CGN).

On 6/26/2013, with the assistance of Smithsonian Archivist Ellen Alkers, SGJJR reviewed six boxes of records relating to the 1968 renovation and contained documents which referenced structural work and mill tests. These were from Record unit 532 and included box#’s 84, 85, 86 and 87.

The following is a summary of findings from this research:

• A letter dated 11/7/68 from Grunly-Walsh to SI indicating the details of the trusses via the contract documents is incomplete. Hand mark-ups on the letter reference specification section 14-09, which refers to “field examination and measurement” of the steel but is not clear if mill certificate test of the existing truss were required.
• The 11/7/68 letter by Grunley-Walsh, does note that “field examination and measurement” was being performed under the contract. However this may refer to dimensional measurement of the trusses and not actual material testing.
• A follow up letter from Grunley-Walsh, dated 11/21/68 highlights that the problem of lack of detail continues.
• A letter from Grunley-Walsh to SI dated 2/4/69 refers to specification section 14-03 and mill tests being forwarded to the architect. We believe this is referring to tests for the new steal being installed which is covered in section 14-03 and not the existing material which should be covered in section 14-09.

No other reference to the SIB roof trusses, steel, or material test data were discovered in the archive boxes or record units. The SGJJR team, having exhausted available Smithsonian Records, assumed that a copy of the material coupon test data (if ever taken) was not available.

A copy of the records noted above are included and annotated in Appendix F for future reference.
The survey identified four representative types of truss which were to be analyzed in order to estimate the structural capacity of the roof. Member sizes and connection details were recorded for these trusses. Bearing details were also examined at a number of locations. The presence of existing coupon samples was noted. Details of the typical roof and ceiling void construction were noted at a number of representative locations.

Four different truss conditions were identified to be analyzed:

1. A typical truss with a tributary width of 8’-0”. This truss is not expected to be the worst case but it provides a useful basis of comparison for the worst cases.

2. A truss on the Eastern end of the main roof loaded by dunnage platforms from AHU 11 and 12 (Truss #23 and 24). This case will impose the maximum loading on any truss.

3. A truss on the Western end of the main roof loaded by the dunnage platform supporting AHU 13 (Truss #07). This case will be the worst unbalanced loading on any truss.

4. A truss in the middle of the main roof where it meets the North and South roofs (Truss #14). The top chord of the truss is unrestrained for a significant part of its length on both the North and South sides of the truss.

The AISC historical sections book does not cover sections rolled prior to 1873 but the member sizes observed on site were compared to sections dating from 1873 and sections which matched the observed sizes were adopted for use in the analysis.

Detailed results of the Main Roof survey are provided in Appendix A.
Research and Surveys
Main Roof Truss: Survey and Observations

View of attic through Main Hall Roof trusses.

Section A-A through Main Hall looking west.
Research and Surveys
Main Roof Truss: Survey and Observations

- 1968 plywood deck supported by ca. 1867 2x6 framing
- ca. 1867 purlins span between trusses at 10" (254mm) O.C.
- ca. 1867 gutter
- ca. 1867 cove framing
- ca. 1867 ceiling framing hanger
- Exterior
  - Enlarged section of existing Main Hall truss.
  - 16 100% Final Report

- ca. 1867 back to back angles span between trusses to support 2x6 wood framing
- Roofing Paper (ca. 2003)
- Batt Insulation, 64mm (2 1/2") thick, R7.2 (ca. 1968)

- Vermont Buckingham Slate (ca. 2003)
  - assume: 6mm (1/4") thick
- Plywood (ca. 1968)
  - assume: 19mm (3/4") thick
- 1/4" gap/spacer

- Interior
  - Detail cross section of Main Hall roof assembly.
  - Truss Top Chord 152mm (6")
  - (ca. 1865)
  - Purlin 75mm (3") spaced 305mm (12") on center.
  - (ca. 1865)
Research and Surveys
Main Roof Truss: Survey and Observations

The truss bearings on the south wall of the main building were examined due to concerns regarding water infiltration and the effect it might be having on the truss bearings.

Two truss bearings were inspected during the site visit on August 9th 2011. The bearings were located above the 4th floor conference room. It was reported that water infiltration through the walls had been a problem in this area. Above the ceiling plaster and paint was peeling off the wall, which suggests that water infiltration has occurred at some point.

It was not possible to obtain an unobstructed view of the bearings on the south wall. However the cast iron shoe connection to the baseplate was visible and appeared to match the connections on the north side. A number of the north wall truss connections were fully visible where the main building meets the north wing.

The truss top chord, tie bars and the base plate showed signs of rust. However there did not appear to be any significant section loss.

Truss bearing on south wall of main building. It was not possible to view the baseplate under the connection but all visible parts of the connection appear to be similar to the truss bearings on the north wall.

Truss bearing on north wall. This figure shows a typical truss bearing where the north wing meets the main building. The original lead gutter is visible in the background. Anchor bolt holes were visible in the baseplate but were full of debris.
Section 2.4

Research and Surveys
Main Roof Truss: Survey and Observations

Two steel platforms were installed as part of the 1968 renovation to support air handling equipment in the attic. The western platform is attached to trusses 7 and 8, and supports AHU 13. The eastern platform is suspended from trusses 23 and 24, and supports AHUs 11 and 12.

These units, and associated steel dunnage, represent a significant load applied to these particular truss members. Per the plan diagrams at right, these air handlers serve the following spaces:

- AHU 11 serves the Theater space on the ground floor, which is currently being used for storage.
- AHU 12 serves the ground floor Café.
- AHU 13 serves the west side Main Hall office space on the second floor.

Detail of dunnage connection to Main Hall truss joint.

Steel dunnage supporting AHU-13, bottom cord rod from Main Hall truss seen at bottom of image above plywood deck, painted orange.
Research and Surveys
Main Roof Truss: Survey and Observations

Excerpt from 2009 Existing Conditions Report showing First Floor plan area served by AHU-11 and AHU 12.

Excerpt from 2009 Existing Conditions Report showing Second Floor plan area served by AHU-13.
Research and Surveys
Main Roof Truss: Materials Testing

To determine the average capacity of the wrought iron trusses of the Main Hall, three material coupons were taken from the existing members. The intent behind taking three samples, in lieu of one, was to determine the consistency of the material in addition to its strength.

Each sample was tested to determine the Yield Stress (Fy), Modulus of elasticity (E) and the ultimate modulus of rupture (Fu) of the wrought iron. Test procedures followed ASTM E8 and ASTM A370, and details of the results can be seen in Appendix B of this report.

The locations of the samples were selected so as to have no impact on truss performance. Therefore the truss ends were identified as the ideal location for material removal as there should be little to no stress at that location. In addition, the design team was sensitive to the fact that these members are 1865 era historic fabric, and altering these in any way should be minimized.

The final locations for the test samples were selected on-site and marked off by the SmithGroup JJR team, described as follows:

Sample #1:
This sample was cut from Truss #14, at the north base plate/shoe intersection. A 9” long by 2” wide portion of the top flange was saw-cut from the truss top chord. This truss is the same that had a previous coupon taken from it in the past (date undetermined).

Sample #2:
This sample was taken from the north end of truss #13. A loose “skewback” was discovered at this location. This skewback served as support for the sheathing in the triangular void between the end of the truss top chord and the gutter edge. It can be seen in the 1865 shop drawing and is labeled as a 3x3 ½” T bar (see enlarged detail at right).

Sample #3:
The 1857 era gutter is present and visible at the top of the bearing wall where this truss connects. During the 1968 renovation, a gable end was added to the North Stair Hall, perpendicular to the Main Hall, which made the gutter in this area obsolete. It appears the skewbacks were abandoned in place.

Since the material was identical in profile and had flange notches to receive the purlins, the design team felt it safe to assume that this piece was of the same era and material as the truss top chord. Testing this piece represented a less destructive, but appropriate, alternate to cutting another truss top chord. The skewback was held in place by a single bolt. The bolt was loosened and the skewback removed for testing.

The results of these tests indicated that the material has a yield stress of 37 ksi and a Young’s modulus of 29000 ksi.
Research and Surveys
Main Roof Truss: Materials Testing

Main Hall attic plan showing truss numbering and location of material samples (coupons)

Enlargement of Main Hall truss shop drawing (Drawer 156, sheet 26) showing skewback between truss end and gutter, and horizontal purlins.
Research and Surveys
Main Roof Truss: Materials Testing

Sample #1 - cut from truss flange (Truss #14)
Sample #2 - Removed (unbolted) skewback (Truss #13)
Sample #3 - Tested loose purlin (similar to those in photo above)
Research and Surveys
Main Roof Truss: Materials Testing

TEST REPORT

REVISED: 11/26/13

ECS MID-ATLANTIC, LLC
ATTENTION: CRAIG HENDRY
14026 THUNDERBOLT PLACE, SUITE 100
CHANTILLY, VA 20151-3232

DATE: November 21, 2013
PO NO: ECS Job #01:22193
LEHIGH NO: P-65-14
Samples 1, 2 & 3
PAGE: 1 of 1

SAMPLE DESIGNATION: (3) SAMPLES: SAMPLE 1 IS FROM THE TOP FLANGE OF A TRUSS,
SAMPLE 2 IS A PIECE OF A TRUSS EXTENTION & SAMPLE 3 IS FROM A STEEL PURLIN
ECS JOB #01:22193 – SMITHSONIAN CASTLE – STEEL TESTING

MECHANICAL PROPERTIES (Per ASTM A370-12a)

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Results are for information only.

*Revised to correct report date, include modulus results and charts.

Lehigh Testing Laboratories, Inc.

Kevin M. Sexton
Kevin M. Sexton, Mechanical Testing Technician
The survey identified the typical roof framing in the cloister area. Member sizes were noted where possible.

The West Range (cloister) roof appears to be constructed from terracotta blocks spanning onto back to back angles at approximately 3 ft on center. It was not possible to measure the vertical leg of the upturned double angle sections. The back to back angles span onto S-section beams at about 8’-6” o.c. below the terracotta units. These beams span approximately 10’-6” between the exterior wall and the interior loadbearing wall. The record drawing indicates that the beam is continuous over the interior wall and this has been taken account of in the beam calculations. The roof assembly is assumed to be as shown in appendix A.

There is evidence of rust on all visible beams and angles. This may be due to water ingress or to a moist environment within the room which contains mechanical services and is not air conditioned. The rust does not appear to have caused any significant section loss on any of the beams.

Cross section through West Range looking west.
Research and Surveys

Cloister Roof: Survey and Observations

View of Cloister Roof from Exterior

View of Cloister Roof from Interior showing terra cotta planks

Detail cross section of Cloister roof assembly.
Research and Surveys
South Tower : Survey and Observations

The South Tower was inspected during a site visit on 9th August 2011. The A/E team investigated a comment in the HSR which identified an unusual bracket detail and cracking in the wall finishes immediately adjacent to the bracket. See Appendix A and D for further observations and photos.

The top floor of the tower appears to be a brick arch floor which spans north-south. Brick arches span onto built in beams (possibly of wrought iron) which span north-south. The span of these beams is supported by a beam running east-west which is located immediately beneath the brick arch floor. The beam beneath the brick arch floor bears onto stone corbels which are supported by diagonal metal circular hollow sections. Cracking in the wall finish was visible during the survey. The lath and plaster wall finish on the west wall was removed locally to allow closer inspection. This revealed that the masonry behind the racking was intact and that the cracking appeared limited to the plaster finish. This exploration also revealed that the beam bearing on the wall was insignificant. Therefore it appears that the beam relies on the stone corbels for support at each end.

The beam bears onto a stone corbel at each end. See figure at right. The stone corbels appear to be supported by circular metal hollow sections. It appears that these are mortared into a pocket within the underside of the stone bracket. It was not clear how the metal hollow circular section connects into the wall as the connection was behind the lath and plaster wall finish.

The mill stamp on the beam appears to say “Phoenix Iron, Phlia.” The AISC historical sections book gives an allowable stress of 12 ksi for beams rolled in 1885 by the Phoenix Iron Company.

This corbel was re-visited shortly after the August 2011 earthquake. While damage did occur to the exterior masonry of adjacent chimney, no additional cracks or damage appeared on the interior wall surrounding the corbel nor at the beam connection.
Corbel detail under roof of South Tower. Cracking is evident in the lath and plaster wall finish to the left of the corbel. The door on the right of this photo leads to the chimney shaft. The cracking in the finish plaster was not visible on the interior side of this shaft, therefore it is assumed that the cracking is limited to the plaster finish.

West end of corbel after removal of plaster and revealing beam seating. The beam stops short of the masonry wall, and is therefore relaying entirely on the corbel for support. In the area of this removal no evidence of cracking could be seen on the masonry backup wall.
The following codes and standards were used:

Loading - ASCE 7-05 Minimum Design Loads for Buildings and Other Structures.*
Historical Structural Steelwork/Ironwork - Historical Record - Dimensions and Properties - Rolled Shapes - Steel and Wrought Iron Beams and Columns - As Rolled in USA, Period 1873 to 1952, Ed. Herbert W. Ferris, 4th printing, 1964, AISC.

Other loading assumptions were as follows:

Wind:
- Basic wind speed, V: 90 mph
- Exposure Category: B
- Wind Importance Factor Iw: 1.15
- Building Category: III
- Internal Pressure Coefficient: +/- 0.18

Snow:
- Flat Roof Snow Load Pf: 22 psf
- Ground Snow Load Pg: 28 psf
- Snow Exposure Factor Ce: 1.0
- Snow Load Importance Factor Is: 1.1
- Thermal Factor Ct: 1.0
- Snow drifting per ASCE 7-05

* District of Columbia Construction Codes Supplement of 2008 DCMR 12A Building Code Supplement was also considered in the analysis. This code requires a basic snow load of 25 psf whereas ASCE 7-05 indicates that the flat roof snow load is 22 psf. The analysis was based on ASCE 7-05; since the trusses are overstressed for this lower loading they will be slightly more overstressed for the higher loading given by the DC building code.
Chapter 3
Analysis and Discussion
Analysis and Discussion

Main Roof Trusses

The analysis was undertaken using the SAP computer program. A linear static finite element model was created. Trusses were modeled as two-dimensional trusses. One support was modeled as a pin and the other as a roller, so that the truss is able to spread.

Load combinations used in the analysis were based on ASCE 7-05 (ASD) and were as follows:

1. 1.0D
2. 1.0D + 1.0L
3. 1.0D + 1.0(Lr or S)
4. 1.0D + 0.75L + 0.75(Lr or S)
5. 1.0D + 1.0W
6. 1.0D + 0.75W + 0.75L + 0.75(Lr or S)
7. 0.6D + 1.0W

Where:

D = Dead Load
L = Live Load
Lr = Live Load at Roof
S = Snow Load
W = Wind Load

Earthquake loads were not included in the analysis since seismic analysis and strengthening are outside the scope of this report.

Loads were calculated per ASCE 7-05. The results of the analysis indicate that some members are overstressed. Therefore the truss was not analyzed for the DC building code snow loads which are greater than the ASCE 7-05 snow loads.

Loads from the roof were modeled as a uniformly distributed line load on the top chords. Loads from the wrought iron hangers were modeled as point loads applied to the top chord. Refer to appendix C for the loading spreadsheets.

Moment releases were assumed as shown in the drawing below.

The SAP program was used to obtain member forces under each of the load combinations. The envelope feature of the program was used to obtain the highest and lowest loads in each member. These member forces were then used to calculate the maximum stresses in each member. The maximum stresses in each member were then compared with the allowable stress limit based on the test results. It was assumed that the compression capacity of the wrought iron rods was zero and a check was undertaken to ensure that the rods are not subject to compression under any combination of loads.

Elevation from SAP computer model. Member releases are represented by green dots. The support on the right is modeled as a roller and the support on the left is modeled as a pinned support. Numbers indicate the member numbers.
Analysis and Discussion
Main Roof Trusses

Four different trusses were identified:

1. A typical truss with a tributary width of 8'-0”.
2. A truss on the Eastern end of the main roof loaded by dunnage platforms from AHU 11 and 12. This case will impose the maximum loading on any truss.
3. A truss on the Western end of the main roof loaded by the dunnage platform supporting AHU 13. This case is the worst unbalanced loading on any truss.
4. A truss in the middle of the main roof where it meets the north and south roofs. The top chord of the truss is unrestrained for a significant part of its length on both the north and south sides of the truss. At other truss locations this member is restrained from buckling by being connected to the roof and purlins. At the intersection of the main roof and the north-south roof, part of the top chord near the bearing points on the walls is not in contact with the roof and is not restrained against buckling.

The AHU loads were estimated at 600 lbs each based on a comparison with similar equipment.

See appendix C for details of the analysis.

Axial forces, bending moments and shear forces were taken from the SAP model. Calculations were performed in excel. Apart from the trusses located at the intersection of the N-S wing with the main building which have partially unbraced top chords, it was assumed that the top chords were fully restrained. The maximum stress in each member was compared with the allowable stress. In addition connections were checked in the spreadsheet.

Attic plan showing the four truss loading conditions analyzed.
**Analysis and Discussion**

**Main Roof Trusses**

**Discussion of Truss Analysis:**

ASCE 7-05 requires a minimum live load of 40 psf for maintenance catwalks, such as at the plywood deck within the ceiling void. This load will have a significant effect on the member forces in the truss. Calculations indicated that a live load of 40 psf would cause the truss members to be significantly overstressed. Given the use of the space, the code required live load is very high. As a comparison the live load for a parking garage is 40 psf. Therefore the live load at the ceiling void was reduced to a more realistic value of 5 psf; this assumes that access to the ceiling void is limited to a small number of individuals at any one time. Any activities which require more people or equipment in this space would need to be assessed on a case by case basis. In addition the roof live load was taken as 20 psf per ASCE 7-05 instead of 30 psf per DC building code.

The slate tiles were assumed to contribute 10 psf to the dead load. Live load on the roof was assumed to be 20 psf per ASCE 7-05 table 4-1 instead of DC building code value of 30 psf. Main Roof Loading is broken down in further detail in appendix C.
Analysis and Discussion
Main Roof Trusses

Discussion of Truss Bearings:

The record drawing shows that a gutter is built into the wall. It is understood that during rainstorms water sometimes overflows from the gutter and flows out between the crenellations and down the façade. The gutter is outside of and below the baseplate so if the gutter overflows it is unlikely that the baseplate would get wet. This may explain why the corrosion damage to the truss bearing does not appear to be significant despite obvious water damage to the walls.

The north wall baseplates have anchor bolt holes but there do not appear to be any anchors to hold the trusses down. Therefore it appears that the truss bearings do not have a positive vertical anchorage into the walls. Checks were undertaken to verify whether net uplift occurs at the truss bearings under any of the load combinations in the AISC specification. The results indicate that net uplift does not occur.
Analysis and Discussion
Main Roof Trusses

Discussion of Roof Thermal Performance:

As it became apparent that snow loading would be a concern for the Main Hall roof, and mitigation measures would need to be considered, an analysis was undertaken to determine the thermal performance of the existing conditions. In the absence of destructive testing, assumptions were made of the existing roof build up. Per the table at right, R-Values for each component were determined based on published data for each material, resulting an overall R-Value of 9.23.

This information was used to determine the typical heat loss of the roof build up and formed the assumption used to size a radiant snow melt system. In order for a heating system to assist in snow melting, from the interior, the system would need to overcome this R-Value to be effective.

It is recommend that for the next phase of this project that temperature monitoring in the attic space be performed during the cold winter months. Recommended locations for temperature sensors would be at the ends of building where temperature is likely to be the coldest. Should a higher rate of heat loss be detected than that estimated for this project, a reduction in the Btuh for the snow melting system could be considered, thereby reducing the cost of the installation.
## Analysis and Discussion
### Main Roof Trusses

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<th>Component</th>
<th>R value</th>
<th>U-value (per component)</th>
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<tr>
<td>Slate tiles</td>
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<tr>
<td>Air barrier - vapor permeable felt</td>
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<td>3/4” Plywood</td>
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Total roof R value: 9.23 0.11

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<td></td>
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</table>
Analysis and Discussion
Cloister Roof

Calculations were performed on the back to back angles and on the beams. The depth of the back to back angles was not stated on the record drawing but the drawing indicates that they are at least as deep as they are wide (if not deeper). Therefore the angles were assumed to be 2”x2”x3/8” in order to estimate the capacity of the roof.

An exact match for the beam was not found in the AISC historical sections book. Calculations were performed on the nearest available match which was of the same depth and approximately the same flange width.

Results:
Two calculations were run on the Cloister roof framing.

The first assumed allowable stress was taken to be 10 ksi. This is based on the allowable stress of wrought iron at the time this part of the building was constructed.

The second assumed an allowable stress matching the values discover by the material tests from the Main Hall roof as these materials are likely to be contemporary. This value is 22 ksi.

Scenario 1: Assuming 10 ksi allowable stress:
The beams can take 69 psf total load. Subtracting 29 psf for dead load gives a live load capacity of 40 psf.

Scenario 2: Assuming 22 ksi allowable stress:
The beams can take 153 psf total load. Subtracting 29 psf for dead load gives a live load capacity of 124 psf.

The maximum applied load (snow drift) is 29 psf dead plus 82.8 psf snow = 112 psf.

Discussion:
The results of the calculations indicate that the beams can withstand a uniform snow or live load of 40 psf over the entire cloister roof. However, while the beam calculations indicate that the beams should meet code requirements for live load but it is possible that the beams will be overstressed under the maximum code snow drift, depending on which allowable stress is assumed for the calculation. Under the worst case ASCE 7-05 snow drift loading and assuming an allowable stress of 10 ksi the beams in this area are overstressed. The worst case snow drift loading is 82.8 psf. This is equivalent to approximately 5 ft of snow.

The snow drift calculation depends on a large number of variables and the maximum snow drift on the cloister roof is due to drifting snow from the main roof. It is possible that the maximum drift has not occurred due to the poor thermal properties of the roof and the fact that the calculation assumes that snow would have to accumulate on the main roof before being blown across and down onto the cloister roof. The worst case snow drift in the Cloister roof area is due to leeward drifting off the main roof, so if the snow does not accumulate significantly on the main roof, drifting might not occur to the extent predicted by the code.

The thermal properties and slope of the main roof may also limit the potential for snow to accumulate on the main roof. In addition, even if snow did accumulate on the main roof there would need to be a combination of snow and wind to create drifting on the cloister roof. The roof members do not appear to show any signs of structural distress; therefore it is recommended that no immediate action is required to strengthen these beams.

Refer to loading spreadsheet in appendix C for further details.
Analysis and Discussion

Cloister Roof

Cloister roof ca.1920

Cloister roof ca.2009

Detail of Cloister roof beam and exterior wall connection.
Analysis and Discussion

South Tower

The stone corbels appear to be supporting the beam that in turn supports the brick arch floor of the roof of the South Tower. The span of approximately 25 ft is long for a brick arch floor and the beam may perhaps have been installed to reduce sagging. It is also possible that it could be an original beam dating to the 1865 era reconstruction.

Other conjectures that would explain the presence of the corbel (as opposed to a traditional beam pocket) is a possible obstruction in the wall cavity, or the beam may have been ordered too short and the stone brackets were added to compensate. In any case, the irregular and unconventional nature of the corbel suggests a field correction or repair.

Calculations indicate that the magnitude of the shear acting on each corbel is approximately 25 kips. Due to the unusual form and material of the corbel construction it is not possible to check this connection detail against any codes. Therefore it is not possible to assess the structural capacity of this connection.

Due to the uncertainty of the connection it is best practice to provide an alternative load path in case the existing connection is not sufficient. A repair detail to achieve an alternate load path is presented in Chapter 4.
Section 3.3

Analysis and Discussion

South Tower

East corbel detail

West corbel detail

Corbel detail and beam supporting roof of South Tower.
Chapter 4
Conclusions and Recommendations
Conclusions and Recommendations
Main Roof Trusses

Conclusions:

The structural analysis indicates that the trusses do not meet current code requirements for the full snow load under criteria presented in Section 3.1. However, there are possible alternative load paths which were excluded in the analysis. For example, the bearing details may provide some degree of horizontal restraint. In addition it is possible that due to the poor thermal properties of the roof, the full snow drift loading will not be able to accumulate. Finally, the calculations and loading criteria rely on a number of significant assumptions such as plywood thickness, slate tile thickness, and deadload of mechanical equipment which were based on modern conventional construction materials. While these assumptions represent best practice and a reasonable level of conservatism, a reduction in any one of the above could result in a more favorable analysis of the trusses. For example, if the snow load portion of the live load is omitted, the trusses are not overstressed. The analysis indicates that all trusses are overstressed by at minimum of 34%. The load assumed in this analysis includes full code based snow load per ASCE 7-05 and a live load of 5 psf in the attic. The trusses which support dunnage platforms (cases 2 and 3) are slightly more overstressed (maximum 38%) than the other trusses and more of their component parts are overstressed compared to the typical truss (case 1) – 5 members are overstressed at the dunnage trusses compared with 2 members at the typical truss. The analysis also indicates that the trusses which are partially unbraced at their top chords (case 4) are overstressed by the same amount as the typical truss (34%) but they are also overstressed in the top chord by 11% whereas this is not the case for the typical truss.

Overstress means that the member forces have exceeded the code limits for design. The code applies a factor of safety at this limit and the degree of overstress is such that it reduces the effective factor of safety on the design, but it is not so high that it exceeds the material strength.

This analysis assumed 20psf live load for snow loading. By back-calculating, it was determined that the trusses can take up to 7 psf of snow before they become overstressed. This indicates that the roof can successfully accommodate a very limited snow load before approaching the stress overstress point of the trusses. 7psf equates to approximately 6” of snow accumulation.

Recommendations:

In response to the potential for overstress in the Main Hall Roof trusses, three recommendations for temporary risk reduction have been developed. These recommendations are intended to be short term actions that minimize the chances of overstress by either reducing the potential for snow accumulation or reducing the threat of damage on the fourth floor due to specific heavy loads being applied to the trusses above. It is assumed that a future major renovation will address Main Hall Roof structural deficiencies in a manner that will be more sensitive to the historic fabric of the building and in the context of a full roof replacement and/or envelope performance upgrade.

The first recommendation (A.1) employs a strategy of adding heat to the interior attic in an effort to melt snow during significant snow events. While this option does not address the code loading requirements since it is a mechanical solution, this option would minimize disruption of the Fourth floor occupants and applies a solution to the entire roof area.

The second recommendation (A.2), is a structural option whereby the load of the two main AHU dunnage platforms is transferred to the Fourth floor beams by way of posts passing the office spaces. This recommendation only addresses the specific loading of the AHUs and steel dunnage platforms. This alternative can be pursued in addition to A.1 for further risk reduction.

The third recommendation (A.3) is to entirely remove AHU 11 and 12, and their associated steel dunnage platforms. This would eliminate the risk of damage to the Fourth floor from these units in the event of a major overstress of the roof trusses. AHU 13 is critical to the function of the Second floor and therefore cannot be recommended to be removed. This option can also be pursued in addition to A.1 for further risk reduction.
Conclusions and Recommendations

Main Roof Trusses

Recommendation A.1 – Add Heat to Attic

The SIB currently benefits from a passive built-in snow melt system currently integrated in the tempered attic space under the roof. Existing unit heaters temper the attic above 65 degrees F. for freeze protection of piping systems routed through the attic. With a roof assembly R-value of 9, this heat automatically dissipates through the roof to warm snow/ice on the roof surface. However, this rate of passive snow melting though heat loss is not sufficient to melt snow at a rate that would combat a heavy snow event and limit accumulation to less than 6 inches.

The design intent of Recommendation A.1 is to create a system, similar in strategy, to an exterior snowmelt system, but applied from the interior. This would be achieved by install electrical fin tube units suspended from top chords of the Main Hall trusses. During predicted snow events of greater than 6” accumulation, these heating elements which could be turned on to raise the temperature of the roof surface in an effort to melt the snow and limit overall accumulation. See the section diagram below for the proposed design concept.

Adding a finned tube system sized for both heat conduction and heat of fusion characteristics of snow will not only boost the current snow melting properties of the roof, but also protect the structure and existing exterior appearance of the SIB roof. Assuming an incidental snowfall rate of 0.1 inches per hour, a recommended minimum heat output would be 76.28 Btuh/sf of roof area, which is estimated to be a total of 936,720 Btuh for the Main Hall attic space. This level of heat output is expected to leave a thin layer of snow on all or part of the roof during 75% of the events during the season. See table on page (49) for preliminary calculations used to estimate this minimum heat output.
A preliminary design was developed for Cost Budgeting purposes, and the calculations used for that development can also be seen on the table on page (49). Using a conventional baseboard type element that produces 100W/lf, it was determined that 14 rows that cover the 200 foot length of the Main Hall would produce 955,920 Btuh. These 14 rows would be divided equally with 7 on each side of the ridge, and would be suspended from unistrut. The goal in the exact placement would be to provide as even a disbursement as possible.

Electric and hot water based fin tube units were considered in the preliminary design and electric units were found to be preferred and are included in the Cost Budget. Electric finned tubes have the advantage of ease of installation, lightweight, and minimal upgrade requirements of other building systems. Hot water finned tubes do have a slightly lower first cost than electric units, but would require the use of the building hot water system and therefore would require additional use of GSA steam for hot water conversion.

In order to service these new electrical units, conduit would need to be run from the basement to the attic, likely through a chase in the SE corner if the building. A new panel will also need to be installed in the basement to handle the new units.

To supplement the fin tube system on the interior, it is further recommended that a heat trace system be installed in the Main Hall gutters and downspouts, to reduce the risk of snow and ice buildup. This would be roughly 500lf of heat tape and is included in the Cost Budget for this recommendation. This should be able to be installed via maintenance lifts (JLG/Reachmaster, etc.) and scaffolding should not be necessary.

A conventional snow melt system that would be applied under the slate and integrated with roof sheathing was considered. This system has the advantage of being in close contact snow in order to achieve complete melting and evaporation more efficiently. There are also examples of this type of system being installed successfully on historic buildings. However, such a system would require complete roof replacement and re-sheathing of the area over the Upper Main Hall. This was determined to be cost prohibitive for a temporary solution considering the scope of re-roofing, new radiant piping, and related scaffolding work. Such an effort would be more appropriate to consider during a major building wide renovation.

Adding heat to the attic via a forced hot air system was considered. While some re-circulated air could be used for this type of system, a moderate amount of fresh makeup air would be required. This make up air would involve creating a new louver or hood which would require altering the building envelope in some way. The air based system could create a uniform heat and positive pressure in the attic space, but would likely be less efficient than a fin tube based system that could more directly apply heat to the roof underside. Finally, an air based system would require mounting more equipment and additional ductwork to the attic framing. For these reasons, a forced air supplemental heating system was determined to be undesirable, and less preferable to a fin tube based approach. However this solution may be worthy of consideration during a future major building renovation project.

**Pros and Cons of this recommendation are as follows:**

**Pros:**
- Recommendation A.1 applies a uniform treatment of Main Hall roof surface, relieving stress from all trusses.
- Recommendation A.1 does not impact the 4th floor occupancy.

**Cons:**
- Recommendation A.1 is a mechanical solution and therefore cannot guarantee compliance with the code loading criteria.
- Recommendation A.1 does not address the specific risk of the AHU’s and their steel support dunnage.
- Recommendation A.1 is the most expensive solution proposed in this report.
Conclusions and Recommendations
Main Roof Trusses

Snowmelt Minimum Requirement Calculation

Sensible Heat Flux (ASHRAE A50.2-(3))

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<td>rho-water</td>
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<td>lb/ft³</td>
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<tr>
<td>s</td>
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<td>snowfall rate, in/h</td>
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<td>cp-ice</td>
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<td>cp-water</td>
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<tr>
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Sensible Heat Flux (ASHRAE A50.2-(4)) MELTING SNOW

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<td>hif</td>
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*NOT USED AS DATA FOR EVAPORATION AND RADIATIVE EFFECT NOT AVAILABLE*

Recommendation A.1 Design

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Conclusions and Recommendations
Main Roof Trusses

Recommendation A.2 – Support AHU Dunnage

The design intent of this recommendation is to divert the deadload of the AHU’s and associated dunnage to the Fourth floor deck. This equipment and associated framing represent the greatest threat to occupants below in the event of damage to the roof trusses due to an overstress condition. While this recommendation does not address the structural deficiency of the Main Hall trusses, directly supporting these units reduces the potential risk of injury to occupants of the Fourth floor.

Recommendation A.2 would be achieved by connecting a new vertical posts to the dunnage beams at the west (AHU 13) and east (AHU 11/12) platforms. For the purposes of this preliminary design it was assume the posts would be steel lally columns spaced approximately 6’ on center. Therefore a total of 6 posts are required for the west platform and 8 for the east. A bracket or clamp would need to be selected to connect the lally columns to the steel dunnage beams. These posts will need to be inserted through occupied spaces on the Fourth floor and may be disruptive to install (see plan diagram below).

Since the load points of these columns will likely not align with the Fourth floor beam locations, a spreader will be required to distribute load to the deck. Wood spreaders at least 12”x12” in plan should be provided at each column. For the purpose of this Cost Budget, the spreaders were assumed to be constructed of 2x4 material, long side vertical, with sheets of plywood nailed to the 2x4’s top and bottom to bind the assembly together.

Recommendation A.2: Plan diagram of 4th floor showing impact of new posts and spreader bases under AHU 11 &12 (East), and AHU 13 (west).
Conclusions and Recommendations
Main Roof Trusses

An alternate option to support all trusses at the major node points was considered and studied. The intent would be to divert some snow load to the Fourth floor deck in an effort to make the roof code compliant. This would require 50 posts penetrating to the Fourth floor deck. In addition, since the roof trusses and Fourth floor beams do not align, spreader beams would need to be installed on the floor surface running east-west for most of the length of the Upper Main Hall in order to properly transfer the load to the fourth floor beams. This level of intervention on the Fourth floor would be significant and, through discussions with the Smithsonian, was determined to be too invasive an option to be considered further.

This recommendation can be implemented in addition to Recommendation A.1 – Add heat to attic.

Pros and Cons of this recommendation are as follows:

Pros:
- Recommendation A.2 addresses the greatest dead load risk elements.

Cons:
- Recommendation A.2 does not provide compliance with the code loading criteria for the roof.
- Recommendation A.2 includes no change to typical truss loading and only addresses dunnage areas.
Conclusions and Recommendations
Main Roof Trusses

**Recommendation A.3 – Remove AHU’s and Dunnage**

The design intent of this recommendation is to reduce risk by removing the heaviest objects occupying the attic, where possible. This proposal is to remove AHU 11 and 12 and their associated steel dunnage platform and framing. At the 1/10/14 meeting with the Smithsonian it was determined to leave AHU-13 in place.

AHU-11 serves the Theater space of the Lower Main Hall, which is currently used for storage. AHU-12 serves the Café in the Lower Main Hall. Plan diagrams showing the extent of these spaces can be seen on page 19 of Section 2.4, this report. Removal of these air handlers would mean the spaces could not be used for any function other than storage. Ventilation air would need to be provided to these spaces, which could be accomplished by fans in the windows. These spaces would also need to be maintained at a minimum temperature of 65 degrees F in order to prevent pipes in adjacent spaces from freezing or damage. The existing fan coil units may be able to provide this minimum temperature.

The Cost Budget for this recommendation only includes the demolition of the air handler units and related steel dunnage in the attic of the Main Hall. All associated piping is assumed to be abandoned in place and capped as necessary.

This recommendation can be implemented in addition to Recommendation A.1 – Add heat to attic. It can also be combined with Recommendation A.2 for the support of the west platform serving AHU-13.

**Pros and Cons of this recommendation are as follows:**

**Pros:**

- Recommendation A.3 permanently addresses some of the greatest dead load risk elements.

**Cons:**

- Recommendation A.3 does not provide compliance with the code loading criteria for the roof.
- Recommendation A.3 includes no change to typical truss loading and only addresses dunnage areas.
- Recommendation A.3 does not address the load of AHU-13 and dunnage.
Conclusions and Recommendations

Main Roof Trusses

Other Notes and Future Considerations specific to the Main Hall Roof:

Total roof replacement of the slate with a metal roof was briefly discussed. While this may limit the risk of snow accumulation, it would not solve the truss overstress concerns as calculated with modern load criteria that assumes a uniform load of snow. In addition, this option would obviously significantly alter the appearance of the SIB in a negative manner, and would not comply with the recommendations established in the 2009 Historic Structures Report. As noted in Recommendation A.1, roof replacement with slate and an integral radiant pipe system may be considered during a future major renovation project.

Removal of the existing roof insulation was also considered, but is not recommended. This action may create unintended stress on the existing mechanical systems which have been operating with the insulation in place since 1968. In the summer, this may create particular heat load problems with other mechanical systems located in the Main Hall attic.

Some Main Hall roof truss members show minor signs of corrosion, especially at the bearing locations, but there does not appear to be any significant section loss. In the event of a major building renovation these members should be surveyed in detail, particularly the portions that are buried within masonry walls under the gutters, and corrected as needed.

In the long term, it is recommended that the trusses be re-analyzed before any future major renovations occur in order to determine the strengthening requirements. It should be noted that the addition of insulation, beyond the existing, increases the risk of overstressing the trusses as this will increase the potential for snow loading. Future renovations will need to take this into consideration when reviewing energy performance and strategies for building improvement.

Repair and/or improvement of the gutter detail should be considered during a major renovation. A permanent and well-designed integral heat trace system should be considered for the gutters to reduce snow accumulation which is historically retained by the original parapet design.
Conclusions and Recommendations

Cloister Roof

Conclusions:

Preliminary calculations indicated that the S-section beams are overstressed, however this assumed the allowable stress of the beams to be 10 ksi. The long catch length on the main roof causes significant snow drift loads of up to 82.8 psf at the cloister roof. The calculations were repeated assuming an allowable stress of 22 ksi which is the allowable stress based on the test results from the Main Roof trusses, and a more likely representation of the actual material strength. At this higher allowable stress, the angles and the beams can support the full code snow load. The analysis indicates that the roof can support up to 40 psf live or snow load assuming an allowable stress of 10 ksi. Therefore the roof is adequate for current code live load. The code snow drift load at this location is very high because of the length of the main roof and the height difference between the main roof and the cloister roof. However it is possible that the full code snow drift load does not accumulate on the cloister roof due to its poor thermal properties and due to the snow accumulation on the main roof (from where it is assumed to drift onto the cloister roof) being lower than predicted by code.

Rusting was observed in many of the beams and angles and may have caused the plaster to spall off from the underside of the roof. However there did not appear to be any significant section loss in any of the members. Therefore the rusting is not likely to affect the structural capacity of the members.

Recommendations:

The long term recommendation for addressing the Cloister beam deterioration is to replace the aging roof system and substrate as noted in the 2009 Existing Condition Report. The cause of the corrosion should be identified through the use of probes and steps should be taken during the design of a replacement roof to prevent further corrosion from occurring. A project of this caliber would be permanent in nature and best performed in the context of a future building-wide renovation that can balance Historic Preservation and building envelope performance.

Since this roof appears to meet code requirements, except for cases of extreme snow drift (5’ of depth), only a minor repair to remove rust and repaint beams is recommended as a temporary measure. Snow drifts from large snow events should be removed as soon as possible from this roof to avoid overstressing of the members.

Recommendation B.1 - Strip and Repaint

The objective of this recommendation is to clean up the interior rusted roof beams in an attempt to curb further corrosion until a major project allows for a wholesale roof replacement. The scope of this repair is to remove loose rust from the roof framing beams by hand, using bushes and appropriate cleaning agents. After cleaning, the wrought iron beams would be repainted with a corrosive resistant paint.

Other notes and Considerations:

The available record drawings did not provide information on the depth of the back to back angles, which are partially hidden within the terracotta tile deck, so it was not possible to reach a conclusion about the capacity of these secondary members. However, observations and photographs on site did not identify any signs of structural distress in these members. As part of a future major building renovation, destructive probes should be undertaken to reveal the depth and condition of the back to back angles.
Conclusions and Recommendations

Cloister Roof

North elevation showing cloister roof during a snow event.

Second floor reflected ceiling plan of Cloister showing area to be repainted (yellow).
Conclusions and Recommendations

South Tower Roof

Conclusions:

The beam which supports the brick arch roof framing of the South Tower appears to be entirely supported by the corbels on either end and not engaged in the load bearing masonry walls. The corbels appear to be constructed of Seneca sandstone similar to the exterior cladding for the SIB. Due to the unusual nature, shape, and composite materials of the corbel construction, it is not possible to verify the existing corbels by conventional calculation methods. While this detail has been in place for many years and has shown no sign of major failure or distress, best practice is to assume that this connection cannot be relied upon to accommodate the shear forces imposed by the beam.

Recommendation C.1 - Add Beam Brackets

The design intent of this recommendation is to augment the corbel support with a bracket that enables the beam to bear onto the wall. This will provide an alternative load path in case the corbel is structurally deficient. The sketch at right illustrates the proposed concept design for a detail that would allow the beam to be supported by the wall. This involves adding a bracket bolted to the roof beam and grouted into a masonry pocket. This detail would need to occur at both ends of the beam.
Conclusions and Recommendations

South Tower Roof

Recommended repair for South Tower corbel
The design scope for the Structural Survey and Roof Repair project scope includes five (5) individual recommendations. The first three (3) recommendations covers the SIB Main Hall Trusses. The last two are for the Cloister roof and South Tower roof.

- Recommendation A.1 involves adding Heat to the Attic to melt snow on the roof.
- Recommendation A.2 includes lally columns to support the existing AHU dunnage.
- Recommendation A.3 includes removing the AHU’s and dunnage.
- Recommendation B.1 includes stripping and repainting corroded beams supporting the terracotta planks on the Cloister roof.
- Recommendation C.1 pertains to the addition of four (4) brackets to add support to the existing beam supporting the South Tower roof.

Recommendations are presented as separate stand alone cost budget items. The Smithsonian can elect to pursue any or all in combination. A detailed Cost Budget report is provided in Appendix D.

The following general assumptions were made for the Cost Budget:

- The Cost Budget is to be used for relative decision making and establishing budgets for future project funding. It is not intended to be a cost estimate as detailed design documents were not developed for this project.
- All recommendations are intended to be temporary, therefore only first costs are presented. Life cycle costs were not considered for this project.
- Work is assumed to be performed in 8 hour shifts, during normal business hours, Monday thru Friday with some night time and weekend work required for incidental service outages and connections.
- With consideration given to the finished and fitted condition of the SIB as well as logistical concerns for working in confined spaces, the Cost Budget assumes a productivity factor of +30% above published product cost data for labor resources to account for these conditions.

Detailed assumptions specific to each recommendation are as follows:

**Recommendation A.1 – Add Heat to Attic**

The objective of this recommendation is the heat up the attic on the interior to melt snow.

In order to heat up the attic to be able to melt roof snow, an additional 955,000 Btuh (approximately) needs to be provided for the Main Hall roof attic, dispersed as even as possible. The Main Hall is approximately 203 feet long in the east-west dimension.

- Vulcan makes units in 4’ and 8’ lengths which create 100watts of heat per linear foot. 14 rows of these units, 200 feet long to produce a total of 954,800 Btus/hr. (14rows x 200LF/row = 2800 LF x 100W/LF = 280000 W x3.41Btu/W = 954,800 Btus/hr.)
- Each roof slope would receive 7 of these rows and they would be hung from the underside of the attic via unistrut, see section diagram.
- Conduit would need to be run from the basement to the attic, likely though a chase in the SE corner if the building. Some drilling of masonry in the attic to access the shaft is assumed. The length of run is approximately 50ft vertically from attic to basement. Assume another 50ft of run in the basement horizontally to a new (TBD) panel location.
- A new panel will need to be installed in the basement to handle the new units.
- Heat trace to 400lf of exterior gutters and 8 downspouts (10 ft vertically) is assumed. This can be installed via maintenance lifts (JLG/Reachmaster, etc.). Scaffolding should not be necessary to install the heat trace.
Recommendation A.2 – Support AHU Dunnage

The objective of this recommendation is to support steel beams that support the air handling units of the attic. Support is assumed to be by steel lally columns from the fourth floor level to the dunnage bottom, thereby diverting their load to the fourth floor deck.

- 6 posts for the west platform and 8 posts for the west are assumed.
- At the fourth floor, for each post, is a wood spreader, 12”x12” in plan. The spreaders would be constructed of 2x4 members, long side vertical, with sheets of plywood nailed to the 2x4’s top and bottom to bind the assembly together.
- The existing ceiling is ACT, however an allowance is included for incidental demolition that may be required of some GWB walls that may be directly below a steel beam above.
- The lally columns are assumed to be clamped to the steel w-sections.

Recommendation A.3 – Remove AHU’s 11&12 and Dunnage

The objective of this recommendation is to reduce risk by removing the heaviest objects occupying the attic. This is a demolition only cost, mechanical systems are not being replaced for this recommendation.

- Air Handlers #11 and #12 are removed entirely.
  These are the two units on the eastern platform.
- AHU’s weigh about 600 lbs each.
- All piping is capped, ductwork is left in place.
- Steel dunnage is removed, approximately 60 linear feet of W6 beams and associated framing.

Access to the attic is via ladder from the 4th floor only. Demolition material could be lifted out of a small man-sized attic hatch in the North Stair Hall roof to avoid traffic in public spaces.

Recommendation B.1 – Strip and Repaint

The objective of this recommendation is to clean up the interior rusted roof beams in an attempt to curb further corrosion until a major project allows for a wholesale roof replacement. Access to this area is via a stairwell from the basement, no elevators. Mechanical equipment in the room makes for tight quarters in some locations.

- Loose rust from 186lf of beams is to be removed using wire bushes and cleaning agents. These are historic beams and should be handled appropriately.
- Wrought iron beam bottoms would be coated with corrosive resistant paint.

Recommendation C.1 – Add Beam Brackets

The objective of this recommendation is to supplement the beam connection detail with a new steel bracket that engages with the masonry wall. Access to the South Tower is only via a small circular stair. Any material or equipment will need to be carried up a few flights of stairs to the space.

- One bracket is added to each side of the beam and at each end of the beam, for a total of 4 fabricated brackets.
- Drilling of existing beam for bolted connection is included.
- Some incidental demolition of masonry to install the bracket is included.
- Repair of masonry and grout for the new bracket is included.