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Ties That Bind: The Emergence of Iron Tie Rod Reinforcement in Load Bearing Masonry Buildings of Charleston, S.C.

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TIES THAT BIND:
THE EMERGENCE OF IRON TIE ROD REINFORCEMENT
IN LOAD BEARING MASONRY BUILDINGS OF CHARLESTON, SC

A Thesis
Presented to
the Graduate Schools of
Clemson University and the College of Charleston

In Partial Fulfillment
of the Requirements for the Degree
Masters of Science
Historic Preservation

by
Jamie Lynn Wiedman
May 2012

Accepted by:
Ashley Wilson, Committee Chair
Kristopher King
Ralph Muldrow

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ABSTRACT

Comprehensive studies of structural tie rods in Charleston's load bearing masonry buildings are scarce. While the presence of these anchors on building facades fascinates passers-by and is appreciated throughout the city, a general knowledge of their history and emergence as a technology is lacking. Often associated with the earthquake of 1886, iron tie rods were in fact present in Charleston buildings as early as the 18th century. And while the effect of the 1886 earthquake on Charleston's built environment is typically looked at from a sociological or political perspective, and the physical effect has been studied extensively, rarely has the history of Charleston's buildings been looked at in relation to their earthquake safety and resistance. In the direct aftermath and subsequent years following the earthquake of 1886, extensive studies were conducted to determine the effect on the Charleston's buildings. The use of iron tie rods abounded as an earthquake damage repair technique, yet documentation of their use is limited.

The following thesis presents an overview of iron tie rod form and function in Europe and America. Further narrowing in scope, it compiles information on the subject matter focusing specifically on the use of tie rods and 'earthquake bolts' in Charleston. While this has not been an exhaustive study, its findings establish a solid foundation for future research of the Charleston building tradition and the universal preservation of wrought iron tie rod reinforcement in North America.

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CHAPTER ONE

INTRODUCTION

Visitors to Charleston, South Carolina are often fascinated with the small iron plates that seemingly dot every façade on the historic peninsula. Often called “earthquake bolts”, these caps anchor long iron tie rods that are best known for repairing and reinforcing Charleston’s masonry buildings following the great earthquake of 1886. Tour guides point them out, and bed & breakfasts and other establishments around town actively promote the fact that they have them. Yet, the tie rod is a structural reinforcement technique that has been incorporated into buildings in Charleston and elsewhere long before the earthquake, and well before the emergence of seismic requirements in building codes. Many of these buildings still survive today.

Wrought-iron structural tie rods are a historic building technology that has served both utilitarian and stylistic purposes worldwide since ancient Greek and Roman times. They acted as structural reinforcement within arches, vaults, domes and load bearing walls, a fast and economic repair method, and a decorative accent element on the exterior of buildings. First developed in Ancient Europe, the simplistic concept and manufacture of tie rod reinforcement endured and spread to the American colonies with Dutch and British settlers as a familiar construction technique from home. While the fascination for these builders’ work is ever present, research has been limited and few details are known about the technology’s materialization in North America.

The first section of this thesis discusses the development, manufacture, and exportation of iron tie rod reinforcement from Europe to America. It briefly traces the history of iron tie rods through ancient and medieval Europe, and then segues into a description of tie rod use by American colonists in the 18th and 19th centuries. It investigates the historical production of iron in this country, its importation and ease of availability such that these criteria may be applied to dating the ironwork forms and fittings. Evidence of iron tie rods and wall anchors has been uncovered at several 18th century properties in North America, most located in northeastern cities primarily settled by the Dutch. However, very little study has been

conducted on the use of tie rods in the southeastern American colonies, particularly in Charleston where a significant amount of 18th century buildings still survive.

Subsequent sections, and the bulk of this thesis' subject matter, address the proliferation of the iron tie rod in 19th century Charleston following the 1886 earthquake and the misnomer of the "earthquake bolt" throughout the city. The southern city of Charleston, South Carolina provides an excellent case study, exhibiting surviving evidence of 18th, 19th, and 20th century tie rods. Like buildings themselves, iron tie rods are subject to changes in form, function, design, and material. Ideally, one should be able to date structural repairs by closely examining the iron reinforcement. The quality of its material, its design, and its method of attachment are all chronological indicators to potentially help date significant phases of alteration or building campaigns. By including specific case studies, this thesis provides a narrative of the overall history and use of iron tie rod reinforcement in Charleston by focusing on remaining physical and written evidence.

How did Charlestonians respond to the great earthquake of 1886? The answer to this question is typically looked at from a sociological or political perspective, and the physical effect on Charleston's built environment has been studied extensively. However, rarely has the history of Charleston's buildings been looked at in relation to their earthquake safety and resistance. This preliminary survey of earthquake tie rod reinforcement conducted by the author is the first of its kind in Charleston. It documents pre-1886 load bearing masonry structures and the damages assessed following the great earthquake in 1886, compares those assessed damages and reinforcement recommendations to documentation of work actually completed, and compiles historical information based on visual examination. It addresses the questions of when tie rod reinforcement was recommended, how often these recommendations were taken and carried out in repairs, and describes the variety of anchor plate designs found on buildings throughout the survey area.

The final section of this thesis utilizes published conservation sources and a case study to determine the most effective and economical way to conserve these structural objects. The history and evolution of the iron tie rod technology demonstrates that they were manufactured and installed out of structural needs bound by time and place, warranting the historic preservation and conservation of these iron artifacts.

This thesis attempts to clarify the historic availability, characteristics and use of the tie rod technology, and align these historical features with those that are used today. Although it does not expressly address reinforcing historic structures in retrofit today, there are implications in the history of tie rod technology for retrofit planning. It is the intention of this thesis to provide an original study of the structural iron tie rod technology and categorize the forms to make a connection between style, function and period in Charleston. It is hoped that this will provide a better understanding of the iron tie rod in Charleston building traditions.

CHAPTER TWO

THE ORIGINS OF IRON TIE RODS IN LOAD BEARING MASONRY BUILDINGS

Inherent Weaknesses in Load Bearing Masonry Buildings

When load bearing masonry cracks, in terms of engineering analysis, it is usually described as having “failed”, even if collapse does not occur. Cracking causes the internal strength of the wall to diminish, as the walls undergo deformations through movement along the mortar joints (in-plane) or in bending (out-of-plane). A load bearing masonry building (abbreviated here as LBM) has primary structural walls constructed of brick, concrete block, adobe, or some other type of masonry material, and is not braced by reinforcing rods or bars. The exterior bearing walls conduct structural loads to a foundation system. Generally constructed prior to 1933, LBM buildings predate modern reinforcement standards and earthquake-resistant design.

The fundamental question as to why iron reinforcement rods are important enough to warrant study can be answered reasonably simply. Iron reinforcement and repairs represent some of the earliest known low-tech, minimum intervention repairs to failed masonry. These interventions acknowledge the inherent shortcomings of the building material, its fabric, construction and properties, and the efforts that the craftsmen went to in overcoming limitations. The initial section of this chapter addresses some of the causes of these failures and the performance of unreinforced, load bearing masonry wall systems in a seismic event.

Settlement

Apart from questions of appearance, masonry is seldom remarked on unless it fails, usually with the surprise appearance of cracks, which quite often signal a problem with the foundation itself or settlement of the entire structure. Settlement usually refers to movement of the finished building structure. Differential settlement is movement that causes the structure to change shape, and therefore introduces new strains within the building. Figure 2.1 illustrates the three customary forms of differential settlement, though a building will usually be subjected to a combination of all of these. Under uniform bending by sagging (or reverse bending), the façade of a load bearing masonry structure goes into bending like a simple beam. If excessive, hair line cracking will typically emerge above the damp-proof course. Tilting occurs when the

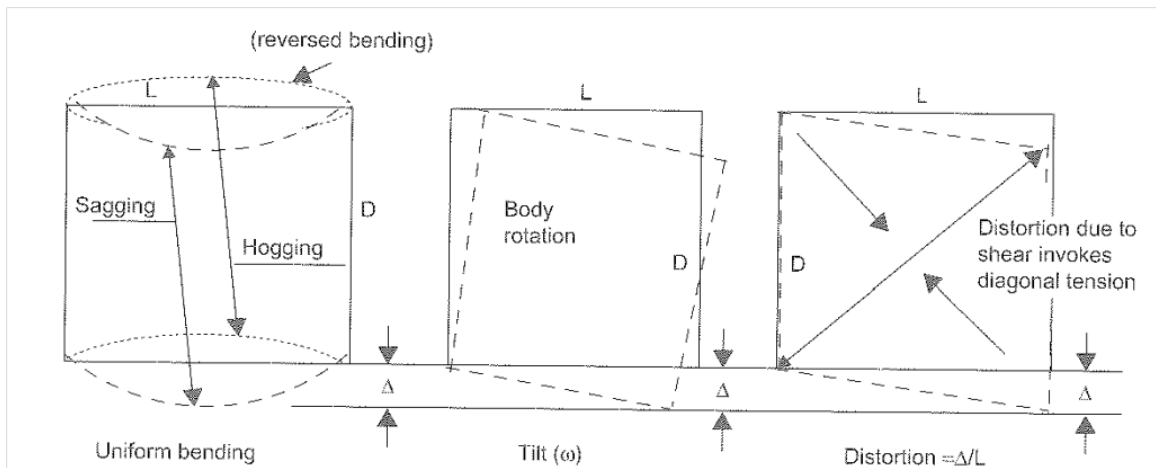


Figure 2.1: Customary definitions of building settlement (Source: Forsyth, Michael, ed. *Structures and Construction in Historic Building Conservation*. Oxford: Blackwell Publishing, 2007).

entire structure or façade rotates within the same plane. Although ground settlement is typically present, it might not always introduce torsion stresses if it settles evenly without deformation of the building. Therefore, a wall panel can tilt without showing evidence of strain or cracking. Deformation due to shear is effectively illustrated as a distorted parallelogram shape. Shear forces are the source for diagonal tension; therefore, diagonal cracking develops within the masonry walls at right angles to the direction of the tension stress¹.

Inadequate Design/Restraint

Perhaps the most common reason for the failure of load bearing masonry walls is the lack of restraint as a result of the inadequate structural design of LBM buildings in the 18th and 19th centuries. Lateral restraint is fundamental to prevent buckling - the tendency of the wall system to suddenly bow outwards - but also to prevent the wall from completely falling away from the frame. Lateral restraint is often provided by floor and roof framing members spanning onto the wall; their weight, and the floor or roof acting as a stiff horizontal 'plate', are adequate to hold the wall in place at floor level. However, any load bearing masonry structure with a timber frame has two opposite walls supporting floor joists spanning between them, but the other two walls run parallel with the joist system, providing no restraint for the walls (see figure 2.2). If such an external wall has no ties into the adjacent construction, the building will

¹ Michael Forsyth, ed. *Structures and Construction in Historic Building Conservation* (Oxford: Blackwell Publishing, 2007), 101-103.

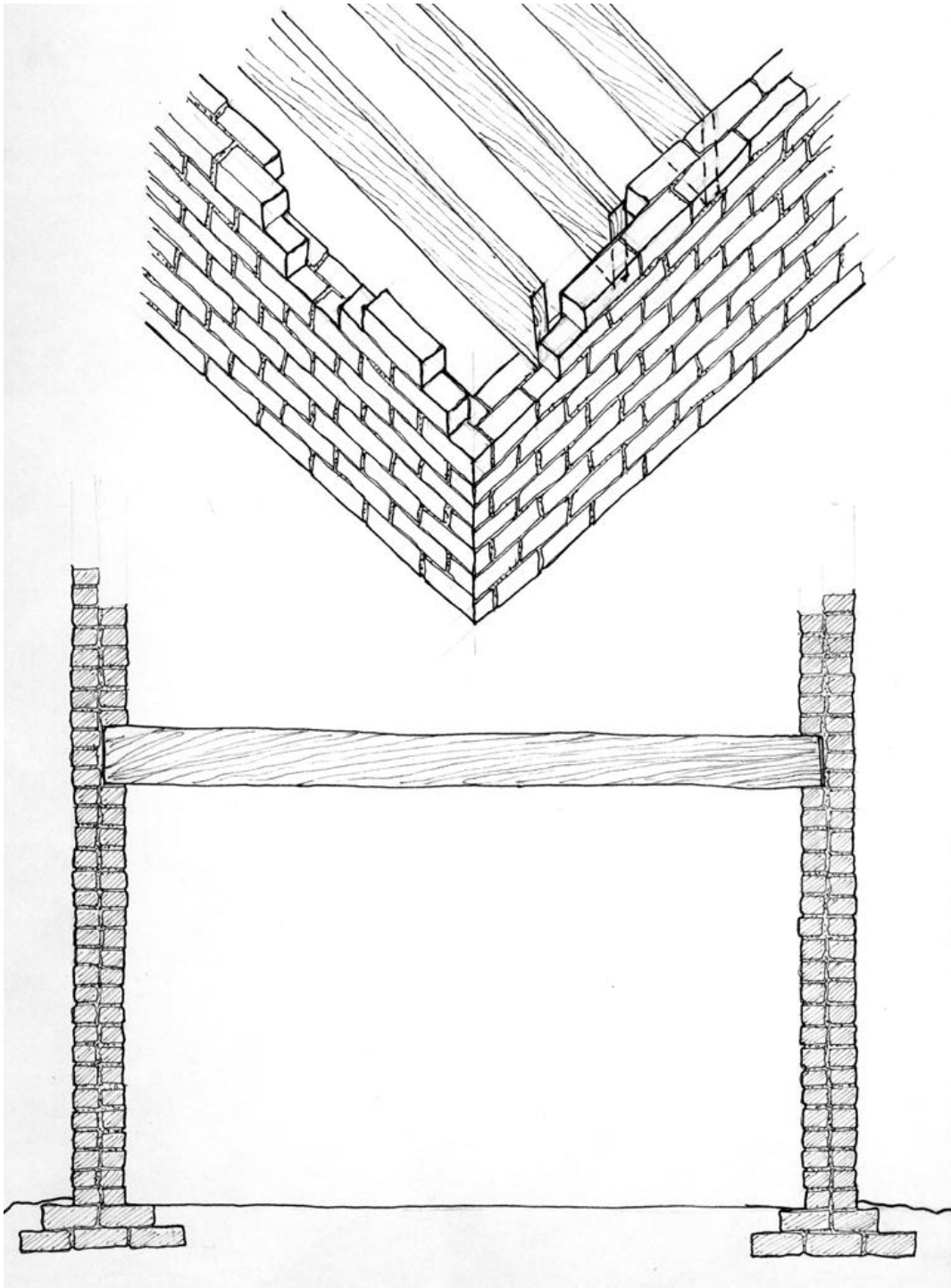


Figure 2.2: Sketch: inadequate wall restraint in load bearing masonry construction (Sketch by author).

effectively rise up two or more stories with no lateral restraint². Thus, outward movement of these walls is not uncommon.

Alterations or Misuse

It is rare that a historic building is not altered during its lifetime. Unfortunately, it is not uncommon that the alterations are made with disregard (or lack of knowledge) to the consequences. Additionally, over time the building's many occupants subject the structure to other uses – or misuses – that test its structural capacity. There are two common alterations that directly affect the performance and survival of load bearing masonry walls. Historically, the addition of one or more floors was a frequent alteration when an increase in living space was demanded. Today's knowledge of structural performance clearly indicates that this increases the load on the structure below, including the foundations, and this is likely to produce noticeable structural movement within the masonry wall system. Another alteration disturbing the stability of foundations below an existing LBM wall is the excavation of ground nearby. For example, one should investigate the risk of movement arising from ill-considered excavation of trenches to assist with ground drainage near damp masonry walls³. Even more common is the excavation of deep foundations on a neighboring lot, which will inevitably affect the settlement of adjacent structures.

Material Deterioration

Traditionally, historic brick and stone buildings were constructed using a softer lime mortar mix than is the custom today. This early construction practice initially carried with it an ability to sustain higher structural deformations and ground settlement, with the soft mortar acting as a "pillow" for each masonry unit. Indeed, this feature worked to the advantage of the building's stability. However, as a sacrificial material, the soft mortar can also be a disadvantage under heightened stress and in cases of extreme ground movement. The historic lime mortar consists of nothing but fine sand or clay mixed with lime and water. Thus, as an inherent fault of the material, a sandy mixture easily dries out and commences to crumble. A clay mixture is quick to absorb moisture from the air, but expands in consequence. When the dampness dries

² Forsyth, *Structures and Construction*, 129.

³ Forsyth, *Structures and Construction*, 135-136.

out, the mortar between the bricks contracts again and the walls soon begin to crack and weaken.

Sulphate attack on brick mortar is another component of material decay in masonry walls. The deterioration is caused by sulphates in the bricks, which are leached into the mortar by rainwater. If movement of the building materials is unrestrained, then the brickwork expands resulting in spalling and loss of material. If movement cannot take place freely, then the mortar may fracture internally, still weakening the joints between masonry units⁴.

Early Iron Reinforcement

Historic Availability, Characteristics and Use in Europe

Various forms of iron reinforcement in masonry structures have been commonly employed in Europe since ancient Greek and Roman times to overcome the inherent faults of the load bearing masonry construction. One might be tempted to regard ancient structural iron as consisting solely of iron clamps and dowels to fasten together stones and iron braces used to hold the positions of stones. Yet, there are instances which exhibit advancing technical knowledge. The greatest advantage of an iron bar is its ability to resist tension and the forces tending to thrust a structure apart. With a tensile strength as much as 70 times that of concrete⁵, the material most commonly used was wrought iron. An example of the earliest lateral reinforcement techniques occurred at the Theban Treasury at Delphi. The foundations of the Treasury were constructed of a soft limestone and laid upon a steep slope; thus, the foundations were in need of reinforcement with the initial placement of the stone. This was provided by great iron bars cut into the stone courses, overlapping and hooked at the corners so as to provide a strong rectangular frame in between each limestone course (see figure 2.3). The bars themselves were wider than they were deep as this was thought to aid in the prevention of lateral displacement⁶.

⁴ Forsyth, *Structures and Construction*, 133. Chapter 5 goes into further detail regarding the effect of sulphates on the masonry material.

⁵ Janet Delaine, "Structural Experimentation: The Lintel Arch, Corbel and Tie in Western Roman Architecture", *World Archaeology*, Vol. 21 No. 3, (1990), 417.

⁶ William Bell Dinsmoor, "Structural Iron in Greek Architecture", *American Journal of Archaeology*, Vol. 26, No. 2, (April-June 1922), 148-149.

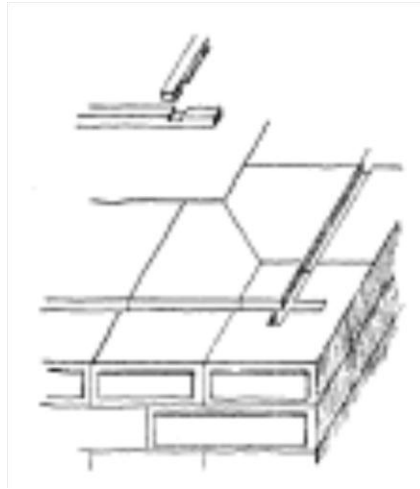


Figure 2.3: Foundation of Theban Treasury at Delphi. (Source: Dinsmoor, *“Structural Iron in Greek Architecture”*, 149)

Although iron ties were used to reinforce foundations and the span of architraves above columns in Greek architecture, the Romans set the initial precedent for the use of iron tie bars across barrel-vaulted spaces⁷. Perhaps the earliest example is in the upper floor arcades of the Augustan Horrea Agrippiana in Rome. Evidenced by cuttings in the upper surface of several Travertine blocks at the top of the arcade piers, iron tie bars (roughly 6-10cm in cross section) spanned the passage at the level of the springing of the vaults (see figure 2.4a). In the Basilica Ulpia, iron bars were attached to the entablature blocks over both sets of columns and passed through the crown of the barrel vault (see figure 2.4b). Slight evidence has also been found for exposed tie bars at the level of the architrave cornice in the upper level. Similar to Basilica Ulpia, a concealed system was utilized in the colonnades of the Baths of Caracalla. However, here, one of the ties was anchored to the outer wall by means of stone blocks embedded in the concrete (see figure 2.4c) – an indication that the development of iron reinforcement was progressive, one innovation inspiring another.

⁷ Delaine, *“Structural Experimentation”*, 418.

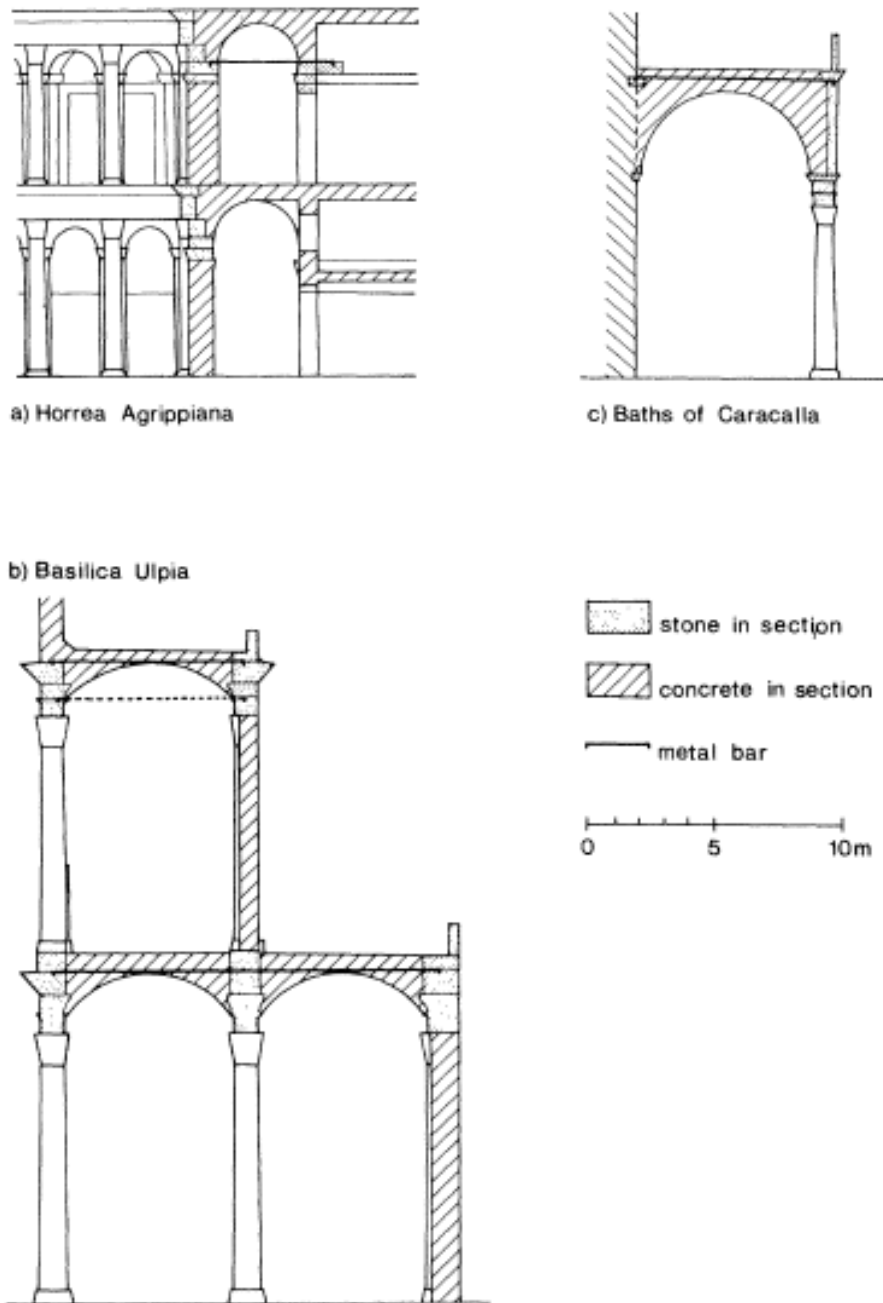


Figure 2.4: Early Greek & Roman methods of iron reinforcement; iron reinforcement was used in structures such as the Basilica Ulpia and Baths of Caracalla. (Source: Delaine, "Structural Experimentation",

As a fundamental means to ensure the stability of colonnades and vaulted spaces against lateral thrust, the same structural concept of iron tie reinforcement was further developed in Byzantine and medieval construction. Exposed iron ties were used in early Christian and Byzantine religious architecture at least from the 6th century, and entered Western European architecture circa 1000 A.D. Tie rods are ubiquitous in early Renaissance architecture, from Brunelleschi to Michelozzo. Iron reinforcement was routinely used to strap domes in order to eliminate heavy buttresses. Three rods reinforce the dome of St. Peter's in Rome, and Claude Perrault used iron tie rods to counteract the outward thrusts of interior vaults inside the Louvre in Paris. Medieval builders also discovered from past experience that iron reinforcement embedded in stone masonry was likely to rust and thereby swell, and in so doing potentially crack the masonry. If iron was in direct contact with stone, there was a risk that highly uneven bearing could lead directly to local splitting or crushing. In order to prevent these mishaps, medieval masons sometimes coated the grooves in which iron rods were placed with molten lead or mortar⁸.

Further evolution of the reinforcement concept is evident in medieval English-Dutch vernacular buildings. Perhaps coinciding with the development of timber frame construction, the Dutch set the earliest precedent for the short wall anchor, or also called a stay. The Dutch stays are wrought iron fittings in the exterior wall of the building and part of a distinctive framing technique which originated in the Dutch low country, eventually spreading with Dutch colonies around the world. These wall anchors have two parts: one, on the exterior face of the wall, is an iron bar or motif which is slotted through a loop in the second part, another iron bar called the 'tongue'. The tongue is encased in the wall and fixed to a timber in the wooden frame of the structure (see figure 2.5)⁹. These anchors were commonly inserted during initial construction, rather than serving as a post-construction repair technique. The earliest attachments for Dutch stays were all plain bars of wrought iron. Having been introduced to strengthen the overall structure of the house, the attachments were unconsciously ornamental. However, around 1550, the potential of wrought iron to form diverse shapes was exploited. The

⁸ J. Fitchen, "Arch and Vault Ties in Medieval Construction," in *The Construction of Gothic Cathedrals* (Oxford: 1961), 276.

⁹ Patricia Reynolds, *Transmission and Recall: The Use of Short Wall Anchors in the Wide World*, (PhD diss., University of York, 2008), 24-25.

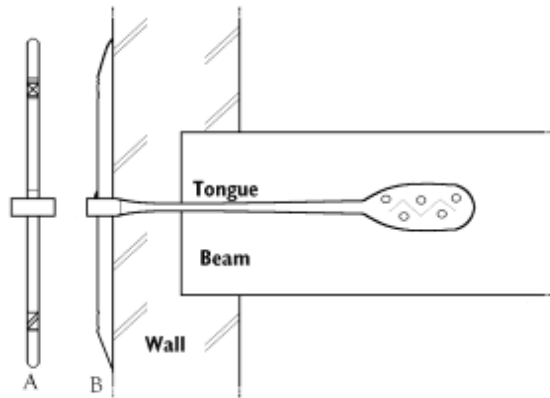


Figure 2.5: Elevation (A) and section (B) of a Dutch short wall anchor. (Source: Reynolds, *Transmission and Recall*, PhD diss.)



Figure 2.6: Dutch short wall anchors with decorative exterior attachments revealing the date of construction in Great Yarmouth, Norfolk. (Source: Reynolds, *Transmission and Recall*, PhD diss.)

exterior attachments became decorative and could be formed into the initials of the builder/owners, merchants' marks, even the date of construction (see figure 2.6).

The British-Colonial Iron Trade in the Eighteenth Century

The dominance of the British iron industry was solidified in the eighteenth century by revolutionary innovations such as the 1775 patent of James Watt's steam engine, Henry Cort's patent of his puddling and rolling technique in 1784, and the perfecting of coal-fired refining methods in the 1780s¹⁰. As a proprietary colony of Great Britain, North American colonial craftsmen began manufacturing ironwork soon after settlement, but most specialty items and decorative iron were purchased abroad. It can only be assumed that structural tie rods would have been included in this imported ironwork as a specialty item. American craftsmen could not compete with the efficient, specialized manufacturing methods for English iron making centers¹¹.

Iron trade partnerships are understood as loose, commercial alliances, the merchants of which exchanged intermediate goods and semi-finished products like scrap pig iron, bar iron, rods, nails, etc. Interestingly, until the change in coal technology at the end of the 18th century, the majority of British bar iron was itself imported, primarily from Sweden and Russia. As early as the 17th century, Sweden became Europe's leading iron exporter, and Britain was their leading market. Russian iron first appeared in significant quantities at British ports in the 1730s, and by the 1760s Russian shipments had overtaken those from Sweden in the British ports. Out of the thousands of tons of bar iron that came through the ports, most of it was distributed to London. However, a great deal was subsequently reshipped to faraway colonial outposts like the West Indies and the North American colonies¹². Being the proprietors of anchor shops in the colonies, the British were in essence dealers in Swedish and Russian iron.

One of the leading commercial iron merchants in 18th century Britain was Graffin Prankard. Operating out of Bristol, it is known from his account books that he imported bar iron from Sweden on a large scale as early as the 1720s. By the late 1720s, Prankard was also dealing

¹⁰ Chris Evans, Owen Jackson and Goran Ryden, "Baltic Iron and the British Iron Industry in the Eighteenth Century", *Economic History Review*, Vol. 60 No. 4 (2002), 642.

¹¹ Orlando Ridout and Willie Graham, *An Architectural and Historical Analysis of the Nathaniel Russell House, Charleston South Carolina*, Vol. III, A Historic Structures Report prepared for the Historic Charleston Foundation (Charleston, December 1996), 87.

¹² Evans, et. al, "Baltic Iron and the British Iron Industry", 643-646.

in Russian iron. By 1728, he had established a pattern of trade with the newly formed North American colonies and was fully engaged in the Atlantic trade, shipping iron products and other manufactured goods to America. In fact, one of his ships, the *Parham Pink*, was dispatched in 1728 to Charleston, carrying nails, pots, steel, bar iron and gunpowder¹³. The account books of Prankard and other British iron merchants alike indicate the high likelihood that the iron material used in early 18th century structural applications, like tie bar reinforcement, is in fact of Swedish or Russian origin by way of Great Britain. In addition to importing the material, the construction methods of early American iron reinforcement are undoubtedly imported as well.

Historic Availability, Characteristics and Use in America

Documentary evidence for the early use of iron tie bars in America is very scarce. However, one of the earliest recorded observations of wall anchors in America is by Dr. Benjamin Bullivant, who observed in 1697 that “most brick houses have the date of the years on them, contrived as iron cramps to hold in the timber to the walls”¹⁴. He was undoubtedly describing the presence of short wall anchors mimicking those of the Dutch. The Flemings who flocked to England in the mid-16th century left many examples of these anchors in numeral and other forms on the houses where they settled. It is not surprising, therefore, to find these wrought iron anchors on the early brick and stone houses of the Dutch settlers in the American colonies. This medieval building technique dates back to at least the 10th century in the Netherlands, but continued as late as the 1760s in the Hudson Valley region of North America¹⁵. In 1744, Alexander Hamilton visited New York and observed many of these features:

“... The houses are more compact and regular and, in general, higher built, most of them after the Dutch model with their gravell (sic) ends fronting the street. There are a few built of stone, more of wood, but the greatest number of brick, and a great many covered with pan tile and glazed tile with the year of God when built figured out with plates of iron upon the fronts of several of them.”¹⁶

¹³ Evans, et al. “Baltic Iron and the British Iron Industry”, 647.

¹⁴ Quoted in Reynolds, *Transmission and Recall*, 31.

¹⁵ Roderic H. Blackburn and Ruth Piwonka, *Remembrance of Patria: Dutch Arts and Culture in Colonial America, 1609-1776*, (New York: SUNY Press, 1988), 115.

¹⁶ “*Muurankers - Wall Anchors*”, Lowlands-L Gallery, Accessed 1 March 2012, http://lowlands-l.net/history/reynolds_muurankers.php.

The Hudson Valley region of present-day New York abounded with these Dutch settler houses, most of which exhibited such wall anchors, generally of the fleur-de-lis design like the ones at the Pieter Winne house (1720) and the Van Hoesen house (1740) in Claverack Township, NY (see figure 2.7). In other cases, these anchors were numerals announcing the date when the house was built, or letters indicating the owner's initials like those on the old Salisbury House in Leeds, NY (see figure 2.8)¹⁷.



Figure 2.7: Dutch inspired wall anchors: Left: Van Hoesen House (1740) & Right: Pieter Winne House (1720). (Source: *Van Hoesen House Historical Commission*, <http://www.vanhoesenhouse.org>)

¹⁷ Albert Sonn, *Early American Wrought Iron*, Vol. III, (New York: Charles Scribner's Sons, 1928), 11-12.

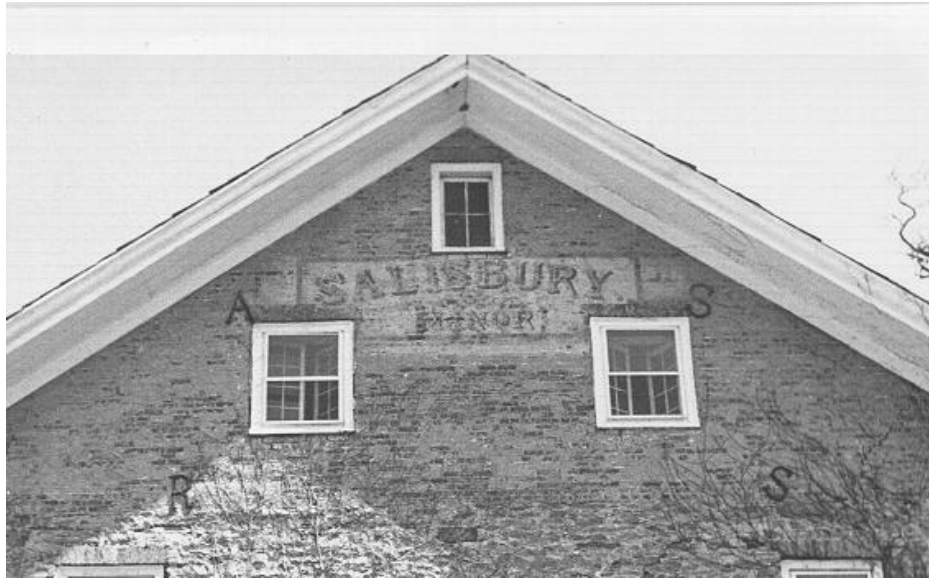


Figure 2.8: Wall anchors in the form of owner's initials on the old Salisbury House in Leeds, NY (Source: *National Register of Historic Places: Salisbury Manor*, <http://www.oprhp.state.ny.us>).

In the southern colonies, the 'S' type is the most common type of early wall anchor plates found in various forms, a form which also has English prototypes. This type of stay is of almost infinite variety in Charleston and was a favorite with the smiths in nearly all of the colonies from the Carolinas to Canada¹⁸. In Berkeley County, SC just outside of Charleston, Mulberry Plantation has possibly some of the earliest forms of iron ties in the lowcountry. The house was built circa 1714 in the Jacobean baroque style with 'S' type anchors in the upper brick gable (see figure 2.9)¹⁹. Although the precise date of installation of the wall anchors at Mulberry is not known, or whether or not they were installed during initial construction, the form and style of the 'S' shaped plates could certainly be of an early 18th century origin. Figure 2.10 shows a 'butterfly' shape wall anchor on an 18th century structure at 91 Exchange Street in Charleston. This pattern and other similar forms can be found on other houses in New England and Canada that date to roughly 1745-1750²⁰.

¹⁸ Sonn, *Early American Wrought Iron*, 106.

¹⁹ "Mulberry Plantation, Berkeley County SC", SC Department of Archives and History, National Register Properties in South Carolina, Accessed 8 March 2012, <http://www.nationalregister.sc.gov>.

²⁰ Sonn, *Early American Wrought Iron*, 108.



Figure 2.9: 'S' type wall anchor plates at Mulberry Plantation (1714), Berkeley County, SC (Source: "Mulberry Plantation, Berkeley County SC", National Register Properties in South Carolina, <http://www.nationalregister.sc.gov>).



Figure 2.10: An early 'butterfly'-shaped wall anchor on an 18th century structure at 91 Exchange Street, Charleston, SC (Photos taken by author)

Following the American Revolution and within the first half of the 19th century, Americans matured industrially by developing factories capable of sustained mass production, which in turn put the country on the path to manufacturing self-sufficiency. By the turn of the 19th century, iron manufacture was in actuality an old industry in America, and the United States already contained many furnaces to smelt iron from ore, as well as forges, foundries and smith shops to convert pig iron to useful shapes. Yet, the refining and processing stages of iron manufacture remained disjointed, stunting the development of mass production methods for iron products. However, in the years following 1800, large firms solved this problem, clearing the way for integration of all stages of manufacture²¹. The spread of manufacturing together with the advent of the railroads, one of the greatest causes of social and economic change in the 19th century, allowed for the standardized fabrication of iron products including reinforcing tie bars and rods.

Wrought iron continued as the structural material for carrying tensile loads. With recent advances in iron production methods, tie rods could be rolled directly from flat bar. The rolling process produced a rod with excellent longitudinal strength, which could easily be worked to form a flattened end that could then be drilled to receive a pin. Such rods were frequently used as components in 19th century 'composite' structures, such as trusses, trussed beams, and timber partitions as well as in direct applications like wall tie rods²². With the exception of a few rare 18th century examples, the application of full-length, wall-to-wall iron tie rods first appeared in America on a regular basis in the 1800s. In New York and Chicago, rods were patented and advertised for frequent use in controlling uneven settling (see figure 2.11). Some even make the claim that these rods became a standard practice in all construction in the 19th century²³. In San Francisco, architects and engineers appear to have maximized cross-structure tie-rods for earthquake-resistant purposes, specifying tie rods in both new designs and retrofits. In 1853, Gordon Cummings, an English architect, was commissioned to build a massive brick structure at Montgomery and Washington streets, known as the Montgomery Block (see figure 2.12). The symmetrical building's design featured innovative structural features. The building contract

²¹ Harold Livesay, "Marketing Patterns in the Antebellum American iron Industry", *The Business History Review*, Vol. 45 No. 3 (1971), 269-275.

²² Forsyth, *Structures and Construction*, 183.

²³ Stephen Tobriner, *Bracing for Disaster: Earthquake-Resistant Architecture and Engineering in San Francisco, 1838-1933* (Berkeley: Heyday Books, 2006), 46.

stated that “transverse and longitudinal rods of iron will run through each wall and also through partitions ... and every 8th joist will have an iron anchor.” The bills for iron confirm the building was tied by a quantity of iron rods, bars, and anchors per the original design²⁴. The use of these reinforcing devices illustrates a growing understanding of the threat of lateral bending and possible collapse of masonry walls.

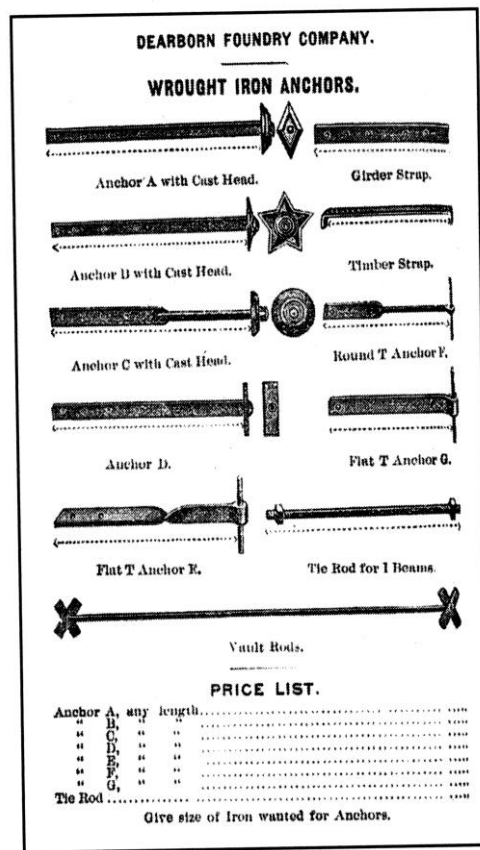


Figure 2.11: Advertisement for iron anchors and rods from Dearborn Foundry Company brochure, Chicago 1887. (Source: Tobriner, *Bracing for Disaster*, 46.)

²⁴ Tobriner, *Bracing for Disaster*, 17-18.



Figure 2.12: The Montgomery Block in San Francisco, CA, erected with iron reinforcement probably designed to resist earthquakes and settlement. (Source: *Tobriner, Bracing for Disaster*, 17)

Despite their design and specification for new construction in 19th century San Francisco, tie rods in Charleston were used to retrofit and reinforce masonry buildings affected by disaster. They were used to repair buildings after a tornado swept across the peninsula in 1811, when an 1885 cyclone bombarded downtown with powerful gales, and later recommended by the United States government after the earthquake of 1886, years after the earthquake of 1865 in San Francisco. Natural disasters are a fact of life in the lowcountry, and tie rods quickly proliferated as a remediation technique and affordable stabilization method.

Common Forms of Iron Ties & Plates²⁵

Early structural iron ties were assembled of long pieces of wrought iron several inches in diameter that were inserted through the walls of buildings as reinforcement. A distinction must be drawn between reinforcement rods, which are of round section, and bars, which are of square or other polygonal section. The earliest hand-wrought iron tie bars were commonly square in cross-section until the 1783 patent by Henry Cort for the use of grooved rollers, which marked the advent of rolled stock iron bars²⁶. The following illustrations of common variations of iron reinforcement ties are not an exhaustive compilation. The purpose of this section is simply to assemble and demonstrate the most common variations found in American historic structures from the 18th and 19th centuries, especially those observed in Charleston, South Carolina.

In some instances, tie rods and bars are used to connect two opposite masonry walls, spanning the full-length of the building, and to arrest their relative movement. Historically, the installation of these full-length tie rods or bars typically involved inserting two separate ties through the opposite walls and then joining them (usually at the center point), where adjustments can be made to “tighten” the building. At their center connection point, rods and bars were either screwed into turnbuckles or toggles (see figure 2.13), or hooked together end-to-end (see figure 2.14).

²⁵ No catalog of common iron tie rod or bar types could be located. References for the types illustrated in this thesis include: Albert Sonn, *Early American Wrought Iron, Volume III* (New York: Charles Scribner’s Sons, 1928), 108-109; Barry Hillman-Crouch, “Historic Ironwork Repairs to Timber-Framed Buildings” (Master’s Thesis: Oxford University, 2003); Walter Edgar, “Earthquake Rods”, *The South Carolina Encyclopedia* (Columbia: University of South Carolina, 2006), 280.

²⁶ Reginald A. Mott (ed. P. Singer), *Henry Cort: The Great Finer* (London: Metals Society, 1983), 31-36.

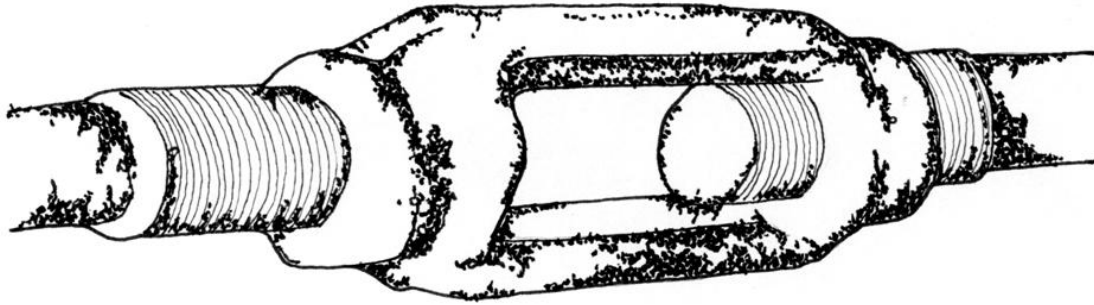


Figure 2.13: Sketch: Turnbuckle center attachment detail (*Sketch by author, not to scale*).

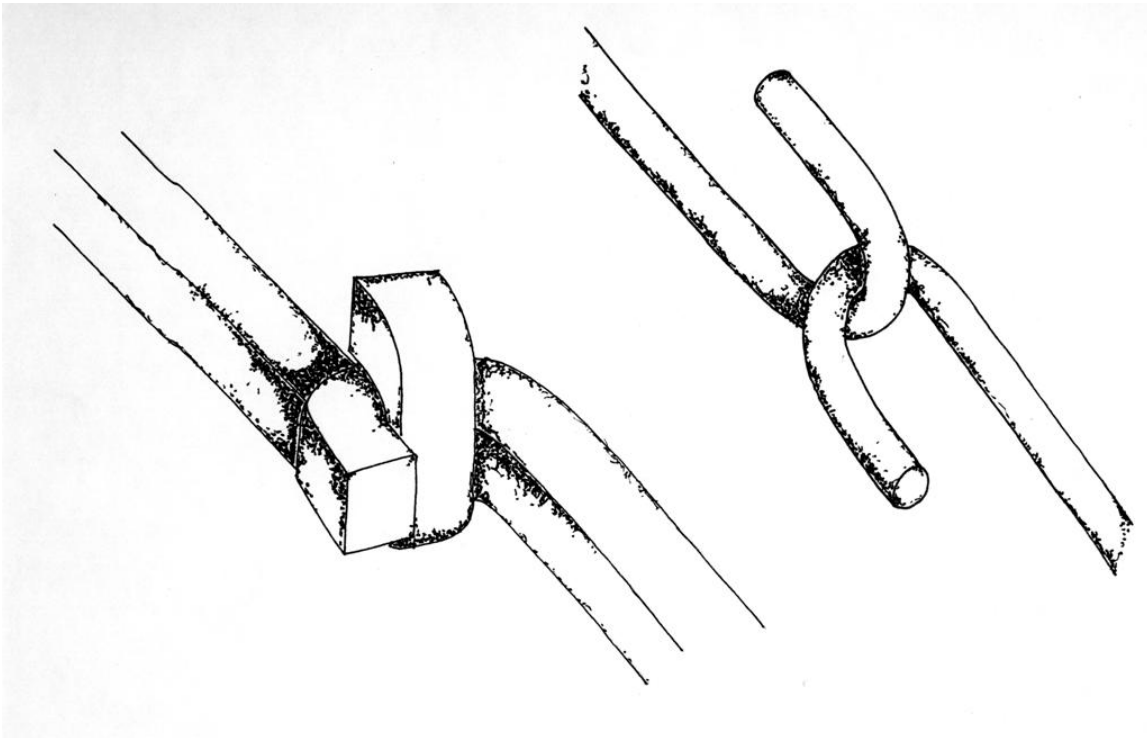


Figure 2.14: Sketch: Hooked center attachment detail (*Sketch by author, not to scale*).

On occasion, tie rods and bars do not span the full distance of the building, connecting opposite walls. In fact, tie rods were commonly threaded through an exterior masonry wall and then bolted or hooked onto an interior framing member, sometimes just eight to ten feet into the structure. This type of reinforcement is also sometimes referred to as a 'stay' (see again figure 2.5).

Tie rods and bars are anchored at their ends to the exterior structure in a number of ways. Most commonly, attachment methods include nailed or stapled terminals if concealed within the structural frame (see figure 2.15), or their threaded ends are passed through an exterior plate and secured with nuts or bolts on the exterior of the masonry wall. These exterior plates are commonly called a number of terms including: gib plate, anchor plate, bearing plate, and/or patress plate.

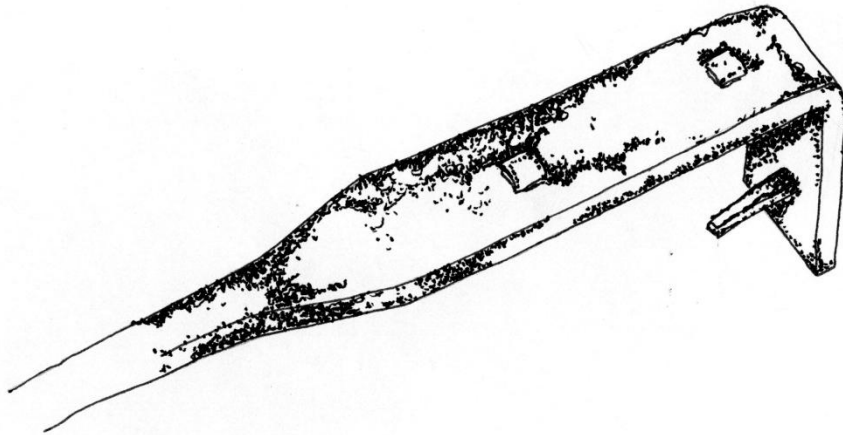


Figure 2.15: Sketch: Bent, nailed end connection (*Sketch by author, not to scale*).

The exterior anchor plates used to secure the outside ends occur in countless shapes and sizes. The illustrations in Figures 2.16 and 2.17 were done in 1928 by Albert Sonn for his book *Early American Wrought Iron*²⁷, and demonstrate some of the earliest, European-inspired types of plates found in America. Circular bearing plates grew in popularity in the 19th century, and truly proliferated in the Charleston area after the earthquake of 1886. These round plates are also sometimes called bosses (see figure 2.18). Additional photographic documentation of common anchor plate types observed throughout Charleston can be found in the *Survey Findings* section of Chapter Three.

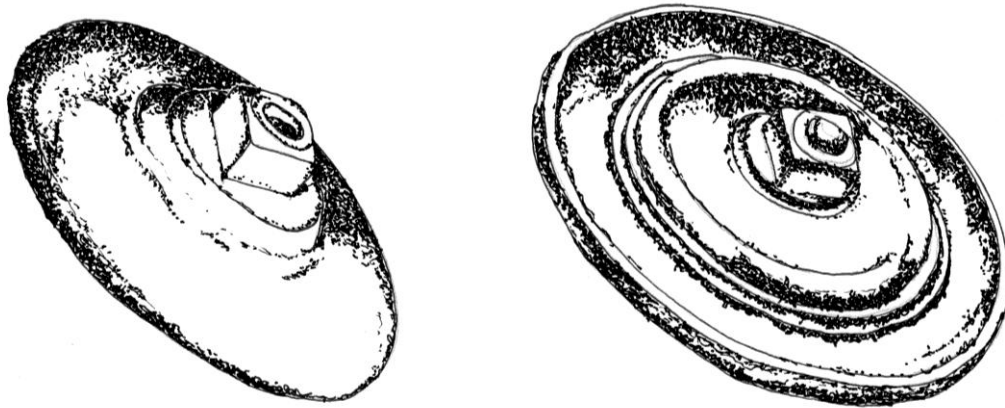


Figure 2.18: Circular anchor plates; (L) flat, plain profile; (R) decorative, moulded profile. (Sketch by author, not to scale).

²⁷ Sonn, *Early American Wrought Iron*, 108-109.



Figure 2.16: Plate 256 in Sonn's *Early American Wrought Iron* (Source: Sonn, *Early American Wrought Iron*, 111)

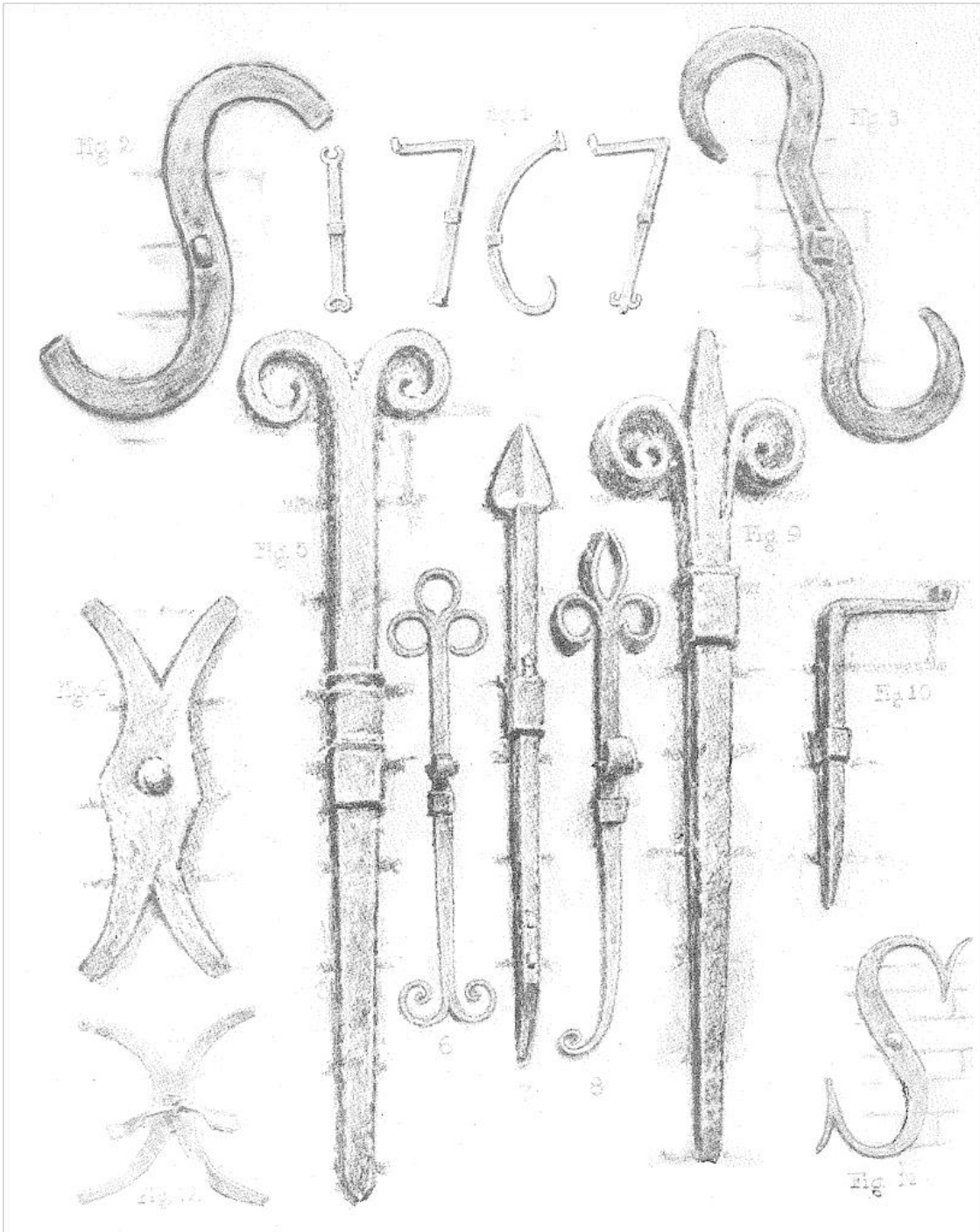


Figure 2.17: Plate 255 in Sonn's *Early American Wrought Iron* (Source: Sonn, *Early American Wrought Iron*, 109)

CHAPTER THREE

CHARLESTON “EARTHQUAKE BOLTS”

Load Bearing Masonry Buildings in Pre-1886 Charleston

The unique dwelling types and construction characteristics in Charleston, South Carolina are largely the result of the city’s history, location, and its climate. Yet, the construction methods used in the lowcountry region are comparable to general construction practices throughout the rest of the country. A 1986 survey indicates percentages by dwelling type in Charleston County. The percentage results for the peninsula of downtown Charleston were: 55% wood frame, 16% wood frame with brick veneer, and 29% unreinforced masonry (masonry bearing wall construction)²⁸. These percentages reflect the character of Charleston’s historic district, particularly the continued presence of load bearing brick masonry structures.

Brick and stone are undeniably some of the oldest building materials in history, and builders have been reliant on the load bearing capacity of the masonry since the beginning. The majority of Charleston’s brick residences were built in the 19th century, with some surviving from the 18th century. The bearing walls of these historic brick structures are generally quite thick – sometimes three wythes or greater in the lower portions of the exterior walls. The primary brick bond used in the earliest surviving colonial structures is Flemish bond, typically laid on the front and side façades, and English bond on the rear facades. Masonry units are primarily locally-made Charleston ‘grey’ brick (in reality a reddish-brown hue) with a mortar that was also locally produced with a high content of lime made from burning oyster shells. The building materials of the post-Revolutionary era and of the earliest part of the 19th century differ only slightly from those used earlier. Yet, no significant change in construction techniques of load bearing masonry buildings is evident during this time²⁹.

In 1838, a great fire swept across the Charleston peninsula destroying a large part of the city including a large section of King & Meeting streets and the Ansonborough residential neighborhood. Partly, as a consequence of the great fire, subsequent changes were mandated

²⁸ Maurice Harlan, "Evaluation of Dwelling Types Unique to Charleston", Manuscript Draft, National Association of Home Builders Research Center (Charleston: 1992), 1.

²⁹ Robert Stockton, *The Great Shock: The Effects of the 1886 Earthquake on the Built Environment of Charleston, South Carolina*, (Easley, South Carolina: Southern Historical Press, 1986), 12-13.

for construction methods. So many of the burned structures had been wooden frame that a new law was put into place that required all new construction to be of brick or stone. Consequently, load bearing masonry buildings replaced much of the devastated area, and brick construction grew exponentially. Along with the masons that flocked to town to reconstruct the city following the great fire, came new styles and materials for masonry construction. A new brick bond called American or Common bond was popular among these masons considered ‘from off’. It was regarded as a fairly strong bond, but much more economical than Flemish bond because it utilized less material – consisting of five rows of stretchers to one row of headers. Mortar made from stone lime, which was available in other cities where the masons had worked, also replaced the local burnt shell lime mortar so popular in previous years³⁰. Just as “all the elements of Charleston’s built environment ... all combined with the spectacular natural environment ... make Charleston in 1886 ‘beautiful as a dream’”³¹, so did all of these 17th, 18th, and 19th century elements of Charleston’s load bearing masonry buildings – the quality and method of construction, the variety of brick bonding patterns, the mortar mix recipes - combine with Mother Nature to test the limits of the built environment.

Load Bearing Masonry Buildings in Earthquakes

A deficiency of many historic masonry buildings, besides the innate brittleness of the material, is that they were built before the recognized need for seismic safety precautions. In the 19th century, there were no engineers exclusively responsible for structural systems. “Earthquake-proofing” was essentially not possible and even small safety redundancies were not required. As such, many unreinforced, load bearing masonry buildings exist that do not have the reinforcement needed to maintain public safety during and after a seismic event. The area of greatest concern, and seismic weakness, is the connection between the walls and the floor, ceiling, and roof framing. It is typical that no connections exist; the beams of joists simply rest on the wall or are set in pockets in the brick. However, this system lacks the required lateral support for the wall.

Ground waves generated by an earthquake create dynamic forces that vibrate the structure and change rapidly. Because buildings are primarily designed to resist vertical forces,

³⁰ Stockton, *The Great Shock*, 15-16.

³¹ Stockton, *The Great Shock*, 16.

the horizontal, lateral forces are the most dangerous in earthquakes. Shear forces, which tend to distort the shapes of walls, occur when lateral forces push a wall along its length. Lateral forces are transferred from the ground through shear walls (typically interior, wood-frame partition walls) to diaphragms (exterior masonry walls), and then back to the ground again. If a brick wall is pushed sideways via lateral forces, it will resist until the bond breaks between the masonry units, or the masonry itself breaks. A diagonal crack or sometimes an X-shaped crack, called a shear crack, will appear³². This is most commonly seen above and below window or door openings in the spandrel wall (see figure 3.1).

Reasons for the known poor performance of load bearing masonry structures in past earthquakes are the inherent brittleness, lack of tensile strength, and lack of ductility of the materials – that is, a lack of properties given to reinforced masonry by reinforcing³³. The wider the brick wall in relation to its height, the fewer the openings, and the better the masonry bonding, the more it can resist shear. However, even on an extensive uninterrupted masonry wall, earthquake forces are problematic. The heavier the wall, the greater the inertial forces an earthquake will create within it. Thus, instead of bending to dissipate energy, a heavy, brittle, stiff masonry wall will crack, or the walls may rupture and collapse. Additionally, parapets and gable ends tend to disconnect from the building and fall outward, creating a hazard for people below and sometimes causing the building to collapse. Shockwaves can also reduce the bearing capacity of soils beneath the building in a process known as “liquefaction”, in which the soil behaves structurally like a liquid. Any weight resting on this soil sinks into the ground, causing structural deformations resulting from settlement³⁴. Figure 3.2 illustrates the ways in which unreinforced, load-bearing masonry can fail in the event of an earthquake.

³² Stephen Tobriner, *Bracing for Disaster: Earthquake-Resistant Architecture and Engineering in San Francisco, 1838-1933* (Berkeley: Heyday Books, 2006), 62.

³³ Richard Hess, S.E., “The Shakeout Scenario, A Supplemental Study: Unreinforced Masonry (URM) Buildings”, Report prepared for the United States Geological Survey and California Geological Survey (California: Hess Engineering, 2008).

³⁴ Robert Young, *Historic Preservation Technology* (New Jersey: John Wiley & Sons, 2008), 23.

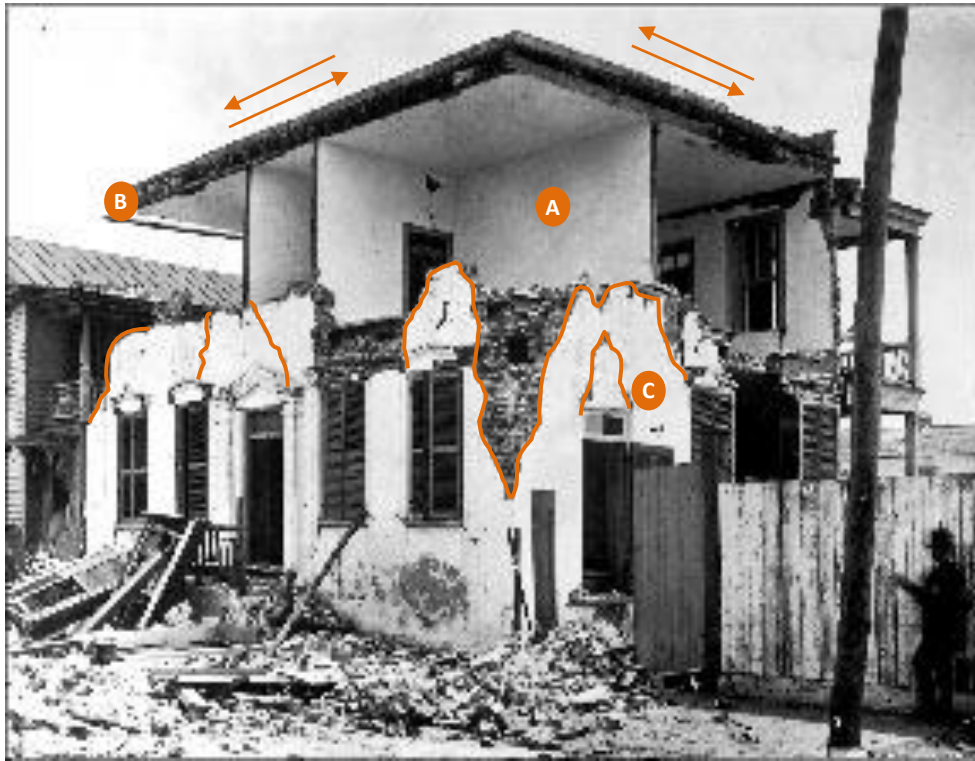


Figure 3.1: Masonry buildings in earthquakes: 157 Tradd Street, Charleston, immediately after the earthquake of 1886. The wooden interior walls (A) moved back and forth on both the ground floor and the second floor. The exterior walls shook, and the diaphragms (B) transferred the load to the exterior walls. However, the whole interior assembly of floors and partitions was more flexible than the brittle exterior walls. In some areas, the walls resisted the shear forces and then broke along the diagonal. The shear cracks are highlighted. Torsion, or the direction of the movement, might explain why the end walls (C) collapsed. (Photo Source: <http://earthquake.usgs.gov>; diagramming by author)

Load bearing masonry failures have been responsible for earthquake deaths in California since at least 1868, most recently as Loma Prieta in 1989 and San Simeon in 2003³⁵. In South Carolina, prior to 1886, the seismic risk of these LBM structures was relatively unknown. Geologically, the city of Charleston lies in one of the most seismically active areas in the Eastern United States. However, not many large earthquakes occurred during the first 200 hundred years of the city's founding, when Charleston was establishing itself as a city and experiencing a high growth rate. It was during that period that many unreinforced, load bearing masonry buildings were constructed. Today, it is generally accepted that the intensity of earthquakes, which are now known to occur in the lowcountry region, would be sufficient enough to cause

³⁵ "Unreinforced Masonry Buildings Fact Sheet" (Earthquake Engineering Research Institute: Northern California Chapter, 2004), www.quake06.org.

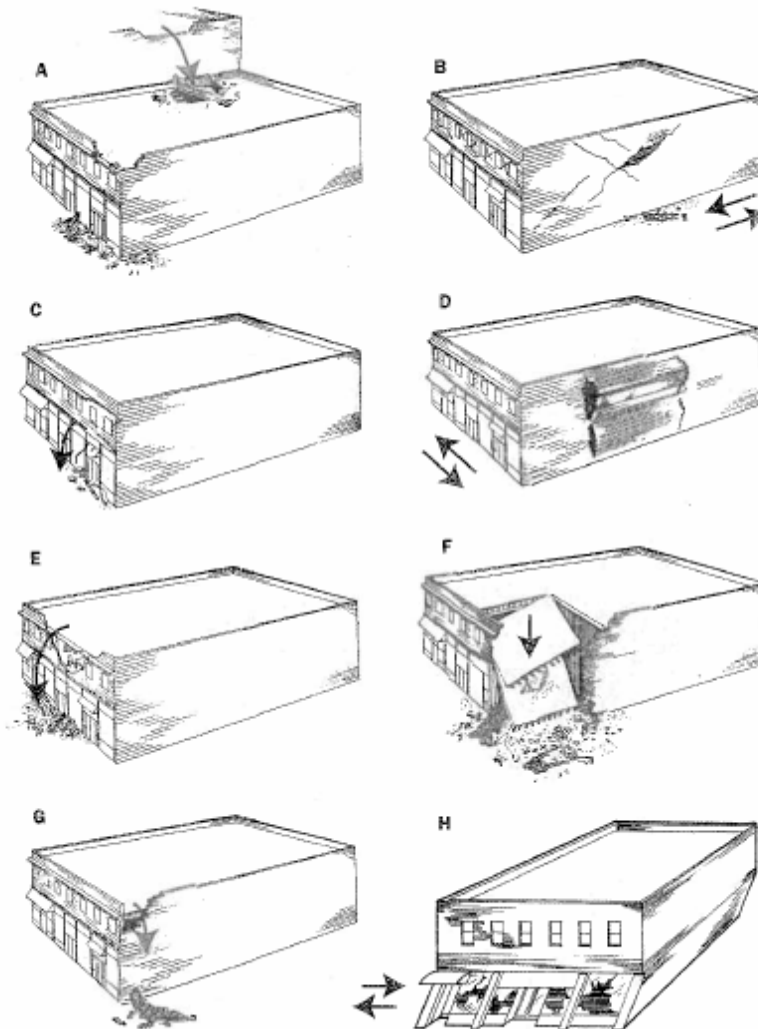


Figure 3.2: The vulnerability of load bearing masonry structures in earthquakes:

- A. Parapet and fire-wall failure
- B. In-plane wall failure resulting in diagonal shear cracks
- C. Nonstructural falling hazards
- D. Wall failure in bending between partition walls, or floors and roof
- E. Façade failure
- F. Roof and/or floor collapse
- G. Wall failure
- H. Soft-story failure

(Source: *Tobriner, Bracing for Disaster, 2006*).

masonry buildings with minimal seismic resistance characteristics to be seriously damaged or collapse. However, not until 1886 were the seismic resistance properties of these Charleston structures truly challenged.

The Earthquake of 1886

In the early summer of 1886, during the month of June and even earlier, several little tremors occurred within the earth, but did not excite much attention. On Tuesday, August 31, 1886, everything in Charleston seemed normal. Then, just before 10:00pm when most had retired to their bedchambers for the evening, a long roll deepened and spread into an awful roar that seemed to come from the earth below. The earth's movement quickly overwhelmed the city and sent everything into a deadly and disorienting pandemonium. As the earth's waves rolled through town, they lifted the ground and the buildings high and then dropped them back again. Over and over again for what seemed like eternity, but only lasted minutes in reality. When it was over, the earthquake had leveled parts Charleston and Summerville causing massive damage, more than 60 deaths, and was so large that it was strongly felt as far north as Chicago. Structural damage extended several hundreds of miles to cities in Alabama, Ohio, and Kentucky, and caused an estimated \$5-\$6 million in damage in Charleston alone. In truth, there was no street in Charleston that did not escape damage. To mention them all in detail would be to no greater purpose. The general nature of the destruction was summed up in comparatively few words by Captain Edward Dutton:

“There was not a building in the city which had wholly escaped injury ... There was not a brick or stone building which was not more or less cracked ... The bricks had ‘worked’ in the embedding mortar and the mortar was disintegrated. The foundations were found to be badly shaken and their solidity was greatly impaired. Many buildings had suffered horizontal displacement; vertical supports were out of plumb, floors out of level, joints parted in the wood work, beams and joists badly wrenched and in some cases dislodged from their sockets.”³⁶ (See figures 3.3 and 3.4).

³⁶ Captain Clarence Edward Dutton, *The Charleston Earthquake of August 31, 1886*, (Washington D.C.: United States Geological Survey, 1890), 248.



Figure 3.3: Cook's Earthquake Views of Charleston and Vicinity: "Calhoun Street" (Source: Charleston Museum Earthquake Photographs, Digital Collection, www.lowcountrydigital.library.cofc.edu)



Figure 3.4: Cook's Earthquake Views of Charleston and Vicinity: "Wentworth Street" (Source: Charleston Museum Earthquake Photographs, Digital Collection, www.lowcountrydigital.library.cofc.edu)

In the direct aftermath and subsequent years following the earthquake of 1886, extensive studies were conducted by the city of Charleston, United States Geological Survey, and other government commissions to determine the effect on the Charleston built environment. On September 4, 1886, a committee of three was appointed by the Secretary of War to inspect and examine the damaged buildings within the city of Charleston, known as the Executive Relief Committee (ERC). Written by William H. Bixby, Frederic Abbott and William E. Speir, the 1886 report titled *The U.S. Government Commission Report on Examination of Buildings in Charleston, SC, Injured by the Recent Earthquake of August 1886* examined approximately 5,000 to 6,000 buildings around the Charleston area³⁷. In the report, Captain Bixby states that “we endeavored to faithfully and impartially decide what buildings were endangering the safety and lives of the people of the city and to indicate briefly the most economical method of rendering these buildings safe for their customary uses”. The focus of the Committee was divided into four main priorities: first priority was given to federal, state and city office buildings and other structures in constant use by the public; second priority included inspection of hospitals, religious structures and factories; the third priority for inspection included firehouses, schools, hotels and assembly halls; the fourth priority was given to private residences and businesses in the lower wards. The Committee estimated \$5-\$6 million in total damages in Charleston alone.

Just as federal engineers were wrapping up their investigation in late September 1886, a more thorough investigation of Charleston’s structures was getting under way. On September 28, 1886, a detailed study of the earthquake damage to the built environment was commissioned by insurance companies doing business in Charleston that year. The report was prepared by a committee of three men, including insurance agents Harry Stockdell, James Thomas and Hutson Lee, along with architect W.H. Parkins and builder Fred S. Stewart. The team documented the condition of 6,956 of Charleston’s buildings in what came to be known as the Stockdell Report³⁸. Following the disastrous earthquake, thousands of dollars in monetary aid flowed into the city from across the nation. In an effort to set up a system for distributing

³⁷ Captain W.H. Bixby, *U.S. Government Commission Report on Examination of Buildings in Charleston, SC, Injured by the Recent Earthquake of August 1886*, (Charleston: City of Charleston Archives, n.d.)

³⁸ Richard Cote, *City of Heroes: The Great Charleston Earthquake of 1886*, (Mount Pleasant: Corinthian Books, 2006), 358.

the aid to residents of Charleston in a fair manner, the local government had already established the Executive Relief Committee. It was the Stockdell Report that was used by the ERC for the purpose of determining aid. After city inspectors submitted their damage assessments to the ERC, the committee awarded money vouchers to residents and property owners seeking assistance for earthquake-related damages to their property. Issued in a pre-determined amount to the property owners, the vouchers were then presented to the contractors doing the repairs. Subsequently, the contractors presented the vouchers back to the city for payment³⁹. By the summer following the earthquake, the ERC reported that it had “disbursed funds in over 2,000 cases of house owners”⁴⁰.

Although the survey reports produced by the Executive Relief Committee and the insurance companies are the most thorough documentation of actual damage caused to the built environment and relief aid distributed to Charlestonians, the report does not make a clear conclusion as to the reasons for damage to certain buildings or certain areas of peninsular Charleston. This study would be conducted four years later. In 1890, the U.S. Department of the Interior and the U.S. Geological Survey published the results of a four year study of the 1886 Charleston earthquake, compiled by Captain Clarence Edward Dutton⁴¹. Unlike the Executive Relief Committee’s report, which primarily focuses on the damage incurred, Dutton’s report attempts to determine and make conclusions about the causes of and the reasons for the extent of damage that the city sustained. One of Dutton’s emerging conclusions was that the amount of damage sustained varied according to the varying nature of the ground on the peninsula. That is, the damage in areas constructed on the original high ground varied from the damages observed in areas on ‘made ground’. In general, Dutton also concluded that there “was not a brick or stone building that wasn’t more or less cracked”, and wood buildings exhibited relatively few signs of shaking or damage. He makes the distinction between the quality of masonry of the colonial period and the quality of a large portion of the brickwork that has been

³⁹ Nicholas Butler, *The City of Charleston’s Executive Relief Committee for the Earthquake of 1886: Money Vouchers for Work Done, September 1886 through June 1887* (Charleston: Charleston County Public Library, 2007), iii-iv.

⁴⁰ Quoted in Butler, *Money Vouchers for Work Done*, iii.

⁴¹ Dutton, *The Charleston Earthquake*,

done in the city since then, specifically noting the difference in durability of the work of the two periods:

“All of these houses are known to have been built before the Revolution ... it can easily be seen how thorough the work was, there being complete adhesion between the bricks and the mortar in which they were laid. The entire walls, when completed, were put together compactly, and have stood for over a century as monuments of the substantial and honest work of the time”⁴².

But as excellent as their quality was, and as carefully as they were laid, the durable character of the walls was attributed more so to the lime from which the mortar was made than the bricks themselves. All of the problems that stemmed from poor post-1838 workmanship and materials soon became apparent when thousands of damaged and fallen buildings were examined after the earthquake. Overthrown walls faced North-South more than East-West, also according to Dutton (see figure 3.5).

These details about masonry and bricklaying might at first appear foreign to the subject at hand - a narrative of the Charleston earthquake. However, they are essential to its being properly understood and to understanding what Charlestonians learned after the earthquake, and how they decided to use it to make their homes habitable and safe once again. Just how did Charlestonians respond to the great earthquake of 1886? The answer to this question is typically looked at from a sociological or political perspective, and the physical effect on Charleston’s built environment has been studied extensively. However, rarely has the history of Charleston’s buildings been looked at in relation to their seismic safety and resistance. Architects and engineers were, in fact, constructing and reconstructing buildings with seismic-resistant features prior to 1886 and after the great earthquake, well before the emergence of seismic requirements in state building codes. Among those concerned with earthquake safety, in addition to the three men composing the Executive Relief Committee, were: John Henry Devereaux, architect, engineer and builder; New York architects William Potter and E.R. Rutledge; city of Charleston engineer Louis Barbot; and the Charleston architecture firm of Abrahams & Seyle⁴³. So just how did the earthquake of 1886 influence these men and how did these men influence earthquake resistant construction in Charleston? As the repairs progressed,

⁴² Dutton, *The Charleston Earthquake*, 228.

⁴³ Beatrice St. Julien Ravenel, *Architects of Charleston*, (Columbia: University of South Carolina Press, 1992).

one feature began to appear on almost every pre-1886 masonry building in Charleston: iron tie rods.

Known in Charleston as “earthquake bolts”, they earned their local name when used to reinforce damaged masonry buildings after the 1886 earthquake. Just one week after the earthquake, Henry Kittridge proposed to Charleston Mayor Courtenay that iron reinforcing rods be used in Charleston:

“Most of the damaged houses in your city can again be drawn into shape and made as strong and reliable as ever, and that, too, without disfigurement, by placing heavy angle iron on the corner, and connecting rods with nuts on the end in such a way as by turning them the displacement will be remedied. These strong rods can remain hidden from sight by ornamental heads.”⁴⁴

When the Executive Relief Committee and teams of insurance agents set out in September to assess the structures, they agreed with Kittridge and specifically recommended the use of reinforcing rods and anchor plates in many of the masonry buildings they encountered:

“All masonry wall should be securely anchored to the floor, ceiling, and roof timbers with iron anchors built into the walls and firmly secured to the timbers ... On each tier of beams there should be at least one anchor to each and every pier between openings and at least one anchor to every eight feet of walls built without openings. In a similar way the tops of all masonry gables should be firmly anchored to the roof timbers.”⁴⁵

Little did these men know that Charlestonians would be so conscientious to their recommendations that iron rods and anchor plates would multiply, and the ‘earthquake bolt’ would become such an enduring icon of Charleston’s built environment.

⁴⁴ Quoted in Cote, *City of Heroes*, 386.

⁴⁵ Bixby, *U.S. Government Commission Report*. Quoted in Stockton, *The Great Shock*, 94-95.

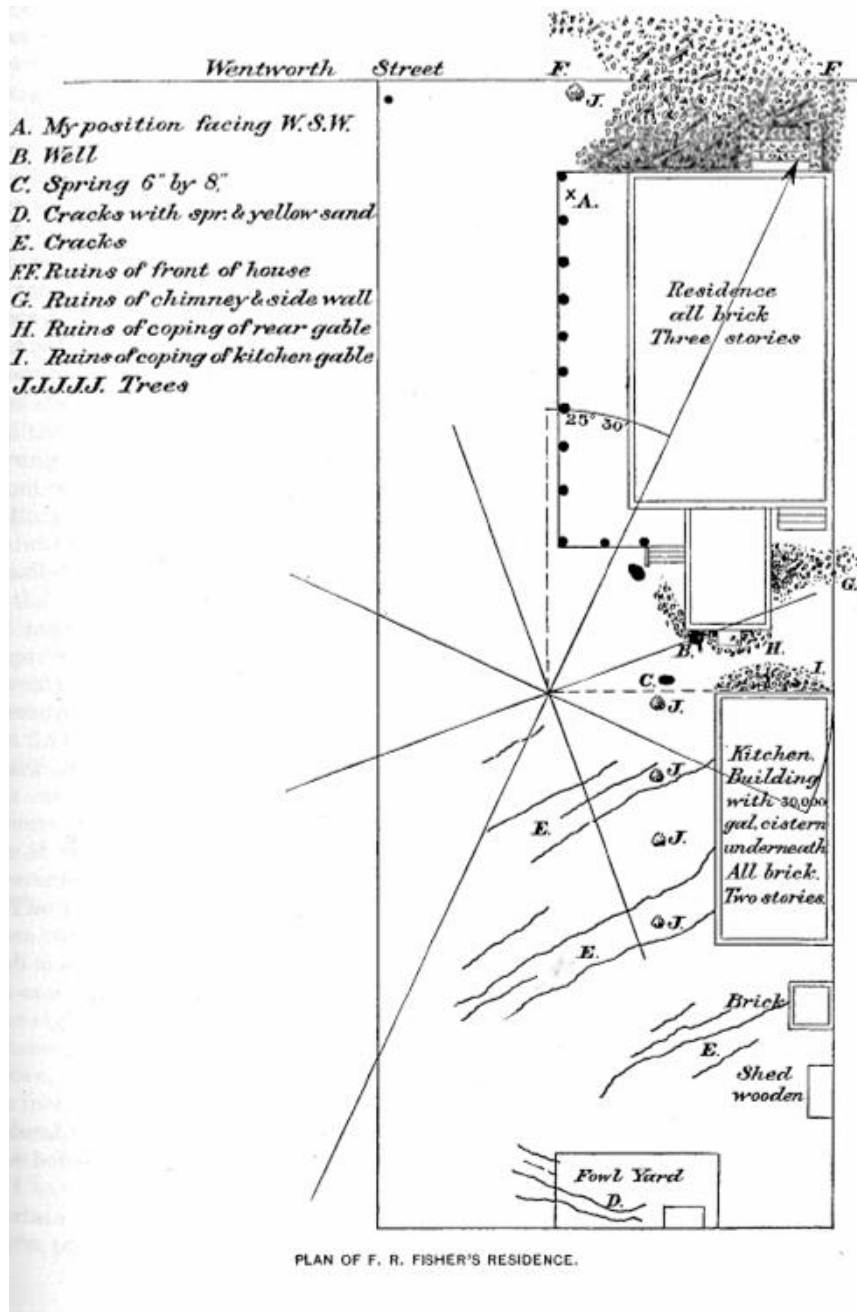


Figure 3.5: Plan of F.R. Fisher's residence in Charleston, SC indicating the damage sustained in the 1886 earthquake. This illustration also diagrams the tendency of overthrown to face North-South more than East-West (Source: Dutton, *The Charleston Earthquake of August 31, 1886* (1890)).

Earthquake Damage and Reinforcement Survey

In order to gain a better understanding of the effect of the earthquake of 1886 on Charleston's built environment, in particular the proliferation of the 'earthquake bolt', the following survey was undertaken on the Charleston peninsula in an area determined to have suffered significant damage. The study documents load bearing masonry structures and the damages assessed in 1886, compares those assessed damages and reinforcement recommendations to documentation of work actually completed, and compiles historical information based on visual examination. For the purpose of this study, earthquake bolts are defined as cast or wrought iron tie rod reinforcement inserted into masonry walls following the post-earthquake recommendations of engineers.

Defining the Survey Area

The general area to be surveyed in this study was initially narrowed down using the Earthquake Damage Distribution map (see figure 3.6) created for a Site Period study conducted by the Citadel Department of Engineering. In 1983, a specific study of site periods as they relate to earthquake vulnerability was undertaken at the Citadel. As a component of this study, 3,888 buildings from the Parkins & Stewart earthquake damage assessment report were analyzed on a block by block basis. The study computed and quantified several items including destruction to masonry buildings categorized by degree of damage, the damage percentage of the walls in each orientation (north, south, east, and west), and an overall building rating computed using area and volume as weighing factors. Subsequently, the average building rating by block was plotted on a map of the city to establish the distribution of damage⁴⁶.

The Citadel's Earthquake Damage Distribution map confirms other post-earthquake reports from the 19th century and their findings: some of the heaviest damage occurred in areas on made ground and closest to the wharves. Thus, the specific survey area for this study is defined by that bounded on the north by Elliot Street and St. Michael's Alley, on the south by Tradd Street, on the east by East Bay Street, and on the west by Meeting Street (see figure 3.7). As seen on the Sanborn Fire Insurance Map from June 1884, the area within these boundaries is comprised of primarily masonry structures, presumably unreinforced before the earthquake.

⁴⁶ Stockton, *The Great Shock*, 62-63.

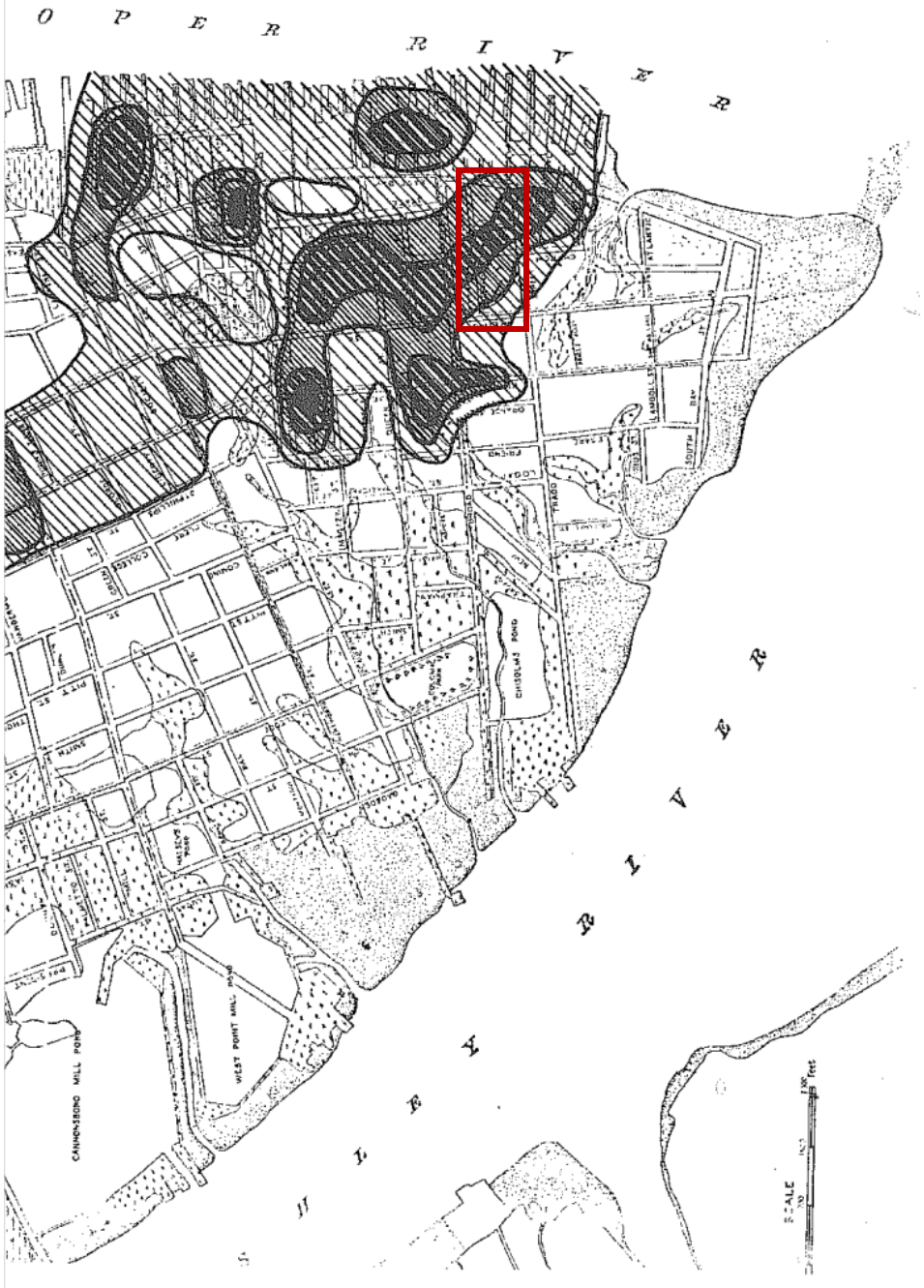


Figure 3.6: Earthquake Damage Distribution map (1983) created by the Citadel Department of Engineering; survey area indicated by red outline. (Source: Printed in Robert Stockton, *The Great Shock: The Effects of the 1886 Earthquake on the Built Environment of Charleston, South Carolina*, (Easley: Southern Historical Press, 1986))

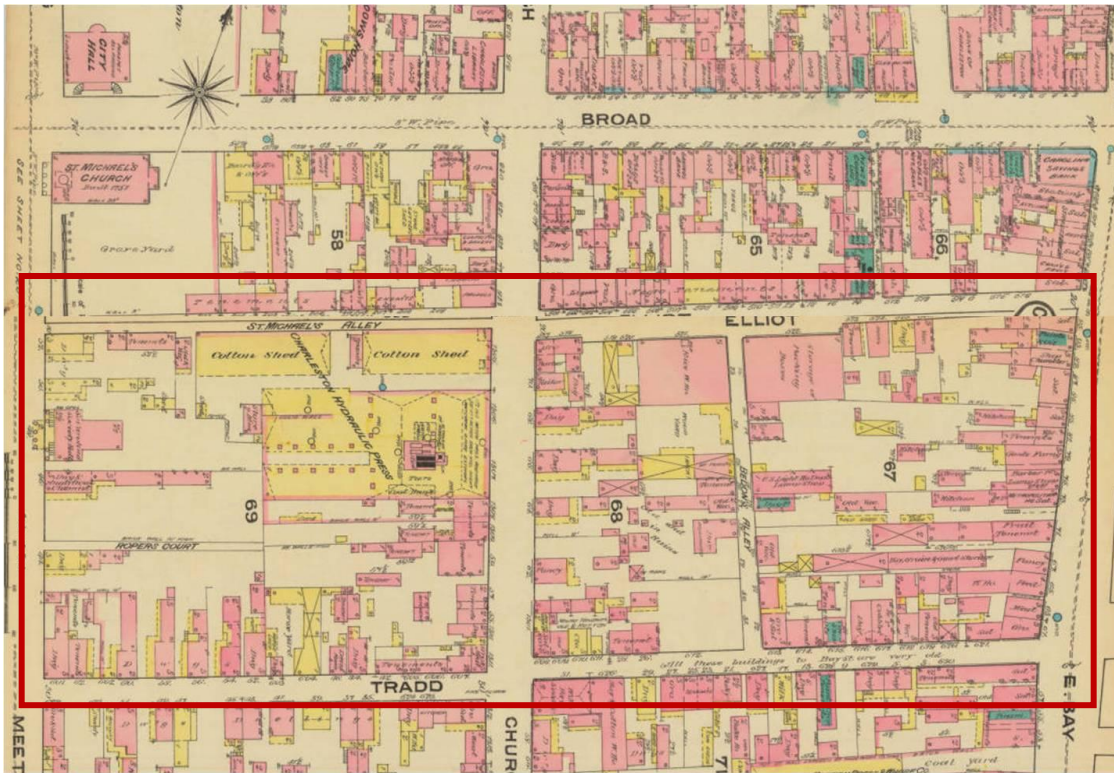


Figure 3.7: Sanborn Fire Insurance Map from June 1884; buildings within the survey area indicated are included in this study. (Source: University of South Carolina Digital Collections, Sanborn Fire Insurance Maps, <http://library.sc.edu/digital/collections/sanborn.html>)

All reinforcement which qualifies as “earthquake bolts”, as previously defined, was examined, photographed and included in the survey.

Archival Research

The following study brings together two main sources of data: the record of earthquake damages based on assessments initiated in 1886 for an association of insurance companies, and the transcription of vouchers issued to individual property owners for structural repairs based on the monetary value of damages determined in the preceding assessment. The Stockdell Report’s damage assessments for the specific properties included in this survey have been transcribed into Table 3.1 at the end of this section. In their report, the team recorded the construction material of each building and its roof; the length, width and height of each building; the condition of each of its walls and chimneys; the estimated cost of repairs; and specific recommendations for repairs. Currently, the complete damage assessment report is

held at the Historic Charleston Foundation. Also included in the same table, for comparison purposes, is the work completed using money vouchers issued by the Executive Relief Committee from September 1886 through June 1887. Edited and compiled by Nicholas Butler, a transcription of the earthquake repair vouchers is currently held in the South Carolina Room of the Charleston County Public Library. Each voucher includes: “the name of the recipient, the address of the damaged building, the unique voucher number, the dollar amount awarded, the issue date of the voucher, and special notes or instructions regarding the work done”⁴⁷.

All masonry structures falling within the survey boundaries are listed, and those with anchoring and/or reinforcement recommendations have been highlighted⁴⁸. If any field within the table is left blank, this indicates no recorded condition in the 1886 assessment. In any field where “G” is recorded, a ‘good’ condition was indicated in the assessment. Additionally, the following terms are used frequently in the assessments, and thus warrant definition⁴⁹:

Slightly Cracked – where a wall showed but few slight cracks principally at openings and under windows, of little consequence and of which no serious results can follow

Cracked – walls that showed breaks in the more solid parts, and which seem to point to interior fractures, and which should have close examination

Badly Cracked – a wall that appeared broken at corners, in the solid parts between openings, and had separated from interior and adjoining walls

Field Investigation

A visual field investigation conducted by the author (2012) provides another element of analysis included in this survey. The information gathered in the investigation is compiled into property survey forms for each structure examined (see Appendix A). The forms were designed to include information specific to this survey, including historical data as well as physical materials and conditions of the visible reinforcement. For each property, the damage assessment remarks are summarized along with the author’s current observations of the structure and visible reinforcement. A number of additional sources were consulted to provide an estimated date of construction for each of the properties examined. Sources of information

⁴⁷ Butler, *Money Vouchers for Work Done*, iv.

⁴⁸ Any frame structures have been omitted from the table due to survey irrelevance.

⁴⁹ *Record of Earthquake Damages, 1886*. On microfiche, Historic Charleston Foundation Archives.

include literature on Charleston architecture, newspaper articles, National Register nomination forms, and the Historic American Buildings Survey website⁵⁰.

Exterior photographs of the buildings and the earthquake bolts are included as part of the field investigation component of the survey. Photographs for each property include, whenever possible, an overall image and a detail of the reinforcement that is visible. The overall photographs offer a complete image of the facades illustrating scale and location of earthquake bolt placement, which is fundamental to understand. The detail photos offer information in regards to style and condition of the anchor plates. In many cases, due to the narrow nature of the residential streets, photos of full facades were near impossible and some of the photos may appear distorted. However, in the interest of accurate representation, none of the photographs included in this survey have been digitally corrected or altered. All photographs were taken by the author with a Nikon Coolpix S550 digital camera, unless otherwise noted on the individual property survey forms.

Survey Findings

In the first steps of investigation, which primarily included the archival research components, significant information was gathered of the great effects of the 1886 earthquake on the Charleston built environment. The *1886 Record of Earthquake Damages* report proved extremely useful in determining how each structure was affected by the disaster. It was initially thought that the money vouchers for work completed would prove a useful tool for further interpretation of the repair work done following the earthquake. Unfortunately, this was not the case. Most vouchers provide scarce details about the specific work done for repairs, while only a few include many notes at all about the repairs. Further research about the vouchers themselves and the information they contain is needed before they can prove useful in other applications. Nonetheless, the study was concluded using a combination of the damage assessment reports and visual investigation.

In 1886, there were 82 properties with 73 masonry structures within the survey area that were included in the damage assessment report. This discrepancy is due to the fact that, in

⁵⁰ Unless otherwise noted, sources cited for Appendix A are: Jonathan Poston, *The Buildings of Charleston: A Guide to the City's Architecture* (Columbia: University of South Carolina Press, 1997); Library of Congress, *Historic American Buildings Survey Collection* (<http://memory.loc.gov>); SC Department of Archives and History, State Historic Preservation Office, *National Register Sites in Charleston County, SC* (<http://www.nationalregister.sc.gov/charleston/nrcharleston.htm>)

some instances, one structure may have dual addresses (i.e. the structure on the corner of Tradd and East Bay streets is in fact identified as 0-2 Tradd and 79 East Bay, or the warehouse on Elliot occupied numbers 21-27 Elliot Street). In instances such as these, the structure itself was only counted once for the purpose of these results. Also, a handful of structures within the survey area were assessed in 1886, but no longer exist today for reasons unknown. These structures were not included in the final reinforcement counts, simply because it cannot be known if they were in fact reinforced with tie rods following the earthquake.

Interestingly, the 1886 damage assessment reports specific post-earthquake recommendations to anchor walls or install iron tie rod 'earthquake bolts' for 34 out of the 73 structures, or for approximately 47% of the buildings. Subsequently, the visual survey revealed that 17 of these structures show visible reinforcement on the exterior. Limitations aside, this would mean that half of the buildings recommended to be anchored were actually repaired in this manner, and only 23% of all the structures within the survey area were reinforced as recommended with 'earthquake bolts' following the 1886 disaster, or approximately 1 out of every 4. An additional 17 structures show visible evidence of tie rod reinforcement that was not specifically recommended in the 1886 report.

The nature of this survey has its obvious limitations. The 1886 damage assessment reports are flawed in that it took several months for the surveyors to complete their work. By the time they made it to a number of the properties, the buildings had likely been repaired in some capacity already. The date each building was surveyed is not included in the report, and notes on repair work already completed are not always detailed. Additionally, the field investigation component of this survey is limited. The visual observations of earthquake reinforcement only include what could be seen from the public right-of-way. Only a small number of structures had four visible facades. There is also the high probability that many of the structures within the survey area have had stucco repair done in the past 125 years. And while it is possible to work around the exterior patress plates leaving evidence of reinforcement visible, this is not always the case, nor the preference of the homeowner.

Although limited, these findings are significant. To a certain degree, they indicate the frequency of the 'earthquake bolt' as a remediation method in 1886-1887. They also provide a sampling of the most common forms of tie rod anchor plates, specifically in Charleston, South

Carolina. Visual surveillance substantiates that although earthquake bolts were cast in a variety of shapes, they were typically plain. Most common of all are the circular style: some with decorative profiles, others with completely flat in profile and devoid of decoration. In other instances, building owners chose to disguise them with cast iron decorations, such as lions' heads, or stars. The images in Figure 3.8 exemplify various types of earthquake bolts around the city of Charleston.



Figure 3.8: Various forms of ‘earthquake bolt’ patrix plates observed around the city of Charleston (All photos taken by author).

Top Row: 28 Tradd: Circular plate with decorative profile; 107 East Bay: circular plate with flat profile

Second Row: 11 St. Michael’s Alley: star-shaped cast iron patrix plate; 9 East Battery: decorative cast iron lion’s heads disguise the tie rod ends

Third Row: 22 Elliott: rare, rectangular-shaped plates; 14 George: rare, rectangular corner bolts are integrated with exterior ornamentation

Bottom Row: 79/81 Church: ‘X’ shaped plates are one of the earliest forms of patrix plates, but still popular post-earthquake; 91 Exchange Street: ‘butterfly’ anchors are typical of the 18th century

TABLE 3.1: EARTHQUAKE DAMAGE AND REINFORCEMENT SURVEY: 1886 RECORD OF EARTHQUAKE DAMAGES/WORK COMPLETED WITH EARTHQUAKE MONEY VOUCHERS

Property	Owner/Occupant	Material		Dimensions (feet)			Condition of Walls				Condition of Chimneys	Estimated Damages (\$)	Remarks & Recommendations	Work Completed with Earthquake Money Vouchers, 1886-1887	Author Notes & Comments
		Building	Roof	L	W	H	N	S	E	W					
*81 Church	G. Duger (dwelling)	Brick	Tin	65	13	23	G	G	G	G	G		Has been repaired.		Corner of Church & Tradd
*82 Church	H. Viohl (store & dwelling)	Brick	Slate	40	20	35	Division	Badly cracked	Division	Slightly cracked	G	150	Anchor east and west at floor of each story		
83 Church	Mrs. N. Whitehead (dwelling)	Brick	Tin	40	30	22	G	G	Slightly cracked	Sprung on ends	Now good	50	Anchor on west wall.		
*84 Church	H. Viohl (dwelling)	Brick	Tin	40	20	30	G	G	G	G	G		OK		
85 Church	George Cormer (dwelling)	Brick	Tin	40	30	22	G	G	Slightly cracked	Sprung on ends	Now good	50	Anchor on west wall		
86 Church	A. Gellfuss (dwelling)	Brick	Slate	40	30	40	G	G	G	G	G		OK		
87 Church	H.M. Fuseler (store & dwelling)	Brick	Tin	50	30	40	G	G	G	G	G		OK		
*88 Church															No damage assessment or money voucher exists for 88 Church Street. However, the structure predates the earthquake of 1886, and was therefore included in the visual survey.
89-91 Church	Mr. Horlbeck (dwelling)	Brick	Slate	50	40	35	Slightly cracked	Slightly cracked	Slightly cracked	Slightly cracked	Now being rebuilt		Anchor north to south, and east to west; attention to kitchen chimney to foundation		
*90 Church	Mrs. H. Adams (dwelling)	Brick	Slate	45	25	35	G	G	G	Badly cracked	Tops all down to roof	50	Rebuild chimneys; Repair west wall and anchor it to floor beams; Kitchen ok		
*92 Church	Mr. Dowle (dwelling)	Brick	Slate	65	25	40	G	G	G	Badly cracked at corner	Tops all down to roof	75	Repair kitchen wall & rebuild the main house chimneys from under the roof		
94 Church	George Paul (dwelling)	Brick	Slate	60	25	40	G	G	G	Slightly cracked	G		OK		
96 Church	Estate C. Momier (tailor shop & dwelling)	Brick	Tile	35	18	30	G	Slightly cracked	G	Badly cracked	G	150	OK; anchor west wall		
98 Church	Estate C. Momier (dwelling)	Brick	Tile	50			Slightly cracked	G	G	Slightly cracked		500	Now good		
99 Church	Estate C. Momier (store & dwelling)	Brick	Slate	35	45	35	G	Cracked openings	Division	Cracked	G		Now good		No longer existing
*100 Church		Brick		35	30	25	Badly bulged	Badly cracked	Cracked	Must come down	Down		Building should come down		
*79 East Bay	J. Klinke	Brick	Tin										Not in an insurable condition unless rebuilt from ground		Corner property; See also 81 East Bay, 0-4 Tradd; counted as 1 structure in survey
*81 East Bay	J. H. Klinke (vacant)	Brick	Slate	50	25	30							Not in an insurable condition unless rebuilt from ground		See above.
*83 East Bay	L. Sahlmann (grocery & dwelling)	Brick	Slate	100	30	50	G	G	Badly cracked between openings	Badly cracked between openings	Tops must be taken down below ceiling joists	600	Should be well anchored; rear building in good condition		
*85 East Bay	L. Sahlmann (storage)	Brick		50	30	50	G	Division good	Must come down	Must come down	Must be taken down to 2d story & replaced	3000	Only by rebuilding; now valueless		
87 East Bay	C.O. Witte (store & dwelling)	Brick		75	30	50	G	Slightly cracked	Badly cracked between windows	Badly cracked between windows	Tops down	530	East and west walls should be well anchored		

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Property	Owner/Occupant	Material		Dimensions (feet)			Condition of Walls				Condition of Chimneys	Estimated Damages (\$)	Remarks & Recommendations	Work Completed with Earthquake Money Vouchers, 1886-1887	Author Notes & Comments
		Building	Roof	L	W	H	N	S	E	W					
89 East Bay	Mrs. R. Murphy (saloon)	Brick	Tin	75	25	35	G	Slightly cracked over openings	Slightly cracked over openings	Badly cracked at corner	One top off; one cracked between story at front	1700	Cracks in east and west walls repaired and well anchored; chimneys require prompt attention; chimney in 2d story room should come down	Voucher #1272 issued for \$700 on 4 November 1886; Accepted under Resolution 19 November 1886 Objections: Cracks in back walls	
*91 East Bay	Jas. Riley (store & dwelling)	Brick	Tin	60	25	35	G	G	G	G	Tops down, upper stores inaccessible	30	Rebuild chimneys		
93 East Bay	Jas. Riley (store & dwelling)	Brick	Tin	60	25	35	G	G	G	G		30	Rebuild chimneys		
*95 East Bay	Jas. Riley (store & dwelling)	Brick	Tin	60	25	35	Division good	Division good	Badly cracked over openings	Badly cracked		917	Rebuild chimneys; repair east and west walls		
*97 East Bay	Redding, Agent (vacant)	Brick	Tile	35	25	30	Division good	Division good	Badly cracked	G	Tops off	350	Repair chimneys; repair roof and east wall and plaster	Voucher # 0024 issued for \$370 (Walls - \$150, Roof - \$150, Chimney - \$70) Accepted under Resolution 19 November 1886 Objections: Some of the cracks in walls only filled with cement. Dutrieux resided at 61 Beaufain Street	
99-101 East Bay	J.W. Oldenbuttel (store & dwelling)	Brick	Tile										In good condition		
*103 East Bay	W.C. Millar (store & dwelling)	Brick	Tile	50	25	40	Badly sprung	Badly sprung	Gables out, now boarded	Gables out, now boarded	Must come down to 2d story	900	East and west walls should be replaced; Kitchen not insurable		
105 East Bay	W.C. Millar (store & dwelling)	Brick	Tile	30	25	35	Division good	Division good	Slightly cracked	Slightly cracked	Good	150	Attention to east wall; Building in rear uninsurable		
*107 East Bay	J.W. Ottenbuttel (store & dwelling)	Brick	Tile	35	25	35							Has been anchored but is not safe; should come down		Corner property; see also 1 Elliot; counted as 1 structure in survey
1 Elliot	-- Oldenbuttle (vacant)	Brick	Tile	16	14	20						100	Kitchen – rebuild gable end from 2d story		Corner property; See also 107 East Bay; Counted as 1 structure in survey
2 Elliot	-- Oldenbuttle (dwelling)	Brick	Tin	40	16	35	G	G	G	G			OK		No longer exists. Lot now occupied by "Printer's Row"
*4 Elliot	J. Duffus (tenement)	Brick	Tile	35	16	35	Slightly cracked	Badly cracked	Division	G	G	40	Anchor south wall at each floor and repair cracks; rear building not insurable	Voucher #1130 issued for \$390 on 1 November 1886 Accepted under Resolution 19 November 1886 Objections: Walls not properly done Duffus resided at 25 Amherst Street	
5 Elliot	R.T. Thompson (dwelling)	Brick	Tin	22	18	18	Repaired	Repaired	Repaired	Repaired	Rebuilt	250	Brick kitchen a complete wreck; should come down	Voucher #0937 issued for \$385 on 25 October 1886	Under construction (renovation) at time of survey
8 Elliot	G.I. Cunningham (vacant)	Brick	Slate	35	30	35	Badly cracked over openings	Badly cracked at openings	Slightly cracked	Cracked at openings	Rebuilt	350	Anchor building at each floor both ways; repair cracked parts of wall with new work		
*10 Elliot	New & Courier printing office	Brick	Tin	30	35	35	Bad	Bad	Bad	Bad			Walls on all sides badly cracked; not insurable		
11 Elliot	Vacant	Brick	Tile	38	22								Condemned; should be taken down immediately, dangerous to life.		No longer exists
12 Elliot	George W. Williams (vacant)	Brick	Tile	45	30	35	Badly sprung	Badly cracked	Division	G	G	40	Anchor south wall at each floor and repair cracks		

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Property	Owner/Occupant	Material		Dimensions (feet)			Condition of Walls				Condition of Chimneys	Estimated Damages (\$)	Remarks & Recommendations	Work Completed with Earthquake Money Vouchers, 1886-1887	Author Notes & Comments
		Building	Roof	L	W	H	N	S	E	W					
14 Elliot	Mr. Mosely (tenement)	Brick	Tin	30	20	20	Cracked at opening	G	Division	G	Repair tops	50	Repair cracked part of window arches		
16 Elliot	-- Hargrave	Brick		40	25	30	Down	Down	G	Division	Down	1250	Rebuild north and south walls entirely; put on new roof and chimneys	Voucher #1160 issued for \$650 on 1 November 1886 Accepted under Resolution 19 November 1886 Objections: Crack in walls filled with cement McCarthy resided at 41 President Street	
*18-20 Elliot	G.I. Cunningham (vacant)	Brick	Tile/Tin	40	25	30	Down	Down	Division	Division	Down	1800	Rebuild north and south walls of number 18 entirely; repair north and south walls of number 20 and anchor; put on new tin roof; rear buildings old & dilapidated, not insurable		Counted as two structures in survey.
*22 Elliot	G.I. Cunningham (tenement)	Brick	Tile				Badly cracked, top off	Slightly cracked	Division	Division, gable down	Down	900	Rebuild west gable; repair north and south walls and anchor at each floor; take off tile and put on tin; rebuild chimneys		
21-27 Elliot	D. Talmare & Sons (warehouse)	Brick	Tin	100	75	35	Down to 2d story	Down to 2d story	G	G	None	1800	Rebuild north and south walls from 2d story; anchor walls and repair roof		No longer exists.
28 Elliot	G.I. Cunningham (tenement)	Brick	Slate	30	30	30	G	G	G	G	Down and dangerous	150	Rebuild chimneys from below roof and repair roof; rebuild south wall of rear building		
30 Elliot	Mr. Plenge (vacant)	Brick	30	25	25							1000	Needs a general overhauling and a new roof to be made insurable		
32 Elliot	G.I. Cunningham (vacant)	Brick	Slate	45	30	35	Must come down to 2d story	Slightly cracked	Division	Division	Down	350	Rebuild upper portion of north wall, chimneys and repair roof		
34 Elliot	-- Stiver (tenement)	Brick	Tin	40	25	25	Badly cracked	Badly cracked over openings	Division	Division	G	350	Rebuild center portion of north wall; repair cracks in south wall & anchor wall		
36-38 Elliot	Estate C. Momier (saloon & dwelling)	Brick	Slate	40	40	30	Slightly cracked	Slightly cracked	Slightly cracked	Slightly cracked	G		Has been put in repair		
*60 Meeting	B.F. Kramer (dwelling)	Brick	Slate	75	30	35	Slightly cracked over openings	Badly cracked	Badly cracked	G	G	350	Should be securely anchored		
68 Meeting	Dr. C.W. Shephard Jr. (laboratory & dwelling)	Brick	Tin				G	G	G	G	Being rebuilt		OK		
70-72 Meeting	South Carolina Society Hall	Brick	Tin	85	50	50	Wing must come down	Wing must come down	Must come down	Badly cracked	Must come down	6000 – 10000	Rebuild entire upper portions; foundation good		
2 St. Michaels Alley	Mrs. Kennedy (dwelling)	Brick	Slate	28	30	25	G	G	G	Division	Good, top off of one	75	Repair chimneys and anchor veranda to wall	Voucher #0199 issued for \$150 on 7 October 1886 (Walls - \$45, Roof - \$40, Piazza - \$40, Gutters - \$25) Accepted under Resolution 19 November 1886 Objections: Related to crack in wall	
4 St. Michaels Alley	J.T. Redding (dwelling)	Brick	Slate	30	16	22	Top part down	Rebuild top part	Division	Division	Top down	100	Rebuild top part of north wall; repair roof and rebuild chimneys		

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Property	Owner/Occupant	Material		Dimensions (feet)			Condition of Walls				Condition of Chimneys	Estimated Damages (\$)	Remarks & Recommendations	Work Completed with Earthquake Money Vouchers, 1886-1887	Author Notes & Comments
		Building	Roof	L	W	H	N	S	E	W					
6 St. Michaels Alley	-- Dingle (dwelling)	Brick	Slate	30	16	22	Repair gable	G	G	G	G	25	Repair and anchor gable and north wall		
*8 St. Michaels Alley	-- Moodie (dwelling)	Brick	Slate	20	50	25	Cracked	Cracked	G	G		300	Rebuild parapet and anchor walls		
10 St. Michaels Alley	-- Muckenfuss (tenement)	Brick	Tile	25	30	22	G	Badly cracked	Down	Division	Good, east one top off	250	Rebuild east wall from 1 st story up, take out cracked portion of south wall and anchor back; rebuild roof and rebuild chimney		
*11 St. Michaels Alley	-- Redding (dwelling)	Brick	Tile	25	18		Cracked	Cracked	Cracked	Cracked		535	Rebuild south and east walls from foundations; walls anchored		
14 St. Michaels Alley	C.O. Witte (dwelling)	Brick	Slate	40	20	10	Cracked		Cracked		Bad	200	Rebuild chimneys down to roof; rebuild and anchor walls		No longer exists.
*2 Tradd	John Klinck												A wreck		Corner property; see also 79-81 East Bay, 0-4 Tradd; counted as 1 structure in survey
*4 Tradd	-- Solomon (dwelling)	Brick	Tile	40	25	35	Badly cracked	Badly cracked	Cracked	G	Down	250	Repair roof and rebuild chimneys from below roof; anchor and rebuild cracked portions of walls		See above.
*6 Tradd	John Klinck (dwelling)	Brick	Tin	50	42	40	Badly cracked	Badly cracked	Division good	Slightly cracked	Down	200	East and west walls have been anchored; repair cracked portions of north and south walls; rebuild chimneys and repair kitchen		
*12 Tradd	Wm. Parker (shop & dwelling)	Brick	Tin	40	24	35	G	G	G	G	G – rebuilt	250	Has been put in good repair		
*14-16 Tradd	J.L. Ahrens (shops & dwelling)	Brick	Tin	40	40	35	G	G	G	G	G – rebuilt	325	Has been put in good order, OK		Counted as 2 structures in survey
18 Tradd	J.L. Ahrens (store & dwelling)	Brick	Tin	95	25	35	Down	G	Cracked	Rear part cracked	Partly down	1200	Repair as indicated		
*26 Tradd	Pat Donohue (dwelling)	Brick	Slate	40	25	35	G	G	G	G	G		Has been repaired, OK	Voucher #1206 issued for \$178 on 3 November 1886 (\$43 for 26 Tradd, \$35 for 28 Inspection St) Donohue resided at 27 Marsh Street	
*28 Tradd	Mrs. N. O'Donnell	Brick	Tin	30	30	35	G	Top part down	G	G	Down	400	Repair top of south wall; rebuild chimneys & repair kitchen walls	Voucher #0035 issued for \$470 on 30 September 1886 (Walls - \$350, Roof - \$30, Frame - \$30, Chimneys - \$60) Accepted under Resolution 19 November 1886 Objections: Cracks in walls only filled up	
*32 Tradd	Mr. Roberts (vacant)	Brick	Slate	30	25	35	Badly cracked	Top part down	Division	Division	Down	500	Rebuild south wall and repair cracks; rebuild north wall; repair roof & rebuild chimneys; rear building is a wreck	Voucher #1658 issued for \$718 on 23 November 1886 (Also issued for 32 Church Street) Accepted under Resolution 19 November 1886 Objections: Walls cracked and only filled up with cement	
38 Tradd	Mr. Daden (store & dwelling)	Brick	Tin	22	25	22	G	G	G	G	Slightly cracked		Cracks in chimneys should be closely examined		
40 Tradd	Mr. Whitehead (tenement)	Brick	Tin	35	16	22	Slightly cracked	G	Division	Division	Down	50	Rebuild two chimneys on north wall		
42 Tradd	Mr. Whitehead (tenement)	Brick	Tile	20	12	18	G	G	G	G	G		OK		

TABLE 3.1: EARTHQUAKE DAMAGE AND REINFORCEMENT SURVEY: 1886 RECORD OF EARTHQUAKE DAMAGES/WORK COMPLETED WITH EARTHQUAKE MONEY VOUCHERS

Property	Owner/Occupant	Material		Dimensions (feet)			Condition of Walls				Condition of Chimneys	Estimated Damages (\$)	Remarks & Recommendations	Work Completed with Earthquake Money Vouchers, 1886-1887	Author Notes & Comments
		Building	Roof	L	W	H	N	S	E	W					
44 Tradd	Mrs. K. Warren (dwelling)	Brick	Tile	35	25	30	G	G	Top part down	Slightly cracked	Tops off	200	Chimneys badly cracked below roof, should be taken down and repaired to be safe; rebuild east wall and repair roof	Voucher #0110 issued for \$115 on 4 October 1886 (Walls - \$45, Roof - \$30, Chimneys - \$40) Warren resided at 83 Columbus Street	
*46 Tradd	Thomas Tully (dwelling)	Brick	Tile	40	25	35	Cracked at opening	Cracked over openings	G	G	Now rebuilding	150	Anchor north and south walls		
48 Tradd	Mr. Dougherty (shop & dwelling)	Brick	Tile	35	25	12	G	G	G	G	G	50	Kitchen – repair and anchor walls and rebuild chimneys from below roof		
52 Tradd	Dr. Dawson (dwelling)	Brick	Tin	25	12	20	G	G	G	G	G		OK		
*54 Tradd	Mr. Dougherty (dwelling)	Brick	Tile	50	25	35	Cracked	Cracked at openings	G	G	G		Repair walls at openings and properly anchor	Voucher #0276 issued for \$210 on 9 October 1886	
56 Tradd	Mrs. C. Mahoney	Brick					Cracked at corner	Slightly cracked	G	G	Tops down	50	Repair and anchor walls; rebuild chimney tops	Voucher #1826 issued for \$140 on 23 December 1886 Accepted under Resolution 19 November 1886 Objections: Walls very badly cracked	
*58 Tradd	Dr. Kellers (dwelling)	Brick	Tile	45	25	35	Badly cracked	Badly cracked	Cracked	G	Now being rebuilt	150	Walls need anchoring north and south		
*60 Tradd	Dr. Kellers (dwelling)	Brick	Tile	65	25	35	Badly cracked	Badly cracked	G	G	Now being rebuilt	100	Anchor and repair south wall		

Sources: *Record of Earthquake Damages, 1886*. On microfiche, Historic Charleston Foundation Archives; Nicholas Butler, *The City of Charleston's Executive Relief Committee for the Earthquake of 1886: Money Vouchers for Work Done, September 1886 through June 1887* (Charleston: Charleston County Public Library, 2007).

* Indicates the structure shows visible exterior evidence of tie rod reinforcement and a form has been included in Appendix A for the property.

Notes:

1. If left blank, no entry was reported in the assessment of damages. G = good condition.
2. Only brick structures within the defined survey area are included in this chart. Frame structures have been omitted for their irrelevance to the survey.

CHAPTER FOUR

CASE STUDIES

Methodology

Case studies are widely used in many professions like law, engineering, business, planning, and architecture. The practice is also becoming increasingly common in historic preservation. The primary body of knowledge in historic preservation is contained within written and visual documentation – that is, stories of projects. Together, the following illustrated case studies provide a collective record of the technological advancement and development of iron tie rod reinforcement in Charleston’s load bearing masonry buildings.

The following narratives discuss the construction history and reinforcement technology in three of Charleston’s historic load bearing masonry structures. The selection of case studies covers a range of time periods and provides insight into tie rod technology of the 18th, 19th and 20th centuries; each account includes a construction history of the respective building as it relates to structural reinforcement campaigns, descriptions of the various tie rods, and field sketches and photographic representation. The first case study represents an early 18th century military structure, illustrating one of the most unique load bearing masonry structures and perhaps the earliest form of iron tie rods in the lowcountry region. The second case study examines two separate reinforcement campaigns in one of Charleston’s wealthiest residences, both responses to natural disaster in the 19th century. While the third and final case takes a look at the oldest institutional structure on the College of Charleston’s campus, presumably with pre-1886, post-1886 and 20th century tie rod reinforcing. The chosen case studies do not provide information on the use of iron tie reinforcement in middle and lower class Charlestonian residential structures due to the fact that many are still private residences today, and access to the structures are therefore limited. The damage and reinforcement area survey completed in Chapter Three of this thesis (see also Appendix A) provides a surface investigation into the reinforcement of masonry structures, primarily residences, within Charleston following the earthquake of 1886.

The primary goals for including these case studies are to provide documentation of existing iron reinforcement in Charleston and descriptions of technical and historical

information. It is not the aim of this case study presentation to analyze manufacture quality, structural integrity, or date of manufacture. Parts of the case study discussions touch on the approximation of age and estimated date of installation of the iron tie rods, if it is not already known. However, it is very difficult to determine a tie rod's exact origin and date of manufacture. Only an analysis of construction and design correspondence, which is limited to letters, drawings, committee minutes, etc. would perhaps reveal these facts if they survive.

79 Cumberland Street: Old Powder Magazine

Site and Construction History

The Old Powder Magazine at 79 Cumberland Street, one of the oldest public buildings in Charleston, was constructed as an early storehouse for the public and the city's gunpowder. In 1703, the Commons House of Assembly authorized "that a brick powder house be built ... within the said line ... which said lott, or part of a lott ... shall be and remaine, with the house thereon to be built, for the sole use of and benefitt of the publick"⁵¹. However, it wasn't until 1712 that £50 was allotted from the public treasury to build a powder magazine, which was completed in 1713. Still, it was not weather-tight, and did not keep powder acceptably dry. The roof was eventually reworked, and in 1719 the Powder Magazine officially became the repository of all government-owned powder. But the building continued to suffer from structural issues and dampness. In 1739, The Commons House made the following suggestions for repairs⁵²:

1. *A new floor of Cypress or Pine to be fastened with pegs.*
2. *The walls inside of the magazine to be lined with boards.*
3. *A new outer door to be well fortified with nails.*
4. *New & stronger window shutters.*
5. *That the passage between the two doors be rammed with clay and not to be boarded.*
6. *That while the magazine is repairing, the powder be removed into Cravens Bastion and kept under guard.*

Despite many grievances and petitions, gunpowder was kept in the Old Powder Magazine until 1748, shortly after the dismantling of the walled city, when a new powder magazine was built further from the city population. During the American Revolution, the structure was again used

⁵¹ Thomas Cooper, *Statutes at Large of South Carolina, Volume VII*, (Columbia: A.S. Johnston, 1837), 30-31.

⁵² Nora Davis, *Public Powder Magazines at Charleston*, (Charleston: Historical Commission of Charleston SC, 1944), 190.

as a public magazine as the demand for gunpowder increased. Afterwards, the structure seemingly underwent another series of repairs when on March 3, 1780 a sum of £286 was paid “to Richard Peroneau for Boards & Carpenters work to repair the Magazine behind the old Church”⁵³.

When initially authorized, it was laid out by the Commons House that the structure would be built on private property and rented annually by the State “until the same shall be delivered into their possession”⁵⁴. As a new magazine was constructed further up the peninsula in 1748, the assumption is that the old magazine was “delivered” to its owners. Nineteenth century ownership remained in the Izard and Manigault family and was eventually passed to Dr. Gabriel Manigault (1833-1899) in the late 19th century. Over time, the interior was partitioned for a variety of purposes. The vacant building was described in the *News and Courier* on January 10, 1897, in unstable condition and “gradually falling to pieces”. Dr. Gabriel Manigault, an amateur architect, told the newspaper's editor that he felt the “time has almost come when it must be removed altogether”⁵⁵. The Old Powder Magazine was not demolished as Manigault felt it should be, but the structure has been plagued with conservation issues ever since. In 1902 the National Society of Colonial Dames in the State of South Carolina bought the historic structure (see figure 4.1), and in 1993 leased the building to the Historic Charleston Foundation for ten years. The Foundation undertook a complete renovation directed by architect Glenn Keyes with the help of Richard Marks Restorations, and returned the Old Powder Magazine to the Colonial Dames in 2003.

⁵³ Military Affairs, Accounts, A.A. Muller's *Memo Accounts from February 1779-May 1780*, (Columbia: Historical Commission of South Carolina).

⁵⁴ Davis, *Public Powder Magazines*, 193.

⁵⁵ “Old Powder Magazine”, Preservation Society of Charleston, Alfred Halsey Map Preservation Research Project, accessed 25 October 2011, <http://www.halseymap.com/Flash/window.asp?HMID=17>.



Figure 4.1: Old Powder Magazine at 79 Cumberland Street, Charleston, SC; ca. 1902. (Source: Library of Congress)

As it stands today the Powder Magazine is a low, square structure of stuccoed brick (stuccoed appearance not original, but dates to mid-18th century⁵⁶). The square magazine features a pyramidal roof and pairs of low brick gables breaking out on each of the four facades. The resulting irregular roofline is covered with a red, pantile roof tile replaced in 1996 by Richard Marks Restorations, Inc. The walls are approximately 3 feet thick constructed on solid masonry, typical of military construction. The east wall currently features two inoperable windows; the south wall an altered doorway; the west wall a large doorway currently gated with heavy wrought iron; and the north wall a modern double door leading into the courtyard. Most of the openings have been concealed on the interior by current museum exhibits. The west door of wrought iron is currently secured with an exterior glass door. Presently, the door on the south side is used as the primary entrance into the museum. The doorway adjoins a connecting hallway leading to the basement of the single house to the rear of the magazine. The interior of the magazine features four intersecting groin vaults arranged around a single central column. The exterior walls enclose the vault ends in between eight additional English bond piers.

⁵⁶ Davis, *Public Powder Magazines*, 192.

An Investigation of the Structural Iron

Visible on the interior are the two, full-length iron tie bars that run east-west connecting the roof gable pairs (see figures 4.2 and 4.3). The rectangular bars themselves are made of hand wrought iron, a material known and used for its tensile strength. The plates and bolts on the exterior that are used to secure the ends of the reinforcement bars are more than likely forged of wrought iron also. Later iron tie rods had plates made of cast iron, a material with greater compressive strength. The plate on the north-west gable and both plates on the eastern façade have a much more uniform visible appearance (see figure 4.4), and show evidence of several possible makers' marks. However, the plate on the south-west gable has a more uneven surface appearance, and shows evidence of severe deterioration (see figure 4.5). Perhaps this is an indication that the southwest exterior plate is an original, while the others are later replacements. However, the exact reasoning for the accelerated deterioration of this anchor plate over the others is hard to discern.

Often called 'earthquake bolts' in Charleston, these iron reinforcement rods were in fact incorporated into Charleston's buildings well before the great earthquake in 1886. The exact

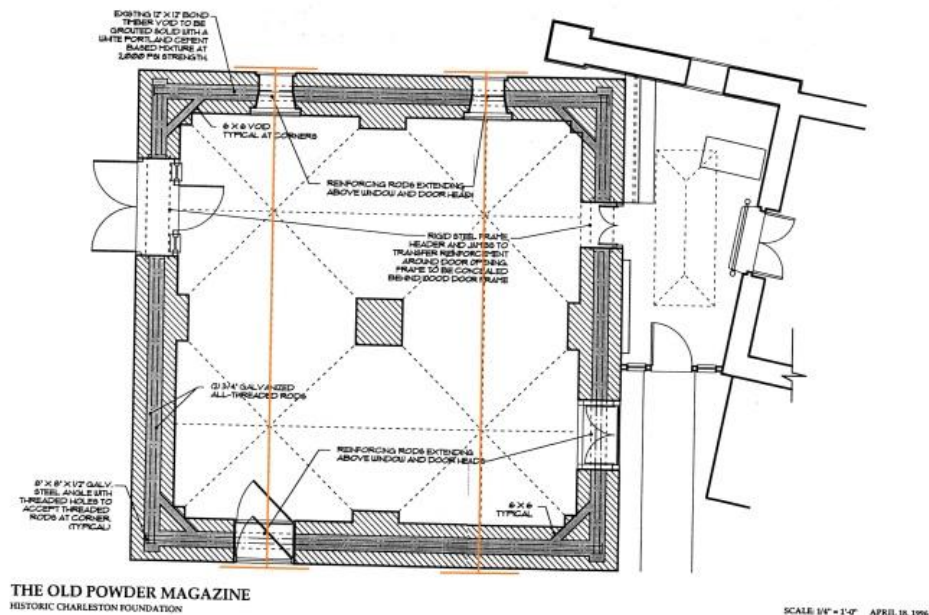


Figure 4.2: Floor Plan of the Old Powder Magazine with the location of the iron tie rods illustrated in orange. (Source: Historic Charleston Foundation)



Figure 4.3: Interior panorama showing the iron tie rods exposed in the powder magazine ceiling vault. *(Photo taken by author)*



Figure 4.4: Exterior anchor plate on north-west gable. *(Photo taken by author)*



Figure 4.5: Close up photograph of the anchor plate on the south-west gable showing extreme deterioration. *(Photo taken by author)*

date of the manufacture or installation of the iron tie bars at the Old Powder Magazine is not known, but one of the large cross-shaped anchor plates appears in the 1860 Harper’s Weekly engraving, one of the earliest images of the Powder Magazine (see figure 4.6). The bars, anchor plates and bolts all have a quality that could date from the 18th or early 19th century. They could, in fact, be the unidentified solution to a 1743 warning from the Commons House “that the present Powder Magazine in Charles Town has given way on every side and is cracked in several places ... it must be secured on every side”⁵⁷. The aforementioned stamps, possible maker’s marks (see figure 4.7), which are visible on the two eastern plates and on the north-western plate, could potentially provide a clue to the origin of the tie bars. There is no visible evidence of a stamp on the south-west exterior plate, the highly deteriorated plate.

Two additional observations were made of the Powder Magazine tie bars that may also provide interesting clues to dating the bars themselves. First, on the interior of the Powder Magazine, where the bar is exposed within the ceiling vault, there is slight evidence of the hand-wrought center connection. Later tie rods that were installed throughout Charleston in the 19th

⁵⁷ *Council Journal*, 22 February 1744, (Charleston: South Carolina Historical Society Archives).



Figure 4.6: November 1860 issue of Harper's Weekly illustrating the Old Powder Magazine in Charleston with exterior tie rods visible. (Source: *Harper's Weekly*. New York, NY. 3 November 1860)



Figure 4.7: Maker's marks stamped onto the iron anchor plate arms. Images were taken during the 1996 restoration. (Photo courtesy of Richard Marks Restoration, Inc.)

century were typically rounded rolled iron bars that were extruded through opposite masonry walls and then connected in the center via a turnbuckle or hook. Second, when visually inspecting the southwestern-most anchor plate, one can see that its attachment method is different as compared to the other three plates. When looking at the southwest exterior plate, the vertical arm is threaded onto the bar first. Then the horizontal arm is threaded and followed by the very square, box-like bolt. The other three bearing plates have their horizontal arms threaded first, followed by the vertical arm and then a pyramid-capped bolt (see figure 4.8). Although the northwestern-most plate appears to have a replacement bolt cap, if all four anchor plates are of the same time period and the same building campaign, it is hard to discern why the builders would have installed one anchor plate one way and the other three plates in a different manner. The manufacture and installation techniques of the Powder Magazine tie bars may be evidence of possible earlier methods, offering further support to the theory that the southwest exterior plate is of an earlier date than the other three. Other case studies of iron tie bars installed around the lowcountry in the 18th century need to be analyzed for further comparison.

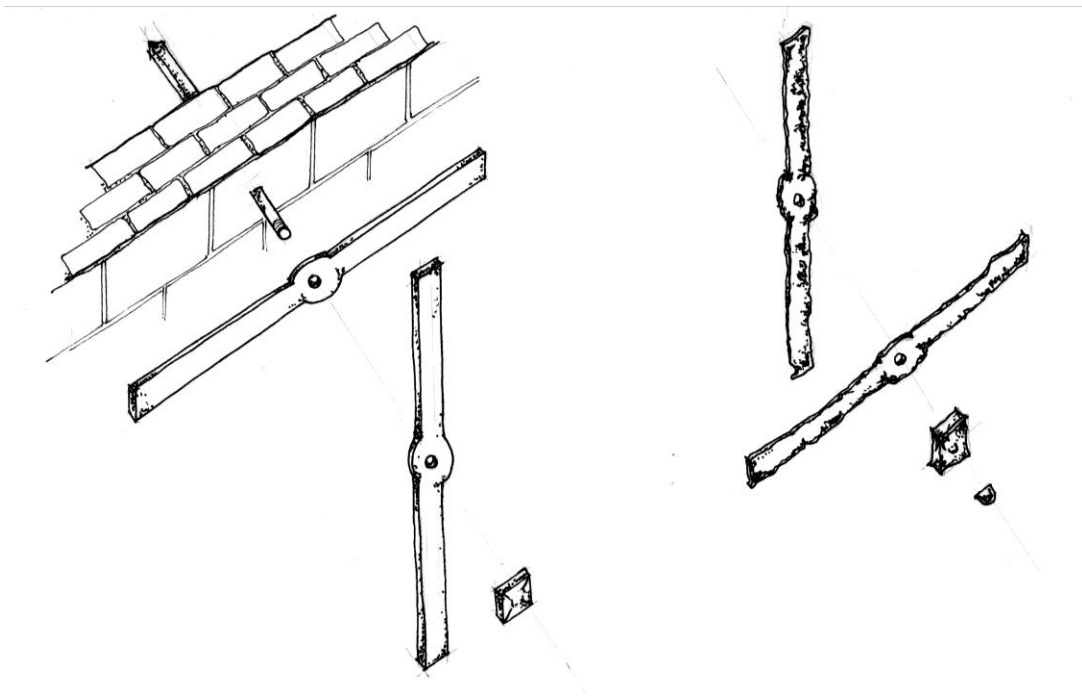


Figure 4.8: Attachment methods used on exterior anchor plates (*Sketch by author*)

In an attempt to learn more about the manufacture and origin of the Old Powder Magazine tie bars, a clay mold was taken of one of the exterior plate stamps on November 29, 2011 (see figure 4.9). In preparation for the mold, the area around the chosen stamp was first cleaned of all dirt, and loose paint was removed with a wire brush and metal scraper. Paint removal was attempted with a few different techniques. First, a coat of Smart Strip⁵⁸ paint and varnish remover was brushed onto the surface (see figure 4.10). The water-based, zero-VOC paste seemed to remove the top layer of paint, but did not remove all layers. Subsequently, a layer of Strypeeze⁵⁹ paint stripper was applied to the taped off surface. The semi-paste material was easy to apply to the vertical surface and penetrated the paint layers deeper, stripping the majority of the paint with the exception of within the smaller crevices. To remove the last traces



Figure 4.9: Taking a clay mold of the maker's mark on the upper arm of the south-east anchor plate. (Photo taken by student Elyse Harvey)

⁵⁸ Manufactured by Dumond Chemicals, Inc. New York, NY.

⁵⁹ Manufactured by Savogran Company. Norwood, MA.



Figure 4.10: After initial cleaning and brushing, a layer of Smart Strip is applied to the area to remove top layers of paint. *(Photo taken by author)*

of paint within the stamp area, a multi-temperature heat gun was used to heat the surface and the paint scraped using metal hand tools. Using Sargent Art natural-colored modeling clay⁶⁰, a total of three clay impressions were taken of the cleaned iron surface, and then transported to the Clemson/College of Charleston graduate conservation lab at 292 Meeting Street.

To date, there is very little publication on early iron maker's marks, especially in the pre-Revolutionary colonies. If the iron ties were in fact installed in the 1740's as a recommendation in the 1743 Council Journal, Charleston was still a colony under the proprietary rule of King George II of England. As a colonial military structure, the Old Powder Magazine was constructed under order from the King, and the anchor plate stamp could be an English mark. As it appears today, the stamp is a simple capital 'L' within a circle (see figure 4.11), and the nature of the stamps indicates that they were made while the bar was hot. Thus, the marking would have been done at the mill or workshop where the iron was processed⁶¹. However, the exact origin of

⁶⁰ Manufactured by Sargent Art, Inc. Hazleton, PA.

⁶¹ Ken Schwarz, email correspondence to author, 4 March 2012.

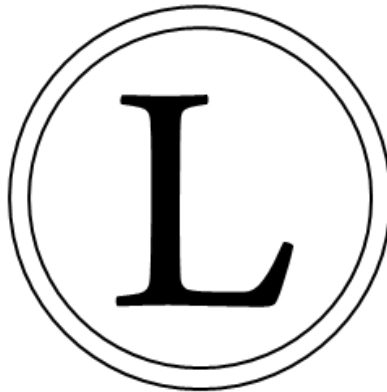


Figure 4.11: Recreation of the stamp applied to the Old Powder Magazine anchor plates (*Drawn by author*)

the 'L' is unknown. While the initial tendency is to assume the stamps are an individual's maker's marks, there is perhaps another story for these early examples of iron stamps. References in both the *Virginia Gazette* and the *New Hampshire Gazette* refer to iron bar being stamped with a mark to indicate that appropriate duties had been paid prior to shipping. In 1760, the following notice was printed in the *New Hampshire Gazette*:

“Taken up in the Town of Portsmouth
The 13th of November last near the
Swing Bridge, a BARR of IRON: The
Owner may have it again, by telling
the Marks, and paying the Charges.
Enquire of the Printer.”⁶²

These comments suggest that such marks were sometimes used in colonial America as an aid in identifying iron stock. There is also evidence that some bar stock was marked with a "rolling mark", which identifies the mill at which the bar was formed. This is also a possible determinant in the origin of the Old Powder Magazine's tie bars. Although, it would seem unusual to place multiple marks close together on the bar and in such random fashion as was done in this case⁶³.

Similar iron marks have been observed in other locations throughout the mid-Atlantic and New England regions. Specifically, marks have been discovered on some iron reinforcement

⁶² Notice, *New Hampshire Gazette*, 7 March 1760. Quoted in letter from Lee Nelson, "Follow up to the Query Regarding Marks on the Iron Stirrups of the Roof Trusses at Christ Church, Lancaster County, Virginia", *Association for Preservation Technology Bulletin*, Vol. VIII, No. 4 (1976), 120.

⁶³ Ken Schwarz, email correspondence to author, 4 March 2012.

at Christ Church in Lancaster County, Virginia. This colonial brick church, a modified Greek cross in plan, was built in 1732 from a bequest of Colonel Robert "King" Carter of Corotoman, and has survived remarkably complete. In the roof structure, a series of iron straps band the king posts and principal rafters to the lower chords of the roof trusses. It is on these iron straps that several different impressed marks can be seen. There are six distinct marks including some with Roman lettering, specifically the letter 'K', a double 'K', and 'RP'. Others incorporate symbols such as dots, parallel, and intersecting lines (see figure 4.12)⁶⁴. A number of similar marks have also been seen stamped near the ends of wrought iron bars used as fireplace lintels in New Hampshire colonial period houses⁶⁵.

Perhaps, though less likely, the lettered stamps at the Old Powder Magazine are an indication of the blacksmith's name who worked the iron into its final form before installation. If such is the case, there are two particular blacksmiths working in Charleston within the appropriate installation time frame of the Powder Magazine tie bars. Thomas Lovelace is the



Figure 4.12: "Horseshoe" iron mark at Christ Church, Lancaster County, Virginia (1732) (Source Carlton, "Marks on the Iron Stirrups, *APT*, Vol. VII No. 1)

⁶⁴ R.L. Carlton, "Marks on the Iron Stirrups of the Roof Trusses at Christ Church, Lancaster County, Virginia: A Query", *Association for Preservation Technology*, Vol. VII No. 1 (1975), 109-111.

⁶⁵ Letter from Lee Nelson, "Follow up to the Query Regarding Marks", *APT Bulletin* Vol. VIII, No. 4, 121.

first known blacksmith to be recorded in the *South Carolina Gazette*. On March 25, 1732 a notice was published offering his blacksmith tools for sale⁶⁶. Some years later, the ironworker James Linguard is also recorded in the *South Carolina Gazette*. In a 1753 advertisement, he is listed as a “Smith and farrier, makes all kinds of scroll work for grates and stair cases; ship, jack and lock work, and all other kinds of smith’s work at his shop”⁶⁷. It is apparent that decorative ironwork, as well as more common items, was produced at his shop. However, even if the bar iron used for structural applications would have been imported from England in the 18th century, it is possible it could have been worked by a local smith like Lovelace and Linguard.

Obviously, there is more to be learned about such marks, their origin, their identification, their purpose, and their presence in the United States. Various marks could conceivably identify not only iron made in specific locations, but dates of production as well. Even though much of colonial bar iron was provided by England, quantities of Swedish and Russian iron were being imported to England⁶⁸. Thus, some of these iron hallmarks may derive from the countries of northern Europe. If so, research on their precise origins and meanings may prove difficult. Further research in other colonial cities and perhaps from European archives may prove useful for comparison.

⁶⁶ Kelly Ann Ciociola, “Werner Fecit: Christopher Werner and Nineteenth Century Charleston Ironwork”, (Graduate Thesis, Clemson University/College of Charleston, 2010), 11.

⁶⁷ Advertisement, *South Carolina Gazette*, 21 May 1753. Quoted in Ciociola, “Werner Fecit”, 11-12.

⁶⁸ See Chapter 2 of this thesis for more information regarding the origins of colonial iron.

51 Meeting Street: Nathaniel Russell House

Site & Construction History

At the turn of the 18th century, Charleston experienced exponential growth and economic prosperity, quickly becoming one of the new nation's wealthiest cities. As a result, grand residences were built across the peninsula by wealthy merchants and businessmen like Nathaniel Russell. Having arrived in Charleston in 1738, Russell purchased Grand Model lot number 247, now known as 51 Meeting Street, fifty years later in 1779. At the time, a series of working class tenements stretched along the south side of the property, and in 1804 Russell advertised a "large and airy SCHOOL HOUSE, situated in Price's Alley near Meeting Street" for rent on the property⁶⁹. By 1808, the three-story grand brick townhouse was completed at 51 Meeting.

The unknown architect of the Nathaniel Russell house seems to have been inspired by the British architects of the late 18th and early 19th centuries with the use of a tripartite geometric plan, exterior balconies, extensive cast plaster, and composition ornament throughout the interior of the house. The floor plan of Russell's house includes a rectangular, an elliptical, and a square room on each floor. The house appears to conform partially to the form of a townhouse, but with a strong projecting four-sided bay that rises the full three stories on the south façade of the structure (see figure 4.13). The front facade features marble window lintels on the third story, bright red brick arches spanning the tops of the second story windows, and red brick jack arches above the first floor openings. A delicate, custom iron balcony runs fully around the front and southern facades. Centered in the front façade, an elaborate doorway with fanlight opens into a spacious receiving room with Russell's office off to the side (see figure 4.14). Separated from the reception room by glazed doors, the grand cantilevered staircase rises three stories without any visible means of support. A large Palladian window lights the stair hall on the lower flight, while a recessed elliptical gives light to the upper flight⁷⁰. After completion, the Nathaniel Russell house was considered one of the finest Federal-style residences in Charleston, and perception has changed little today.

⁶⁹ *Charleston Courier*, 9 June 1804. Quoted in *Grandeur Preserved: The House Museums of Historic Charleston Foundation*, Historic Charleston Foundation (Charleston: History Press, 2008), 16.

⁷⁰ *Grandeur Preserved*, 16-17.



Figure 4.13: Photograph of the Nathaniel Russell house taken in the late 19th century. (Source: Gibbes Museum of Art, George W. Johnson Photographs 1886-1930, AN1963.018.0105.03. Accessed 10 February 2012, www.lowcountrydigital.library.cofc.edu)



Figure 4.14: Front façade of the Nathaniel Russell house. (Photo taken by author)

Unintentional Alterations of 1811

Just a mere three years after the Nathaniel Russell house was completed, a tornado struck and devastated portions of downtown Charleston. On September 11, 1811, the *Charleston Courier* reported⁷¹:

“... it crossed over to Lynch’s Lane, where it unroofed several houses; from thence it proceeded across Church street ... to Meeting street, where several houses were unroofed, particularly the large new brick house of NATH. RUSSELL, Esq. ...”

In describing its level of destruction, the *Times* described the “Mansion-House ... together with his [Russell] extensive back buildings, entirely unroofed; the windows broken in, and his furniture, (for the most part) entirely ruined”⁷². From these grim damage reports, it appears that repairs were limited to small items like sash replacement and roof coverings, with one exception: the stabilization of the west (rear) wall. Many years after the tornado struck downtown, Nathaniel Russell’s grandson, recounted the disaster as he slept in the front bedchamber on the third floor. His description of actual damage to the house is brief, but he does provide one important detail, saying: “The house was unusually well built, but such was the violence of the wind that a rift was made in one of the walls which had to be secured by iron clamps ...”⁷³

⁷¹ *Charleston Courier*, 11 September 1811. Quoted in Ridout and Graham, Historic Structures Report for HCF, 17.

⁷² *The Times*, Charleston, 11-17 September 1811. Quoted in Ridout and Graham, Historic Structures Report for HCF, 17.

⁷³ Nathaniel Russell Middleton, “Reminisces”, Published in Alicia H. Middleton, *Life in Carolina and New England During the 19th Century*, 185.

Two iron plates set into the brickwork on the exterior west wall of the main house are the only readily visible evidence of a set of four iron straps that were installed after the tornado damage of 1811. The two visible plates belong to the lower pair of iron straps; the plates corresponding to the upper pair of tie rods appear to have been removed or popped away from the wall (see figure 4.15). The historic structures report completed for the Nathaniel Russell house in 1996 makes note of two areas of masonry failure at the top of the rear wall. The areas of bulged and broken brick aligned with the straps below, and led the investigative team to discover the pair of ties installed between the third floor and attic space⁷⁴. The portions of the wrought iron clamps that remain visible on the exterior today measure approximately 3-1/4 inches wide, and 30 inches long set in a vertical orientation. They are at least one inch thick and carefully cut into the brickwork to fit flush with the wall. There are no other fittings on the



Figure 4.15: Vertical Iron straps indicate the insertion of tie rods on the rear wall of the main house following the 1811 tornado. Masonry damage just below the cornice indicates the placement of additional tie rods between the third floor and attic. (*Photo taken by author*)

⁷⁴ Orlando Ridout and Willie Graham, *An Architectural and Historical Analysis of the Nathaniel Russell House, Charleston South Carolina, Vol. III*, A Historic Structures Report prepared for the Historic Charleston Foundation (Charleston, December 1996), 93.

outside face of this plate as is common on reinforcement plates from the mid-19th century and later⁷⁵.

Clear evidence survives of the early 19th century reinforcement within the northwest corner of the attic of the main house. Reinforcement was accomplished by installing the wrought iron tie bars: two between the second and third floors, and two between the third floor and attic. These early tie bars are of a design easily distinguished from later work associated with the 1886 earthquake, and are visible in the attic with some investigation. Each bar appears to have L-shaped clamped ends bedded into the top of the west wall, with one leg extending vertically down into the wall, the other continuing into the attic approximately nine feet. Here, the bars are set between the joists and project through a diagonal roof beam that supports the hip rafters of the roof. Threaded on their ends, each is secured with a nut to the beam (see figures 4.16 and 4.17). Presumably, the pair directly below was finished in a similar manner and extends approximately nine feet into the house at the third floor joist level⁷⁶ (see figure 4.18).



Figure 4.16: An 1811 tie rod installed to correct tornado damage projects through the attic floor in between joists. *(Photo taken by author)*

⁷⁵ Ridout and Graham, Historic Structures Report for HCF, 92.

⁷⁶ Ridout and Graham, Historic Structures Report for HCF, 17,92-93.



Figure 4.17: Tie rods installed in 1811 are threaded at their ends and secured to diagonal roof beams with bolts. *(Photo taken by author)*

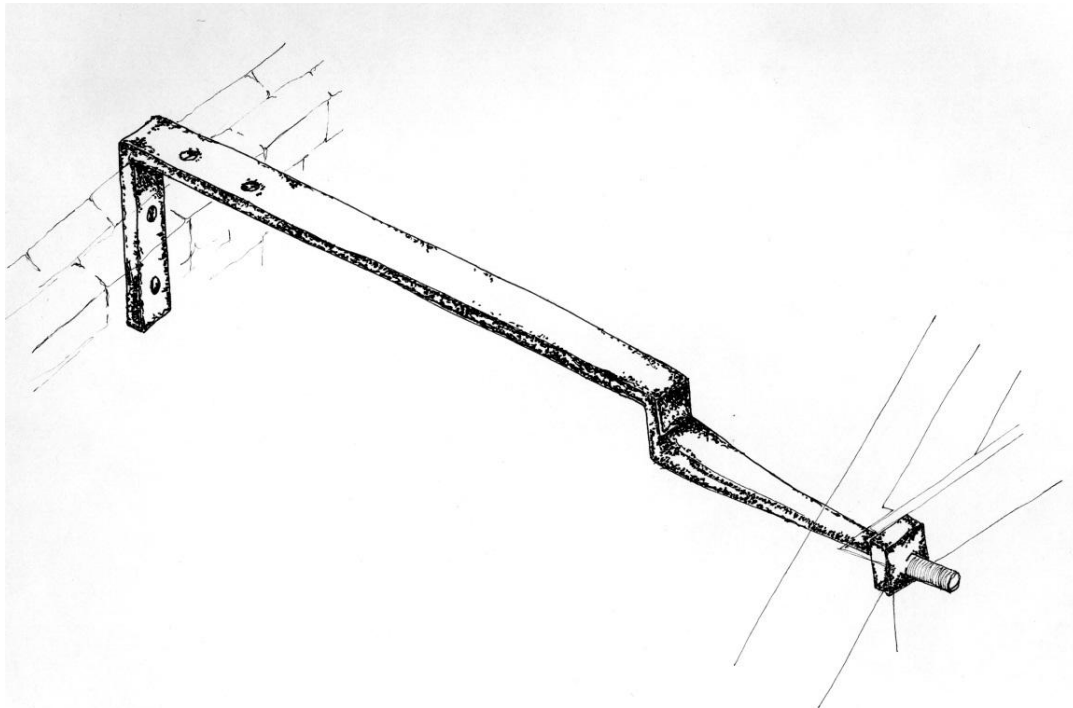


Figure 4.18: Sketch: Iron straps installed in rear wall, circa 1811. *(Sketch by author, not to scale)*

Devastation in 1886

A second, and much more extensive, episode of reinforcement was required following the disastrous earthquake of 1886. The damage assessment that was conducted city wide noted that the east and west walls were badly cracked over the openings and the house was “badly sprung and separated”. The damages at the Russell house and its dependencies were approximated to be \$2500⁷⁷. Damage to the stable and hyphen was evidently severe. The north wall of the hyphen was completely torn down and rebuilt along with the upper west gable of the kitchen house. However, at the stable damages were allowed to go without repair and remediation was deferred. Eventually the structure was torn down around 1890⁷⁸. At the main house, earthquake damages required reinforcement of all four of the load bearing masonry walls. Iron tie rods and anchor plates were inserted into the building at the second and third floors, just below finish floor level. The front façade was reinforced by inserting two tie rods on each floor level; the north and south elevations are supported with four tie rods at the second floor and five at the third floor (see figure 4.19).

According to the Historic Structures Report conducted by Ridout and Graham, the tie rods that reinforce the front façade extend only a few feet into the house. Sub-floor investigations for the report revealed that the circular tie rods are modified into rectangular section bars on the interior end. Although the end connection was not accessible during the investigation, it is probable that the bar is bent over an interior joist and nailed or bolted into place (see figure 4.20). A similar detail can be found on a vertical iron tie rod discovered in the attic of the Nathaniel Russell house, used to strengthen a roof truss. This strap is applied against a plaster and lath partition that probably dates to the 1870s; thus, the iron strap is more than likely part of the 1886 earthquake repairs as well⁷⁹.

The reinforcing rods that tie the north exterior wall (driveway side) to the south of the house (garden side) run the full width of the house and are positioned to either side of the extended polygonal bay. This is designed to allow a clear path through the house without interference from the projecting bay and central stairwell inside the house. Running parallel to the front façade, the exterior ends are threaded to receive exterior anchor plates to distribute

⁷⁷ *Record of Earthquake Damages*, 1886. On microfiche, Historic Charleston Foundation Archives.

⁷⁸ Ridout and Graham, Historic Structures Report for HCF, 39-40.

⁷⁹ Ridout and Graham, Historic Structures Report for HCF, 93-94.

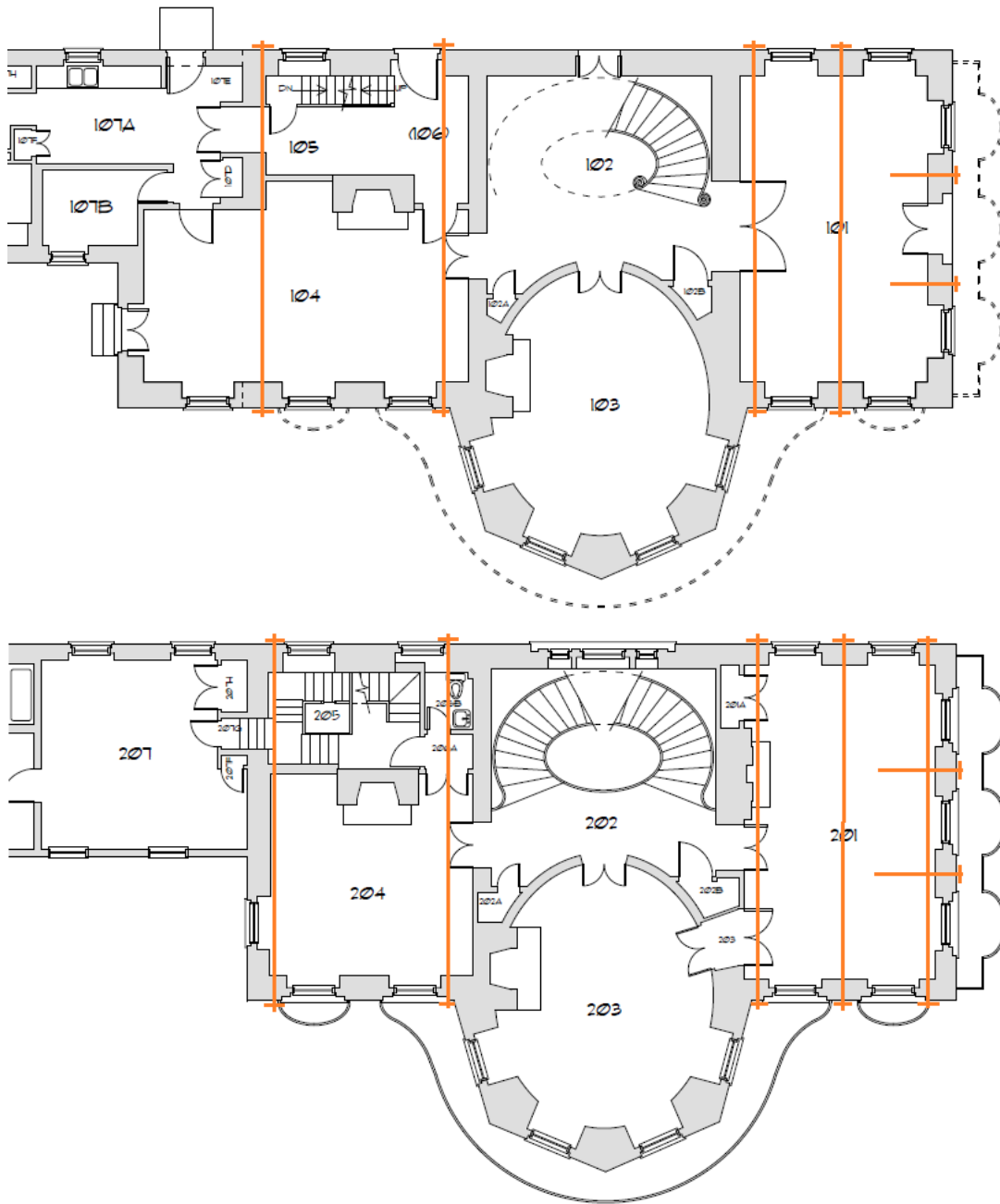


Figure 4.19: After the 1886 earthquake, the original portion of the Nathaniel Russell house had to be reinforced using full length tie rods to secure the north & south walls, and short wall anchors to secure the east (front) wall; Top: First floor plan & reinforcement diagram, Bottom: Second floor plan and reinforcement diagram. (Drawings courtesy of Historic Charleston Foundation, drawn by Glenn Keyes Architects; diagramming by author)

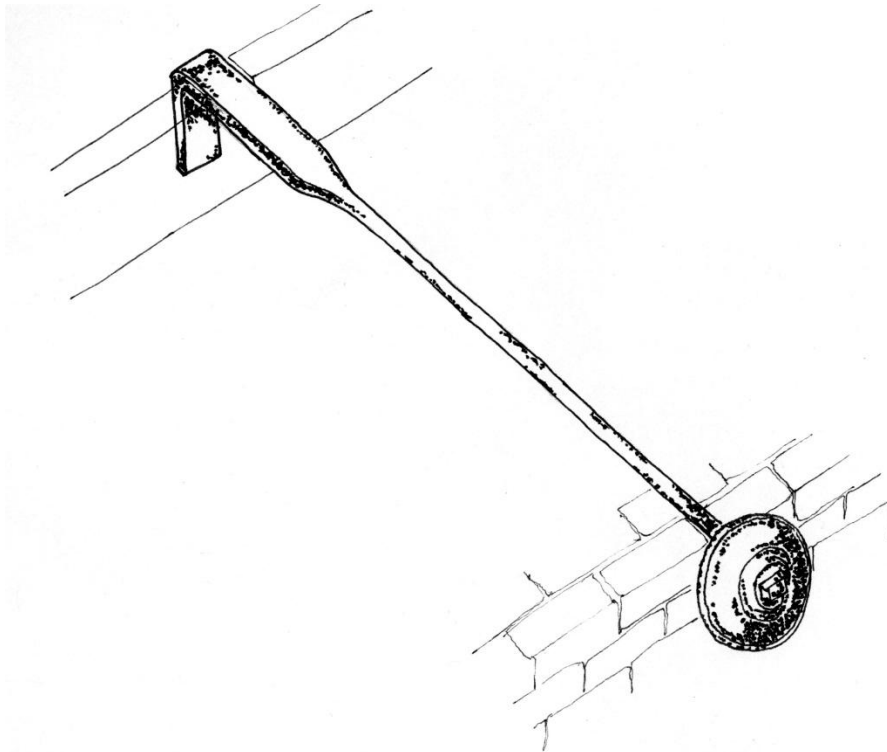


Figure 4.20: Sketch: Tie rods installed in front façade, circa 1886. *(Sketch by author, not to scale)*

the load over the exterior brick wall surface. Each tie rod is joined near the mid-point and tightened by a large turnbuckle (see figure 4.21). This feature was examined by Ridout and Graham just a few inches below the floor boards in the chamber above the withdrawing room by lifting the flooring near the center of the room⁸⁰.

The Nathaniel Russell house tie rods feature a number of cast-iron anchor plates on the exterior facades. In fact, there are 22 total plates in three varieties: circular plates with a plain, non-decorative profile, circular plates with a molded profile, and large, three-dimensional cross-shaped plates that are cast rather than cut from bar stock (see figure 4.22). There is little pattern to the location of each type of plate, and the method of attachment is the same for all three types. Therefore, it is believed that all three plate varieties are from the same period of installation. However, there does appear to be some effort to use the circular molded profile in the more visible areas of the house, with the plain round plates as a second alternative. These two types are the predominant forms in the late 19th century during earthquake repairs

⁸⁰ Ridout and Graham, Historic Structures Report for HCF, 94.

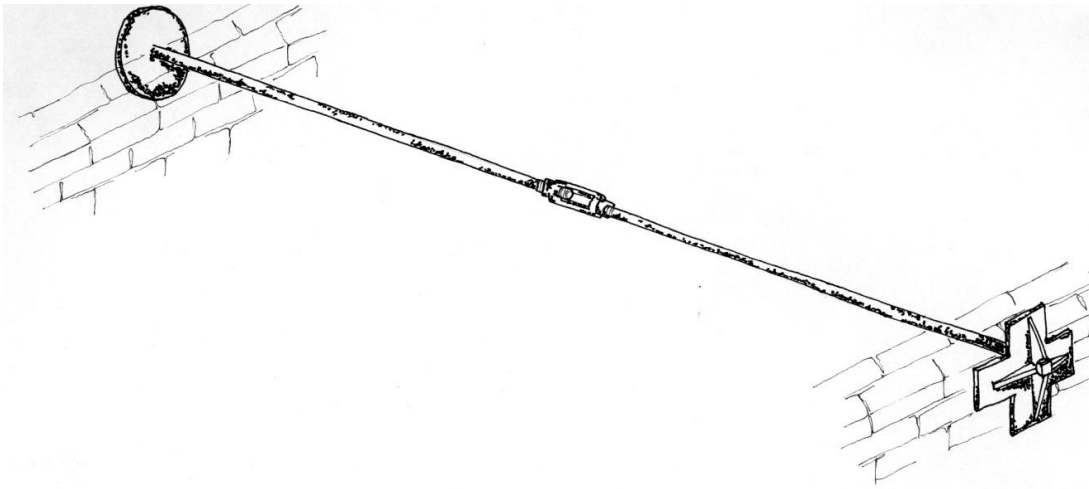


Figure 4.21: Sketch: Tie rods installed wall-to-wall to secure the north and south walls, circa 1886.
(Sketch by author, not to scale)



Figure 4.22: The Nathaniel Russell house features three different tie rod anchor plate forms: Top Left: Plain, circular anchor plate on the front façade. Bottom Left: Circular anchor plate with molded profile on the front façade. Right: Cruciform shaped anchor plates on the north façade.
(Photos taken by author)

throughout the city. Use of the cruciform plates is restricted to the north façade. In the Historic Structures Report compiled by Ridout and Graham, it is stated that a search revealed no comparables for the cast cruciform model⁸¹. However, a visual search conducted during research for this thesis revealed that the same cross-shaped plates were indeed used at 57 East Bay Street, which was also badly wrecked during the earthquake with an estimated \$2,479 in damages incurred to the structure⁸² (see figure 4.23).



Figure 4.23: Left: Cruciform shaped anchor plates installed at 57 East Bay Street; Top: Anchor plate detail (*Photos taken by author*)

⁸¹ Ridout and Graham, Historic Structures Report for HCF, 94.

⁸² *Record of Earthquake Damages*, 1886. On microfiche, Historic Charleston Foundation Archives.

College of Charleston: Randolph Hall

Site & Construction History

Founded in 1770, the College of Charleston is the oldest educational institution south of Virginia, and the 13th oldest in the United States. On March 19, 1785, the College of Charleston was chartered to "encourage and institute youth in the several branches of liberal education"⁸³. The first classes were held on the ground floor of Reverend Smith's home on Glebe Street (now the residence for College of Charleston presidents). Later, rooms for the College were fashioned out of old military barracks located on public land that is now the Cistern Yard. By 1824, the College offered a curriculum broad enough to regularly grant degrees, and during Reverend Jasper Adams' tenure as president, he reorganized the College and orchestrated the construction of the first building specifically designed for teaching⁸⁴.

In 1827, the architect William Strickland of Philadelphia prepared plans for the College of Charleston's first new building, known simply as the Main Building until the 1970s when it was renamed to honor College President Harrison Randolph⁸⁵. The new building was the first 19th century public building designed by an outside architect in Charleston. As a student of Benjamin Latrobe and an architect of the United States Capitol, Strickland also designed both the Second Bank of the United States and the personal residence of Langdon Cheves in Philadelphia - former College of Charleston trustee and President of the Second Bank of the United States⁸⁶. Construction of the Main Building began in 1828 and was virtually completed by March 1829. As designed, William Strickland's building is a simple rectangular, two-story brick structure over an elevated basement, with a pedimented three-bay-wide projecting central pavilion on the south (primary) façade and gable ends on the east/west sides (see figure 4.24). This original structure now forms the center section of the building on campus today. In 1840,

⁸³ J.H. Easterby, *A History of the College of Charleston, Founded 1770*, (Charleston: College of Charleston, 1935).

⁸⁴ Easterby, *A History of the College of Charleston*.

⁸⁵ "College of Charleston Complex: Main Building, Library and Gate Lodge", (National Register of Historic Places Inventory-Nomination Form prepared by the National Park Service, Washington D.C., 20 February 1972).

⁸⁶ Kenneth Severens, *Charleston Antebellum Architecture and Civic Destiny*, (Knoxville: University of Tennessee Press, 1988), 56. It is thought that Robert Mills, also a student of Latrobe, may have been consulted on the project, as the building is similar to Mills' Fireproof Building with simple Greek detailing. However, it is unknown why Mills was not selected as the architect.



Figure 4.24: Architect William Strickland's sketch of the Main Building. This is the view of Randolph Hall when originally built, before the addition of wings and porticos. (Source: *Low Country Digital Library*; <<http://lowcountrydigital.library.cofc.edu>>)

the Main Building received its first coat of roughcast stucco in an effort to complete the building according to Strickland's original plan, thus establishing a precedent that was followed in all later construction⁸⁷. No documentary evidence exists to suggest that any reinforcement rods were installed as a part of the original building plans.

Early Structural Concerns & Remediation

In 1849, the Mayor of Charleston complained of the "desolate" appearance of the campus buildings that were surrounded by a "gloomy and repulsive brick wall"⁸⁸. City Council agreed, and the College Board of Trustees appointed a new committee for the purpose of improvements to the "College Edifice and premises". Among those appointed was Colonel Edward B. White, who would furnish the design concept and drafts for a new South portico and East and West wing additions⁸⁹. White's design added the large two-story brick and stucco wings and the present grand colossal portico, with six giant Roman Ionic pillars and arcaded basement, to the center of the primary façade (see figures 4.25 and 4.26). White thereby changed Strickland's simple and utilitarian design to the existing more elaborate Roman Revival mode⁹⁰.

⁸⁷ National Register Nomination, 2-20-1972.

⁸⁸ Robert Stockton, "Campus Features Architectural Blend", *News & Courier*, 19 March 1973.

⁸⁹ *College of Charleston Board of Trustees Minutes 1785-1894*, (Charleston: College of Charleston Library, Special Collections), 12 November 1849. White's plans also called for the replacement of the brick perimeter wall with the present-day iron fence.

⁹⁰ National Register Nomination, 2-20-1972.



Figure 4.25: Photograph of the front portico and stairs of Randolph Hall, designed by Edward Brickell White and erected circa 1850. (Source: *Low Country Digital Library*; <<http://lowcountrydigital.library.cofc.edu>>)

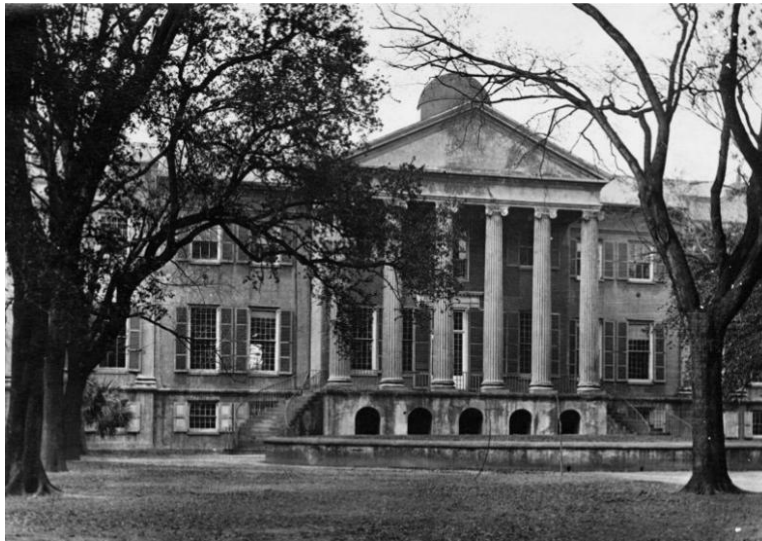


Figure 4.26: Front view of Randolph Hall and the cistern (constructed in 1857). (Source: *Low Country Digital Library*; <<http://lowcountrydigital.library.cofc.edu>>)

Some claim that the first iron reinforcement was inserted into the building during this period of additions and renovations. In February 1851, a delay is noted in the completing of repairs and additions. The exposed condition of the college is a concern, and the faculty requests the immediate attention of the Board of Trustees to the issue. The reason of concern was due to the contractor, William Jones, having removed the masonry pediments from the front and sides of the existing building, in order to facilitate the construction of the front portico and the flanking wings and to continue with roof repairs. Jones had begun on July 30, 1850, with the removal of the eastern pediment, and proceeded to remove the western and southern pediments⁹¹. During this time, the iron tie bars that run north-south within the roof joists of the Main Building could have easily been installed while the structure was open (see figure 4.27). This claim cannot be verified as fact based on existing documentary evidence. However, the form of the iron ties is consistent with earlier methods of reinforcement in the 19th century.

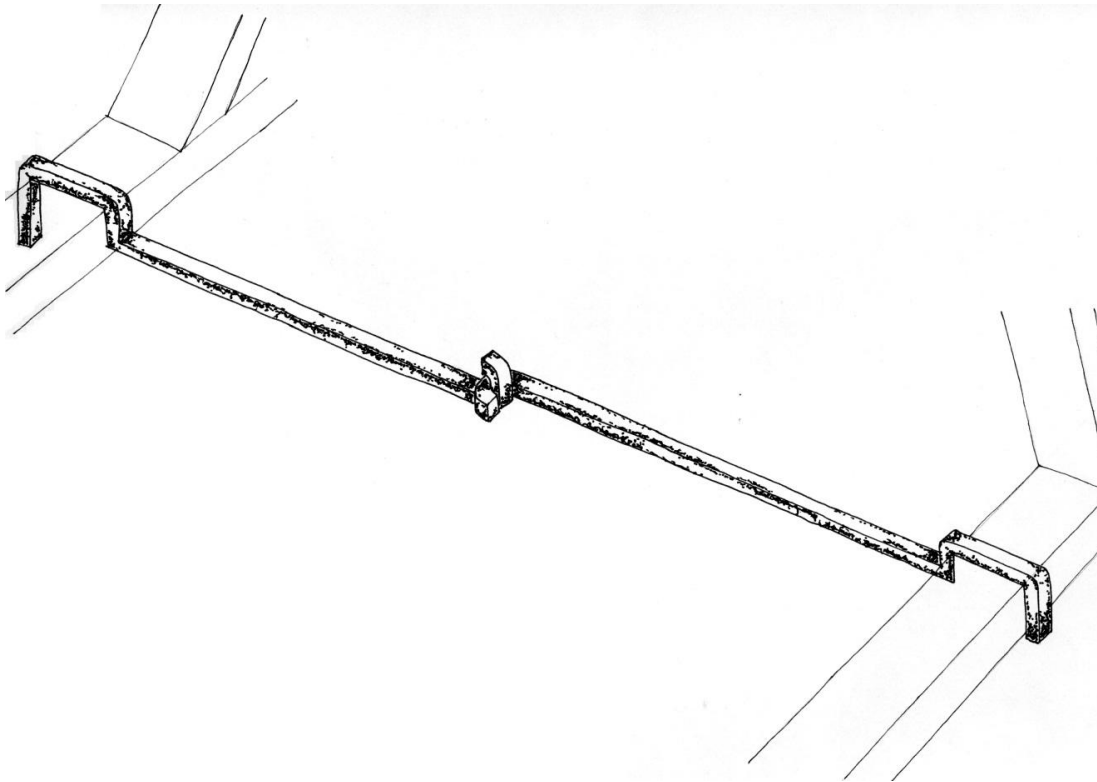


Figure 4.27: Sketch: Iron tie rods installed in the roof structure of the Main Building securing the north and south walls. (Sketch by author, not to scale)

⁹¹ Cummings & McCrady Inc., et. Al., *Conservation Master Plan: Randolph Hall and Porter's Lodge Renovation, Volume 2*, Report prepared for the College of Charleston, 16 February 2007, 15.

Concealed within the roof structure of the original portion of Randolph Hall, and accessible in the attic, are one pair of large wrought iron tie bars (see figure 4.28). Square in section, both bars run the full depth of the building (north to south) and are positioned to either side of the building's centerline. Unlike other iron tie rods that penetrate the outer masonry walls and are threaded to receive anchor plates, the exterior ends of the bars at Randolph Hall are strap-like and cramp down over the top of the ceiling plate timbers (see figure 4.29) to essentially pull the outer walls inward, much like the earliest tie bars in the Nathaniel Russell house that date to 1811. The extended length of these cramped L-shaped ends was indeterminable during investigation. Each tie bar is fashioned into a large hook near the mid-point, providing the connection of the north wall to the south wall (see figure 4.30).

In the decade leading up to the earthquake of 1886, the recently expanded Main Building was not without its structural issues. In 1871, College president Nathaniel Russell Middleton first comments on the apparent cracking in between the main building and its

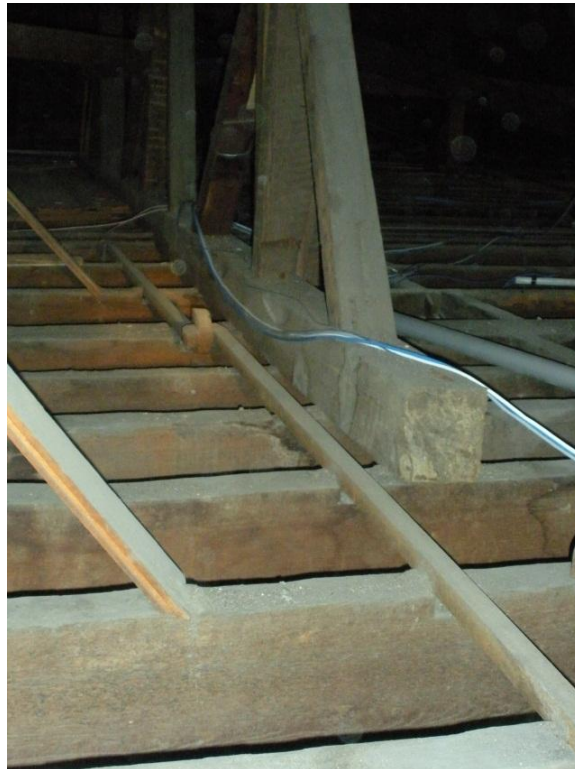


Figure 4.28: Iron tie rods installed at Randolph Hall, presumably in the mid-19th century during renovations, are cut into the roof joists and hook together in the center. (Source: Photo taken by author)



Figure 4.29: End attachment detail of the tie rod where the rectangular rod clamps over the sill plate within the roof structure. (Source: Photo taken by author)



Figure 4.30: Center hook connection. (Source: Photo taken by author)

flanking wings. Middleton comments that after repairs, the walls are “as broken and upheaved as before!” He questions whether this is due to “an opening out of the walls caused by extreme thinness in comparison with those of the main building”⁹². He, thus, declares the wings to be in unsafe condition. Additional repairs were necessary after the hurricane of August 1885. The Board of Trustees, in October, ordered that several bills for the repairs be paid, as it was “understood that the repairs were actually necessary for the preservation of the buildings”. Dr. Manigault was also placed in charge of rebuilding a portion of the north wall of the Main Building in 1885, “which had sunk from defective foundation”⁹³. The following June, in the summer of 1886, the sinking of the foundation and cracking within the wings is again referenced in a report from Professor Gibbes⁹⁴.

Devastation in 1886

Then, on the hot summer night of August 31, 1886, a devastating earthquake shook Charleston and the college campus:

... the night of the Earthquake shock, when the City ... was thrown into confusion and a dreadful scene of distinction was inaugurated. The College was greatly injured, the porch opening on the Campus was thrown down in several places, and the East and West wings split and toppled over, so as to necessitate some action of the Board with a view to making arrangements for opening the College in October and doing some repairs to the building⁹⁵.

One month after the disaster, Dr. Manigault gave his report on the condition of the Main Building:

The basement of the Central Building uninjured: On the second floor, the President’s room and Prof Sachtlebens room uninjured. On same floor the East and West walls of the Chapel uninjured. The South wall uninjured, the North wall leaning out the distance of a half inch at junction with the ceiling. The plaster of the ceiling fallen out near the East chimney and much cracked throughout. On third floor of Central building, the walls of main room of the Museum over the Chapel (North and South) leaning out, especially the South wall, which leans out over 4 inches at the junction with the ceiling. The East and West walls appear sound, and the 2 smaller rooms each side, although somewhat injured, could be strengthened by iron rods, which also secure walls of staircases.

⁹² *Board of Trustee Minutes*, 4 November 1871.

⁹³ Cummings & McCrady Inc., *Conservation Master Plan*, 31.

⁹⁴ *Board of Trustee Minutes*, 22 June 1886.

⁹⁵ *Board of Trustee Minutes*, 31 August 1886.

The two wings of building are much damaged including the ceiling, walls and roof. They are in a dangerous condition, and it will be impossible to repair them. The entire portico is also unsafe, including the stone slabs which sustain the paved floor and the stones of the steps. The whole will require to be taken down ... The floor of the main room of the Museum had settled before the Earthquake, and the repairs to the building should include the straightening of the floor⁹⁶.

Dr. Manigault's report to the Board of Trustees was accompanied by two 'diagrams', which unfortunately have not been located⁹⁷.

Later that September, members of the Executive Relief Committee appointed to inspect the College recommend that the portico along with six of its pillars be removed. They also recommend that the North and South walls of the Museum, then located on the third floor of the Main Building, be taken down to the ceiling of the Chapel below and rebuilt. Both of the East and West wings designed by Colonel E.B. White were badly damaged in the 1886 earthquake (see figures 4.31 and 4.32), and had to be demolished as recommended. The condition of the South wall is also apparent in this photo taken just after the earthquake. Robert McCarrell, a



Figure 4.31: South façade of Randolph Hall after the Charleston Earthquake of 1886. Damage to east wing is visible on right. Both wings were eventually removed and reconstructed. (Source: Low Country Digital Library; <<http://lowcountrydigital.library.cofc.edu>>)

⁹⁶ *Board of Trustee Minutes*, 11 September 1886.

⁹⁷ Cummings & McCrady Inc., *Conservation Master Plan*, 32.



Figure 4.32: Earthquake damaged south façade of Randolph Hall. Damage to east wing is visible on right and evidence of the portico pulling away can be seen on the left. (Source: Low Country Digital Library; <<http://lowcountrydigital.library.cofc.edu>>)

local builder, was hired to carry out all of the recommended repairs on the Main Building as soon as possible⁹⁸.

As the selected contractor, Robert McCarrell is authorized to restore the steps, platform, columns, beams and railings; bolting and securing the portico is also necessary. It is noted that McCarrell does “special work” and he secures the North wall of the College with bolts, instead of requiring its removal⁹⁹. In fact, the following is an itemized list of repairs costs from the account on the Standing & Special Committees on the repairs to the College buildings from 1886 to 1887, which specifically lists iron for reinforcement and to whom the work was given:

October 16th 1886 to April 16th 1887

To paid Robt. McCarrell for amts of contracts to take down wings of College Building and repairs \$3800. Repairing portico, bolting, and securing same \$800. Restoring stone steps, platform and railings of portico, with beams and uprights, artificial stone pavement under platform, all as per contract \$540: also extra charge for 8 iron columns in the chapel and Museum at \$11 each, being of larger size than contract \$88. Total amt of bills of Contract 5228,00
From which deduct discount on contract for N. Wall not taken down

⁹⁸ Board of Trustee Minutes, September 1886.

⁹⁹ Board of Trustee Minutes, 3 February 1887.

	\$200
For old brick and slate sold McC	256
For 4 anchors not inserted in wall	20 = 476.50

1886

Decr 18 Paid F.J. Ortmann for iron bolts for Library Building and Janitors house & putting the same into the walls 55.00

Decr 21 J.H. Steinmeyer for lumber Octr 23, 1886 29.81

1887

Feb 7 W.J. Wallace repairing roof & library chimney 13.75

March 2 J.W. Bliss repairing gutters as per agreement 25.00

March 19 A. OConnell painting exterior beams of ceiling 123.50

March 25 Stiles and Waters cleaning off & Kalsomining walls 85.00

March 26 E.R. Rutledge, Architect, for professional services 75.00

April 16 O.S. Miscally Plumber & Gas fitten bill 39.96

May 14 C.W. Stiles patching and coloring walls 40.00

May 28 Dr. Manigault's carpenters work in museum 16.50

June 7 D.A. Walker for brown stone coping and leveling N. wall cutting holes for iron railing etc. 80.00

June 25 Elisha T. Jenkins locks for Museum 15.20

" Dr. Manigault work by carpenters 24.75

June 28 W.M. Bird & Co., paints, oil, glazing, etc. for repairs since Earthquake 61.66

July 28 J.H. Steinmeyer, for lumber fencing etc. and props to the library 25.48

" F.J. Ortmann ... for repairing and putting up iron railing to North wall 45.00¹⁰⁰

On January 6, 1887, the Special Committee recommended that \$200 could be saved on McCarrell's contract, "by retaining the present North-wall of the Main Building, instead of taking it down – it being now satisfactorily secured by bolts"¹⁰¹. Indeed the north wall was not taken down, and the contract above reflects that deduction. Other smaller outlays included the payment of \$55.00 to F.J. Ortmann, blacksmith, for the reinforcement rods installed to secure the north wall of the Main Building, as well as the walls of the Porter's Lodge and Library buildings.

¹⁰⁰ *Board of Trustees Minutes*, 30 July 1887.

¹⁰¹ Cummings & McCrady Inc., *Conservation Master Plan*, 34.



Figure 4.33: South wall of the Main Building, east of the portico showing two earthquake bolts installed between the first and second floors. Detail: Circular patterned plate with decorative, moulded profile. *(Photos taken by author)*

From both the interior and exterior, there is no visible evidence of reinforcement on the north wall of the Main Building portion of Randolph Hall. Presumably, these 'earthquake bolts' were installed and then covered by a new application of stucco during repairs. Although nothing can be seen on the north wall, on the south wall there are two circular anchor plates to either side of the portico positioned in between the first and second floor (see figure 4.33). Whether or not these reinforcement rods tie the south wall all the way into the north wall cannot be determined; however, they were certainly installed post-1886 as they appear to be missing from the photographs taken directly after the earthquake.

Twentieth Century Alterations and Stabilization Efforts

The Museum of the College, having been located on the third floor of Randolph Hall in since 1852, was moved out of the building in 1907 to the new Thomson Auditorium in Cannon Park. This upper floor within Randolph Hall is now used for offices and administration with most

of the ceilings lowered for modern systems. While some of the pressed-tin ceiling tiles and skylights are still installed, the dropped ceilings obscure this period of the buildings history. The Alumni Memorial Entrance was added to the east end of Randolph Hall in 1928, and an additional extension for the west wing was built one year later. The design was completed by Albert Simons and Samuel Lapham, and is such that from a street view it appears to be original to the building with matching stucco, shutter styles and color, window proportions and placement. In the mid-1970s, restoration began across the College, including the now renamed Randolph Hall. This restoration effort was once again designed by the Charleston firm of Simons, Lapham, Mitchell and Small. At this time, the north portico was added to Randolph Hall to improve the flat appearance of the original Strickland design¹⁰². The addition is devoid of any windows except for the Palladian tri-partite window with no shutters on the second floor and a glass storefront on the ground entrance. A shallow curved piazza protrudes from the second floor with an arched arcade loggia supporting it on the ground level.

Being a building of age in continuous use, there is a constant need for repairs and restoration at Randolph Hall – the most recent having been completed just within the past 5 years. In 2006, the College of Charleston contracted with the architecture firm Cummings & McCrady to examine the stucco on the facades of all three buildings on the cistern yard. In turn, Cummings & McCrady hired the structural engineering firm 4SE Inc. to examine the structural integrity of these facades as a part of their report¹⁰³.

As surveyed by structural engineer Craig Bennett with 4SE, the following observations and recommendations were made in regards to additional reinforcement of the Randolph Hall¹⁰⁴:

1. Observation: Cracking in the entablature is indicative of minor movement of the pediment southward, away from the main building.

Recommendation: Tie portico pediment back to the main building so that it cannot move independently of the building. A conceptual scheme is illustrated in Figure 4.34.

¹⁰² Cummings & McCrady Inc., *Conservation Master Plan*, 4.

¹⁰³ 4SE's work was limited to the examination of the facades and was not to include structural or seismic evaluations of the buildings as a whole. Cummings & McCrady Inc., *Conservation Master Plan*, 2.

¹⁰⁴ Cummings & McCrady Inc., *Conservation Master Plan*, 2-4.

2. Observation: The vertical cracking between windows is believed to be the result of overall building movement. Such movement would include thermal expansion and contraction, movement due to differential settlement within the length of the building and movement due to lateral (primarily wind) loading.

The single crack on the north side is particularly indicative of overall minor settlement of the west end block.

Recommendation: Tie the walls of the older brick masonry sections of the building back to the floor diaphragms and longitudinally to increase the lateral load carrying capacity (i.e. seismic resistance) of the building and reduce cracking in the façade.

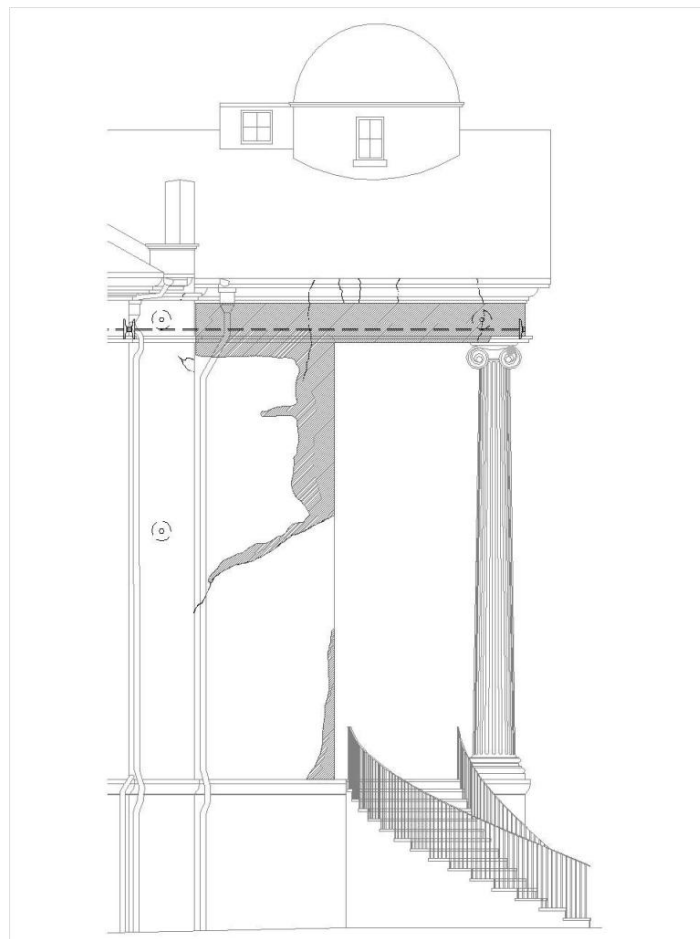


Figure 4.34: A conceptual scheme designed by structural engineers at 4SE, Inc. for securing the portico back to the main building (Source: *Cummings & McCrady, Conservation Master Plan, 19*)



Figure 4.35: Portions of the brick portico gable at Randolph Hall , originally damaged in the 1886 earthquake, showing signs of additional movement. *(Photograph provided by 4SE, Inc.)*

Following the report’s submittal, the College of Charleston did commence with the portico reinforcement, but postponed the additional reinforcement of the main structure as recommended following the second observation. However, it is the intention of the College of Charleston to move forward with the structural reinforcement as finances and priority of other campus projects allows¹⁰⁵.

The restoration and structural repairs of the Randolph Hall portico began, and the original recommendation to tie the portico back to the south wall of the main structure was carried out using two hot-dipped galvanized steel rods inserted into the masonry on both the east and west ends of the portico. In addition to the need to tie the portico back to the main building, it was also discovered that the portico required some additional reinforcement. In damage initially created by the 1886 earthquake, some of the brickwork in the portico gable along the roofline had begun to separate and slip downward along the roof pitch (see figure 4.35). Subsequent movement was again detected by 4SE engineers, and added reinforcement was called for. Two reinforcement tie rods were designed by 4SE and installed behind the entablature of the portico. Interestingly, the 1” hot-dipped galvanized steel rods penetrate the side walls on either side of the portico and are secured by a single, custom-designed anchored

¹⁰⁵ Notes from a meeting at 4SE with Craig Bennett, Hillary King and the author. 2 March 2012.

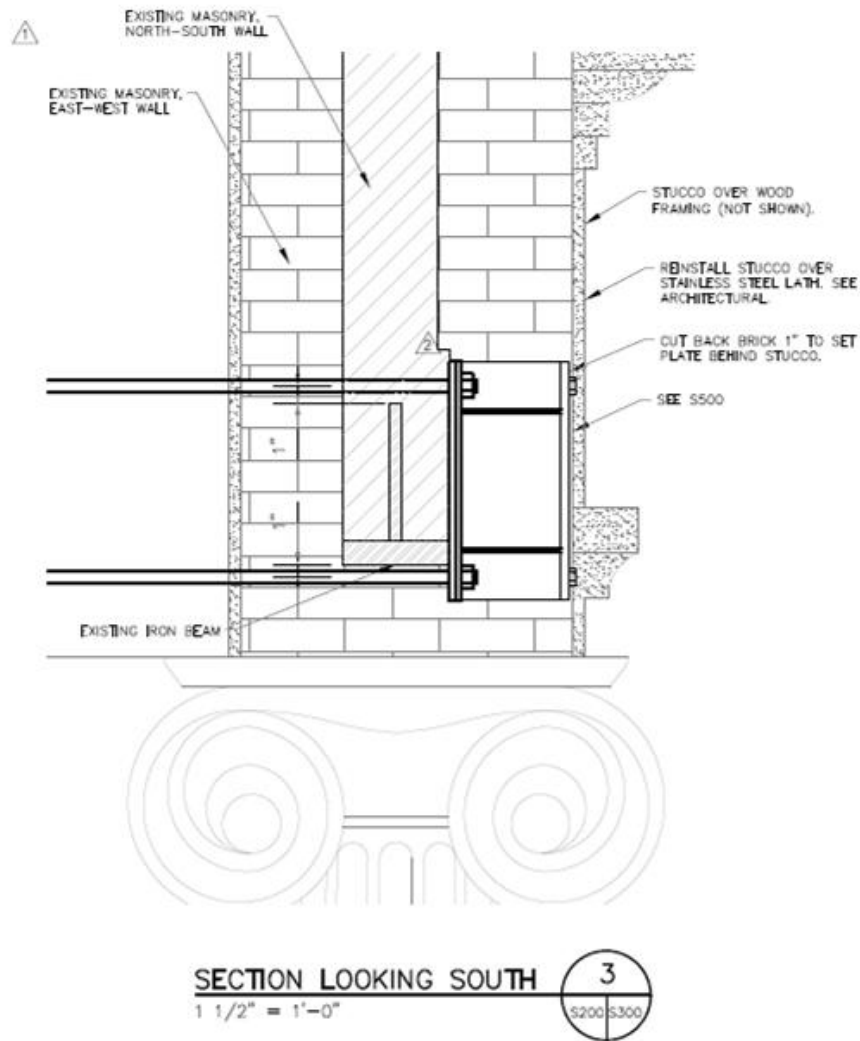


Figure 4.36: Section drawing of tie rods securing the side walls of the Randolph Hall portico. (Drawings provided by 4SE, Inc.)

plate. The section drawing shown in Figure 4.36 shows the attachment method of these tie rods as if cutting through the portico and looking south across the Cistern Yard.

During the restoration, even further reinforcement requirements were determined for the entablature to secure the brownstone slabs that span between the columns. Unlike the horizontal reinforcement that has been addressed in all previous instances, the brownstone would require vertical tie rods to prevent the masonry from sagging further and potentially falling to the ground below. To prevent this from happening, 4SE designed a reinforcement method that involves a 1-1/8" diameter stainless steel reinforcement rod inserted vertically in

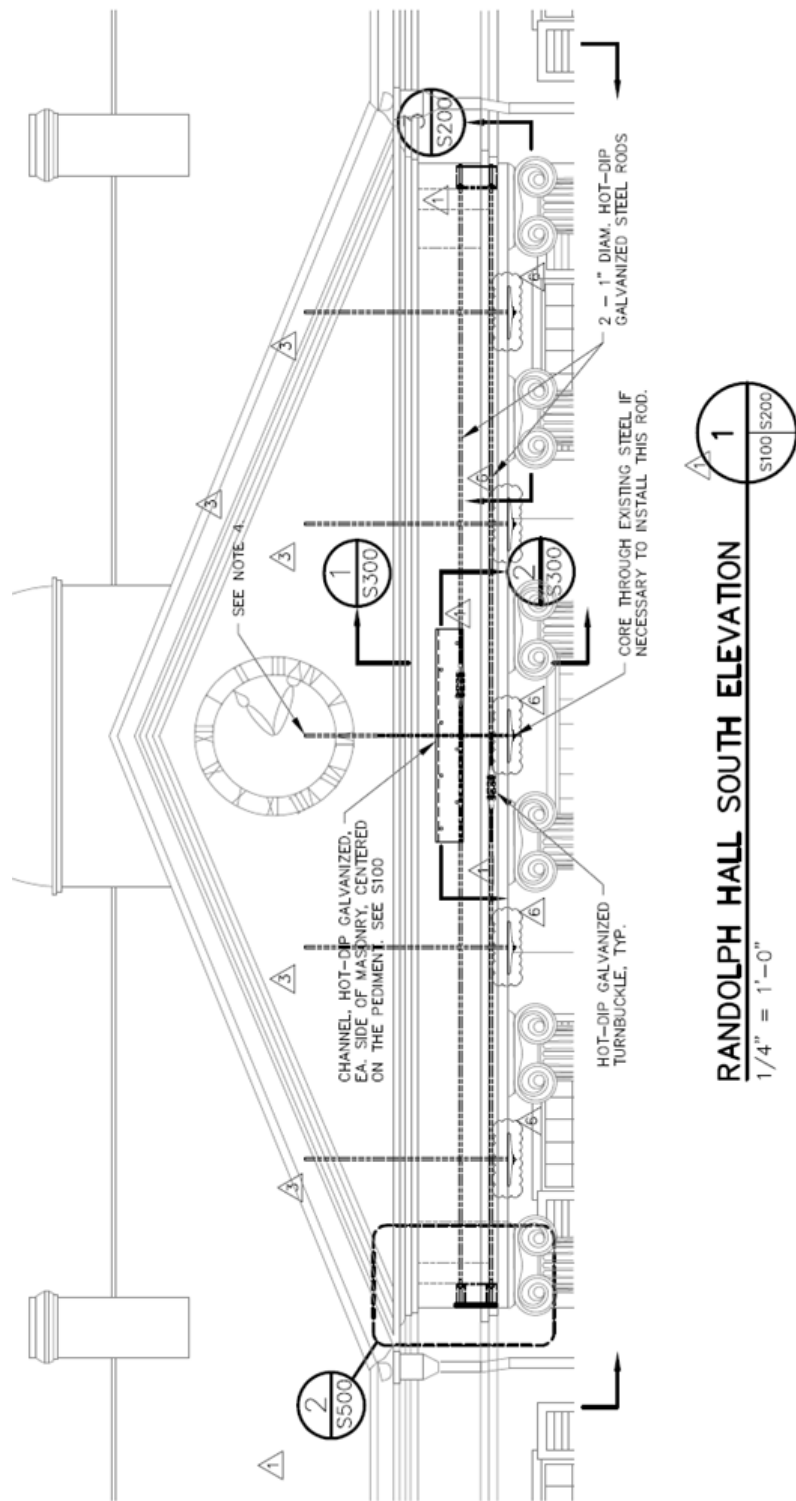


Figure 4.37: Elevation drawing showing the location of vertical tie rods installed to secure the brownstone entablature at Randolph Hall. (Drawings provided by 4SE, Inc.)



Figure 4.38: Photo showing the brownstone entablature and underneath the portico roof. The oval pattern plates can be seen securing the brownstone entablature with the tie rod and turnbuckle tying the portico side walls behind the entablature. *(Photo taken by author)*

between each column through the entablature, a total of five tie rods. Due to the unique nature of the brownstone entablature, the deformed reinforcing bar is cored through the brick gable and penetrates the masonry approximately seven to eight feet above. A concrete grout provides added security around the rod within the gable and an exterior anchor plate serves to secure the exposed end of the reinforcement. The elevation detail drawn in Figure 4.37 illustrates the location of one of these reinforcement rods within the entablature and roof gable. When standing on the portico floor and looking up, these unique 21st century applications of the tie rod can be seen adjacent to one another (see figure 4.38).

Summary

Buildings are essentially documents of construction history and practice. Analyzing structures in terms of their various construction methods and alterations throughout their lifetime reveals them to be products of the technologies and know-how of the time.

When the Council Assembly mandated in 1743 that the Old Powder Magazine in Charleston “must be secured on every side”, they employed the structural solution of wrought iron tie bars from wall to wall – a structural remediation technique that had been in use since ancient times, but not so much in the lowcountry region. The rough, hand-wrought bars and large X-shaped anchor plates are perhaps the earliest remaining example of iron tie reinforcement being used in a load bearing masonry structure in Charleston. The reinforcement practice proved effective, and the technology of producing tie rods and installation methods evolved through the turn of the century. Iron reinforcing plates are evident on all four facades of the Nathaniel Russell house and proliferate throughout the College of Charleston’s Randolph Hall, where multiple episodes of structural reinforcement have been identified at both buildings. At the Russell house, the first occurred in the early 19th century following the tornado of 1811, and is characterized by hand-wrought, bar iron with integral tie rods and rectangular straps bedded in the masonry walls of the house. Randolph Hall also features reinforcement distinguished by its bar iron form and simple, hook-like center attachment. A second episode of reinforcement was required at both structures following the disastrous earthquake that struck Charleston on August 31, 1886, a period in which the tie rod truly proliferated in Charleston. Characterized by their circular-section, rolled stock iron tie rods penetrate the outer masonry walls and are secured to cast iron plates of several distinct types. Randolph Hall provides a unique look into the use of tie rod reinforcement today.

Although this thesis is not about reinforcing historic structures in retrofit, there are implications in the history of this reinforcement technology for retrofit planning. It clarifies the historic availability, characteristics and use of the tie rod technology, and aligns these historical features with those that are used today. Above all, the history and evolution of the iron tie rod technology demonstrates that they were manufactured and installed out of structural needs bound by time and place.

CHAPTER FIVE

CONSERVATION OF IRON REINFORCEMENT: OLD POWDER MAGAZINE

The following conservation study of the iron reinforcement bars at the Old Powder Magazine was initiated as a part of the Conservation Lab course in the Clemson/College of Charleston Graduate Program in Historic Preservation. The report is intended to be a supplement to the aforementioned case study that details the construction of the Powder Magazine as it affects the condition of the iron tie bars today. This chapter details conservation issues that surround the reinforcement and includes a description of the techniques used to take a mold of the iron stamps in an attempt to learn more about the possible manufacture and installation date of the reinforcement. Established methods of iron conservation have been provided with subsequent recommendations for selecting the appropriate treatment of the reinforcement bars and attachment elements (i.e. anchor plates and bolts). It is the goal of this conservation report to provide a preliminary assessment of the current condition of the iron tie bars within the Old Powder Magazine, and to provide recommendations for material testing, cleaning and protection of the structural element until more in depth conservation efforts can be undertaken.

Components of Deterioration & Conservation Issues

The corrosion (rusting) of embedded metals, like iron, is triggered by water penetration on the inadequately protected metal. It usually occurs within the external walls or roof, in damp, wet environments. Rust can be five to ten times greater in volume than the original metal, and consequently can displace the masonry in which it is contained. The resulting movement, visible on the masonry wall surface, is often the primary evidence of problems beneath. In riveted wrought iron, local corrosion from water trapped at connections or at overlaps between members can cause prying apart of the metal and joint failure. Originally, wrought iron ties were heated and plunged into linseed oil for protection. Later, steel ties were galvanized. However, the coating was often not thick enough to assure protection, especially if the masonry mortar contains ash or other sulfate material. If corrosion reaches a point where the reinforcement

becomes ineffective, subsequent consequences can include the buckling of walls from the loss of support¹⁰⁶.

In 1996 the main focus of the Historic Charleston Foundation's restoration effort at Charleston's Old Powder Magazine was to stabilize the exterior skin of the building with a replacement roof and exterior stucco repairs. Yet, the years of exposure to moisture had already directly affected the iron reinforcement bars embedded within the Powder Magazine walls. According to architect Glenn Keyes, "The building has had a roof that's leaked forever, maybe since the building was built"¹⁰⁷. Almost completely exposed, the ties stretch between the east and west walls of the structure, penetrating the 32" thick masonry walls. The only portion of the wrought iron tie bars that are concealed are the portions that cut through the brick – the same brick and mortar that became saturated with water before the roof restoration. The extremely high moisture content of the masonry walls was reaffirmed by the discovery of the condition of bond timbers within the wall during the same 1996 restoration. As masonry was being repaired,



Figure 5.1: Localized corrosion where rectangular tie rod enters interior wall. (Photo taken by author)

¹⁰⁶ Michael Forsyth, *Structures & Construction in Historic Building Conservation* (Oxford: Blackwell Publishing, 2007), 133-134.

¹⁰⁷ Robert Behre, "Building Loses its Patina for the Good", *Post and Courier*, 3 June 2003, accessed 25 October 2011, <http://archives.postandcourier.com/archive/arch03/0603/arc06231218018.shtml>.

a large void was unveiled in the wall where originally a 12"x12" bond beam had run in the middle of the entire circumference of the outer walls. This void was the result of the timber beam that had almost completely rotted away over the years due to high moisture exposure¹⁰⁸.

Another issue at the Old Powder Magazine is the high salinity. "There's so much salt in the walls, the alkalinity is tremendous,"¹⁰⁹ Keyes says. As a result, the integrity of the embedded structural iron has no doubt been compromised. The question is to what extent. The recent restoration of the brick and stucco on both the interior and exterior of the Powder Magazine has possibly hidden any previous evidence of masonry movement as primary evidence of problems beneath. However, there is some indication of corrosion and deterioration of the iron bars at the locations where they enter the interior walls (see figure 5.1). There is also visible evidence of rusting on some areas of the exterior anchor plates. The plate located within the southwest gable of the magazine shows significantly more pitting and deterioration than the other three plates (see figures 5.2 and 5.3).

The primary concern at the forefront of the conservation of the Old Powder Magazine's wrought iron tie bars is the fact that the deteriorating portions of the material are in fact embedded within the load bearing masonry wall. Because the reinforcement ties are structural, they cannot simply be removed temporarily for lab testing or conservation. Any material testing and conservation techniques have to be done in situ, or on site. This presents a severe limitation when considering the established methods of iron conservation and making recommendations for future action.

Established Methods of Structural Iron Conservation

Material Testing

The conservation of structural wrought and cast iron depends heavily on the mechanical properties of the materials. The earliest wrought iron is known as 'charcoal iron'. At first it was produced by the direct reduction process. Iron ore was smelted by heating it with charcoal in small furnaces called bloomeries. The charcoal (essentially carbon) reduced the iron oxide to

¹⁰⁸ Document detailing work completed as of 5 April 1996. Richard Marks Restorations, Project Files. Accessed on 8-9 November 2011.

¹⁰⁹ Behre, "Building Loses its Patina for the Good".



Figure 5.2: Evidence of maker's marks & paint deterioration on the northeast anchor plate arm. *(Photo taken by author)*



Figure 5.3: South-west anchor plate showing severe deterioration. *(Photo taken by author)*

iron, giving off carbon monoxide in the process. Much of the mineral impurities, or 'slag', remained in the iron. After smelting, the blooms of iron were forged by heating to red hot and then beating them out into long bars. The process could be repeated many times. With each forging, more of the slag was removed, and the fine residues left behind became integrally incorporated with the iron, together forming the distinctive fibrous microstructure which gives wrought iron its tensile strength. By the mid-19th century, wrought iron was considered relatively expensive. Cast iron, although weak in tension, was mass-produced and was therefore much cheaper. For structural applications, cast iron replaced wrought iron wherever loads were carried in compression, such as columns or anchor plates, but wrought iron continued to be used in tension¹¹⁰. It is these tensile and compressive properties that must be tested in historic ironwork before an appropriate conservation decision can be made.

In the iron conservation field, there is a need for increasing reliance on non-destructive testing methods. Non-destructive testing is an in situ process employed with the goal of making an assessment of conditions without significantly harming the material being tested. The primary uses of non-destructive testing methods are to investigate subsurface conditions of embedded metals that are not visible to the naked eye and to test various aspects of structural integrity, capacity and movement. These testing strategies employ a variety of techniques ranging from simple tapping of materials to the use of sophisticated instruments that use light waves, x-rays, fiber optics and mechanical pressure to assess conditions. A representative sampling of these non-destructive techniques is the following¹¹¹:

Infrared Thermography: Used to document radiant energy as a thermal indicator. The infrared photograph of energy emanating from a surface is used to identify moisture and dissimilar metals within an assembly.

Impulse Radar: Used to assess a wide variety of materials up to a depth of several feet. This method detects changes in material density within a wall (i.e. metals within masonry) or within the ground (see figure 5.4).

X-ray Analysis: Used to investigate subsurface conditions within an assembly that is accessible from both sides. This method enables x-ray analysis to reveal varying materials, connection assemblies, and abnormalities in the assembly.

¹¹⁰ Jonathan Taylor, "Wrought Ironwork" in *The Building Conservation Directory*, (2000). www.buildingconservation.com.

¹¹¹ Robert Young, *Historic Preservation Technology*, (New Jersey: John Wiley & Sons, 2008), 40-41.

Moisture Meter: Used to calculate the presence of water in a material. The method allows for a preliminary assessment of moisture migration within masonry.

Rebound Hammer/Rockwell Hardness Test: Used to determine the material strength based on surface hardness. This is an in situ test used on concrete and metal structural elements.

The principal advantage of these tests is that they produce minimal or no loss of historic material when compared to the alternative of other destructive testing methods. These include dismantling or demolishing portions of historic buildings to permit visual inspection of concealed assemblies or removal of historic materials to a laboratory for destructive testing.

While mechanical-property data for iron has been tabulated in several sources since the mid-20th century, particularly for iron manufactured in the 19th century, the number of primary research studies is limited. Since there were no manufacturing standards when the material was first being produced, these material properties can vary widely and need to be tested and

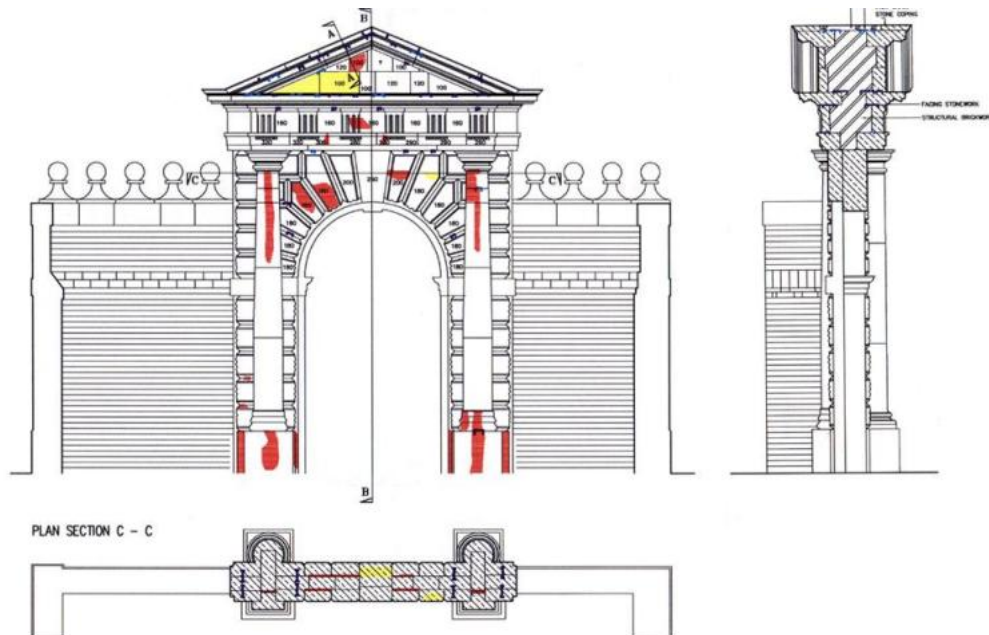


Figure 5.4: Impulse radar used at Inigo Jones' gateway at Chiswick Park to detect the presence of reinforcement metals and the condition of embedded elements. (Source: "Nondestructive Evaluation for Historic Preservation", www.architectureweek.com)

analyzed on a case-by-case basis. One such case study was presented in Volume 29 of the Association for Preservation Technology Bulletin published in 1998¹¹². In *Metallurgical Assessment of Historic Wrought Iron*, the authors report on the study of the material properties of historically important I-beams used to construct the U.S. Custom House in Wheeling, West Virginia. Ball indentation hardness testing was performed to obtain tensile strength estimates for the historic wrought iron material used in the structure, which were combined with results from a more complex metallographic examination. Fortunately in the case of the Wheeling Custom House, sufficient iron samples were available for both traditional destructive and non-destructive tests, allowing comparison of results. The Rockwell hardness testing was accomplished using a Wilson model B 534T machine equipped with a 1/16" hardened steel ball¹¹³. The reported results of the study clearly demonstrate the highly inconsistent nature of the hand-wrought iron material and the presence of defects that reduce the strength of such members. This reinforces the call for an evaluation of historic ironwork on a case-by-case basis. Yet, the Wheeling Custom House provides a cautionary case study. The study relies heavily on the destructive metallurgical testing methods. It is rare that historic material samples are available for destructive testing, as would be the case at the Old Powder Magazine on Cumberland Street.

Cleaning & Surface Preparation

Severe corrosion on the exposed portions of historic ironwork is typically localized around difficult-to-paint areas, which are liable to retain moisture. Aesthetic considerations aside, it may only be necessary to remove loose paint and corrosion in addition to any grease and dirt which will compromise the adhesion of the new coating. Appropriate surface preparation is the key to new coatings reaching their full service potential. A range of targeted cleaning methods may provide the best way to prevent further deterioration, while avoiding unnecessary disturbance of sound paintwork and historic surface finishes. The main methods of removing corrosion and old coatings include: hand and power tool cleaning, chemical stripping,

¹¹² Wayne Elban et al, "Metallurgical Assessment of Historic Wrought Iron: U.S. Custom House, Wheeling, West Virginia," in *Association for Preservation Technology Bulletin*, Volume 29, No. 1. (1998), 27-34.

¹¹³ Elban, "Metallurgical Assessment", 34.

flame cleaning, air abrasive methods such as grit blasting, and high pressure water blasting¹¹⁴. Manual techniques of removing light rust and paint involve using scrapers, wire brushes and chipping tools such as needle guns. These methods are the most common and least expensive methods of removing paint and light rust from iron. However, they do not remove all corrosion or paint as effectively as other methods.

Chemical rust removal, by acid pickling, is best used only on iron elements that can be easily removed and taken to a lab for submersion in dilute phosphoric or sulfuric acid. This method does not damage the surface of iron, providing that the iron is neutralized to pH level 7 after cleaning. Other chemical rust removal agents include ammonium citrate, oxalic acid, or hydrochloric acid-based products. Chemical paint removal using alkaline compounds, such as methylene chloride or potassium hydroxide, is much more easily used in the field and can be an effective alternative to abrasive blasting for removal of heavy paint buildup. These agents are often available as slow-acting gels or pastes. When using these acid-based materials, it is necessary to ensure that any unreacted acid is removed by rinsing and is prevented from getting into crevices¹¹⁵.

Flame cleaning, involving wire brushing while burning with an oxy-acetylene or oxy-propane torch, is sometimes used (either alone or as a pre-treatment to air abrasive and chemical cleaning) to break down thick coatings and detach heavy corrosion. However, the crystal structure of iron can be significantly affected by heat treatment, and cast iron is particularly vulnerable to fracturing. However, it can be very effective on lightly to moderately corroded iron. Wire brushing is usually necessary to finish the surface after flame cleaning¹¹⁶.

Low-pressure grit blasting (commonly called abrasive cleaning or sandblasting) is often the most effective approach to removing excessive paint buildup or substantial corrosion. Grit blasting is fast, thorough, and economical, and it allows the iron to be cleaned in place. The aggregate can be iron slag or sand; copper slag should not be used on iron because of the potential for electrolytic reactions. Some sharpness in the aggregate is beneficial in that it gives the metal surface a "tooth" that will result in better paint adhesion. The use of a very sharp or

¹¹⁴ John G. Waite, "The Maintenance and Repair of Architectural Cast Iron," in *National Park Service Preservation Brief 27*, (Washington DC: Department of the Interior, 1991), www.nps.gov.

¹¹⁵ Keith Blackney, "Cleaning Historic Ironwork for Repainting," in *The Building Conservation Directory*, 2002. www.buildingconservation.com.

¹¹⁶ Blackney, "Cleaning Historic Ironwork".

hard aggregate and/or excessively high pressure (over 100 pounds per square inch) is unnecessary and should be avoided. Adjacent materials, such as brick, stone, wood, and glass, must also be protected to prevent damage. Some local building codes and environmental authorities prohibit or limit dry sandblasting because of the problem of airborne dust. Wet sandblasting reduces the amount of airborne dust when removing a heavy paint buildup; however, wet sandblasting is more problematic than dry sandblasting for cleaning cast iron, because the water will cause instantaneous surface rusting and will penetrate deep into open joints. Therefore, it is generally not considered an effective technique¹¹⁷.

One of the more recent and thorough publications on the conservation techniques for historic iron work (in addition to standard textbook publications) is Willie Mandeno's *Conservation of Iron and Steelwork in Historic Structures and Machinery: Maintenance Handbook*, published in 2008. The handbook was prepared to provide guidance on the basic principles and techniques involved in the preservation of historic iron and steelwork standing outdoors in New Zealand. However, these basic principles are essentially universal in their application to the conservation of the material, despite location. In the chapter entitled "Protective Coatings", he details the surface preparation of the iron material for protective coatings, specifically iron in high-salinity environments near the coast. Since nearly all coatings are permeable to some degree, it is important that the surfaces in coastal areas are cleaned to remove any salts that could draw moisture through the film resulting in blistering or pitting of the coating. Where the metal has been exposed to marine salts, it is also necessary to remove any scale, so that water-soluble salts that would otherwise be trapped in the bottom of any pits in the metal surface can be removed. Failure to do this will result in the early breakdown of the coating in damp and humid environments¹¹⁸.

Willie Mandeno also argues that removal is best achieved by slurry blasting, where an abrasive medium is introduced into a jet of water, or by alternate water blasting and dry abrasive blasting. He contends that high- or ultra-high pressure water jetting is the most effective technique for removing salts from pitted iron or steel. Because salts can be concentrated in pits under rust, they cannot be effectively removed by low-pressure rinsing

¹¹⁷ Waite, "Maintenance and Repair of Architectural Cast Iron".

¹¹⁸ Willie Mandeno, *Conservation of Iron and Steelwork in Historic Structures and Machinery: Maintenance Handbook*, (New Zealand: Science & Technical Publishing, 2008), 9-10.

unless the rust is removed first. He states that this is also why so-called rust-converters are not effective on corroding iron and steel near the coast. However, Mandeno does acknowledge that while abrasive blast cleaning by dry blasting or wet slurry blasting is ideal for rust removal and creating a surface profile that anchors the protective coating, the complete removal of rust is not always practical and abrasive blasting can also be damaging to thin sections. In these situations, the surface should first be scraped and brushed with hand or power tools, then thoroughly rinsed to remove as much embedded salt as possible¹¹⁹.

A typical maintenance painting specification clause will require cleaned surfaces to be primed as soon as possible after cleaning and before re-rusting occurs, to minimize the risk of recontamination. Once dry, hand-prepared surfaces should be treated with a penetrating sealant or primer. Ideally, rust and iron scale should be fully removed before protective coatings are applied. However, it may be unnecessary to strip back to bare metal across the whole structure. Some corrosion products can themselves make a stable crust protecting the underlying metal. Furthermore, the paint layer itself may also contain important historical information, providing an invaluable insight into past coating technology as well as the decorative history of the metalwork itself. The benefits of cleaning must always be weighed against the risk of accelerated decay and loss of historic material. Cleaning should not be carried out as a matter of course. Metalwork repairs can target areas which have corroded more than others, avoiding collateral damage to adjacent areas of sound historic paintwork. If stripping important metalwork of its historic coating cannot be avoided, a specialist should be brought in to take samples for paint analysis¹²⁰.

Protective Coatings

The suitability of different processes and coating materials for the protection of iron elements depends upon the original production & finishing methods and the intended future use or environment. A key factor to take into account in selection of coatings is the variety of conditions on existing and new materials on a particular building or structure. One primer may be needed for surfaces with existing paint; another for newly cast, chemically stripped, or blast-cleaned cast iron; and a third for flashings or substitute materials; all three followed by

¹¹⁹ Mandeno, *Conservation of Iron and Steelwork*, 9-10.

¹²⁰ Blackney, "Cleaning Historic Ironwork for Repainting".

compatible finish coats. Numerous types of these protective coatings exist and can be categorized in various ways: chemical type (metallic or non-metallic), generic type (type of resin or binder), method of curing, type of solvent, etc¹²¹.

Traditionally, red lead was used as an anticorrosive pigment for priming iron. Linseed, plant and fish oils have also been used in protective coatings since Roman times. Boiled linseed oil mixed with lead oxide was used to protect iron-work in Victorian times and became the standard priming material for steel in the first half of the 20th century. According to John and Nicola Ashurst in the *English Heritage Technical Handbook*, "A traditional treatment for wrought iron was to scrape, chip or pickle the surface until all scale and foreign substance was removed. A heavy coat of linseed oil was applied, and then the iron was heated and wiped over with emery cloth. Finally a combination of beeswax and boiled linseed oil was rubbed into the surface"¹²². The most valuable property of these coatings is their low viscosity and ability to penetrate crevices and saturate the surface. However, they have relatively low resistance to moisture vapor and chemicals compared with paints based on synthetic resins¹²³.

Today, alkyd paints are very widely used and have largely replaced lead-containing linseed oil paints. They dry faster than oil paint, with a thinner film, but they do not protect the metal as long. Before paint is applied, an alkyd rust-inhibitive primer containing pigments such as iron oxide, zinc oxide, and zinc phosphate should be applied. These primers are suitable for previously painted surfaces cleaned by hand tools. At least two coats of primer should be applied, followed by alkyd enamel finish coats. Alkyd paints and oil-based coatings should not be applied directly to galvanized surfaces, as they may delaminate (due to a reaction with the zinc). Their main use is as decorative paints, but they are also able to provide long-life protection to iron and steel in moderate environments¹²⁴.

When the iron element under assessment has suffered surface damage, it is possible to correct the damage once the source has been eliminated (i.e. rust). The corrosion can be removed or a corrosion converter can be applied. A corrosion converter works chemically to transform corrosion into an inert material. Next, a corrosion inhibitor is applied to stop

¹²¹ Mandeno, *Conservation of Iron and Steelwork*, 11-12.

¹²² John and Nicola Ashurst, *Practical Building Conservation Vol. 4: Metals* in English Heritage Technical Handbook, (Hants: Gower Technical Press, 1988).

¹²³ Mandeno, *Conservation of Iron and Steelwork*, 12.

¹²⁴ Mandeno, *Conservation of Iron and Steelwork*, 12.

corrosion from recurring. Last, an epoxy material, formed by the reaction of an epoxy resin with a variety of different curing agents, can be worked into the damaged area to build up a continuous surface. This surface is then painted or coated to match the finish of the adjoining original metal. For the most part, these treatments are not reversible, but they are used when the historic significance of the material warrants maximizing its retention, or when replacement of the entire element is not possible. One particularly effective system has been first to coat commercially blast-cleaned iron with a zinc-rich primer, followed by an epoxy base coat, and two urethane finish coats. Some epoxy coatings can be used as primers on clean metal or applied to previously painted surfaces in sound condition. However, these coatings typically require highly clean surfaces and special application conditions which can be difficult to achieve in the field on large buildings. Epoxies are particularly susceptible to degradation under ultraviolet radiation, so they must be protected by finish coats which are more resistant¹²⁵.

Cathodic Protection

Corrosion rates are significantly higher where iron is in direct contact with damp stone or masonry, rather than just air, and conventional conservation methods can sometimes be highly invasive. The conventional remedies involve major 'surgery' to remove iron cramps, rods or ties, and replace them with non-corroding phosphor bronze or stainless steel and then repair the damaged stonework. Cathodic protection (abbreviated as CP) offers an alternative approach to the treatment of ironwork embedded in masonry and stone. CP is not a new process: in 1824 Sir Humphrey Davy presented a series of papers to the Royal Society describing how CP could be used to prevent the corrosion of copper sheathing in the wooden hulls of British naval vessels¹²⁶. Since then it has been applied in many areas and, over the past 20 years, has been applied to reinforced concrete to protect steel reinforcements from corrosion. More recently, it has also been applied to iron and steel embedded in brick, masonry and stone in historic buildings¹²⁷.

Cathodic protection encompasses a range of techniques used to suppress corrosion of metal structures and components. Essentially corrosion is an electrochemical process in which electrons flow between cathodic (positively charged) and anodic (negatively charged) areas on a

¹²⁵ Young, *Historic Preservation Technology*, 146.

¹²⁶ David Farrell and Kevin Davies, "Cathodic Protection of Iron and Steel in Heritage Buildings in the United Kingdom," in *Association for Preservation Technology Bulletin*, Volume 36 (2005), 19.

¹²⁷ Farrell and Davies, "Cathodic Protection of Iron and Steel", 19.

metal surface; corrosion occurs at these two anodes. Cathodic protection is used to control the corrosion of metal by making the whole metal surface the cathode of an electrochemical cell. When used to protect structural iron and steel this is achieved by applying small DC electric currents, via the building material. This supplies a constant stream of electrons to satisfy the cathodic reaction. The corrosive reaction then becomes suppressed. There are two methods of achieving this, either Sacrificial Anode Cathodic Protection (SACP) or Impressed Current Cathodic Protection (ICCP) (see figure 5.5). SACP systems use sacrificial anodes (zinc, aluminum or magnesium), which are placed in close proximity to the corroding metalwork and electrically connected to it. As the sacrificial anode corrodes, it generates a current that passes through the building material to provide protection to the embedded metalwork. The current is ionic-ally conducted by means of pore water contained within the building material. These systems are capable of protecting small metal components, such as embedded iron cramps or restraints set into walls, floors or roofs of a building. ICCP systems use transformer rectifiers, normally mains powered, to provide the DC current to the iron or steel being protected. These systems use corrosion resistant anodes, fixed close to the metalwork, to provide part of the current pathway.

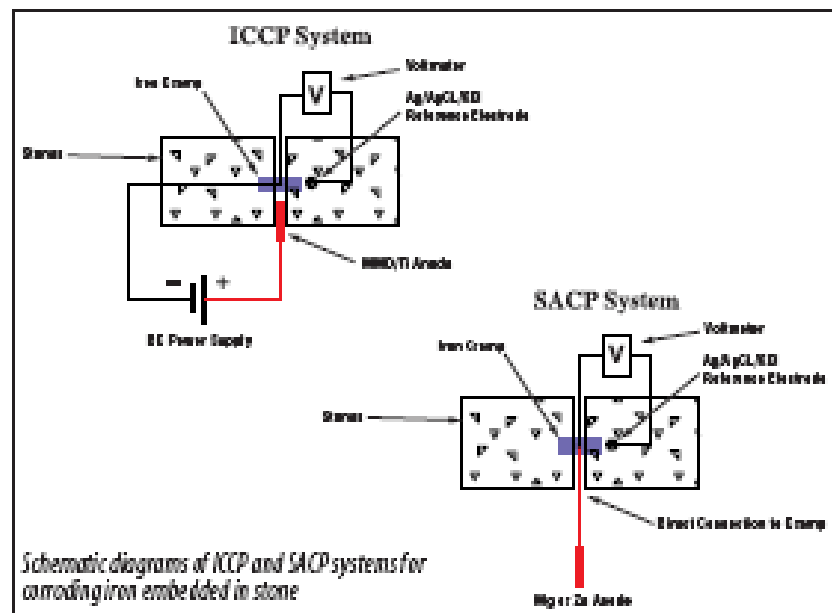


Figure 5.5: Schematic diagrams of ICCP and SACP systems for corroding iron in stone. (Source: Farrell, David and Kevin Davies. "Cathodic Protection of Iron and Steel in Heritage Buildings in the United Kingdom". *APT Bulletin* 36, 2005)

ICCP systems are more complex than the SACP systems, but are suitable for providing CP to much larger areas of embedded steel such as I-beams, supports and columns. However, impressed-current cathodic protection systems are typically the most costly to install, and they require substantial ongoing monitoring and adjustment to ensure a proper voltage output over time¹²⁸.

The power supply and other electronics of a cathodic protection system may be expected to have a lifetime of between 20 to 40 years after which they can readily be replaced. The external wiring may also suffer long-term decay and may require replacement after 40 to 60 years. However, the embedded anodes and internal wiring within the masonry and stonework are not easily replaced, and should therefore be selected to give a maximum service life. The design of a CP system should also take into account other factors including¹²⁹:

- The surface area of iron to be protected
- The resistivity of the masonry or stonework
- The distance of the anodes to the embedded metalwork
- The aesthetics of the building

Cathodic Protection technology has, over the past 30 years, been applied to concrete to protect steel reinforcement from corrosion and, over the past 15 years, it has also been applied to iron and steel embedded in masonry and stone heritage buildings in the United Kingdom. The first CP system installed in the UK on a heritage structure was carried out by English Heritage to protect embedded iron cramps on Inigo Jones' Gateway at Chiswick House in 1996. This structure is still monitored on an annual basis and shows no corrosion or iron staining of the masonry surrounding the cramps. A second CP system was installed in 1999 to protect rusting cramps in the stone faced of four of the almshouses in Whitchurch. The cottages were inspected in 2009, ten years after installation, and a visual assessment showed no spalling or iron staining where the cramps had been protected by cathodic protection. However, a stray cramp that had not been detected in 1999, and which had been omitted from the CP system, had continued to corrode and had blown away the stone¹³⁰.

¹²⁸ Farrell and Davies, "Cathodic Protection of Iron and Steel", 20-21.

¹²⁹ Christopher Hunt and David Farrell, "Cathodic Protection of Tie Bars and Ring Beams in Church Towers", *Rowan Technologies* (2011), 2-3.

¹³⁰ Hunt and Farrell, "Cathodic Protection of Tie Bars and Ring Beams", 2-3.

Most recently, a successful ICCP system has been installed in the Wellington Arch at Hyde Park Corner (see figure 5.6). The arch was built in the late 1820s using Portland stone and the roof slab was formed from concrete, supported by steel I-beams. The arch has recently undergone extensive renovation, including the repair of the steel and concrete structure which supports a large bronze sculpture. During an inspection it was discovered that some of the key steel I-beams in the roof had suffered significant corrosion. This was partly due to rainwater penetrating the roof structure. The worst corrosion, however, was where the beams had been in direct contact with the Portland stone (see figure 5.7). In fact, as a sedimentary rock formed in a marine environment, Portland stone may sometimes contain significant concentrations of chlorides or sulfates which, in the presence of moisture, can accelerate the corrosion process. A similar environment can be found within the Old Powder Magazine brick walls. At the Wellington Arch, English Heritage did not wish to replace the beams, as this would have been both expensive and disruptive. Thus, ultrasonic readings of the steel beams were taken, showing that their remaining thickness was sufficient to support the roof slab and sculpture, provided that ongoing corrosion could be controlled¹³¹.

While cathodic protection as a conservation technique for heritage structures has been widely accepted in the United Kingdom, relatively few examples of the system being used in the United States exist. However, in 2005, a large scale CP installation was initiated at the National Holocaust Museum in Washington D.C., only the fourth such installation in the United States. Constructed in 1904 and located along the Mall in D.C., the building houses the administrative functions of the historic museum (see figure 5.8). The facility is a National Historic Landmark and an important destination for many. In 2005, a condition survey revealed the need for an entire exterior envelope repair, re-pointing of the brick, patching and repair of the terra cotta elements, and other elements typical of a thorough historic preservation and repair report. The report also indicated that the building's terra cotta cornice was deteriorating due to corrosion of the supporting steel elements and that the main joint above the modillions, which was cracked and open in areas, had been previously improperly repaired with a Portland cement mortar. The corrosion product was up to 0.75 inches thick, causing downward stresses and cracking on the terra-cotta modillions. Because the owners required repair options that would protect the

¹³¹ Farrell and Davies, "Cathodic Protection of Iron and Steel", 22-24.



Figure 5.6: The Wellington Arch, Hyde Park, London; constructed in 1820, a recent inspection revealed significant corrosion of the steel I-beam supporting the Quadriga sculpture. (Source: Farrell and Davies. "Cathodic Protection of Iron and Steel in Heritage Buildings in the United Kingdom". *APT Bulletin* 36, 2005)

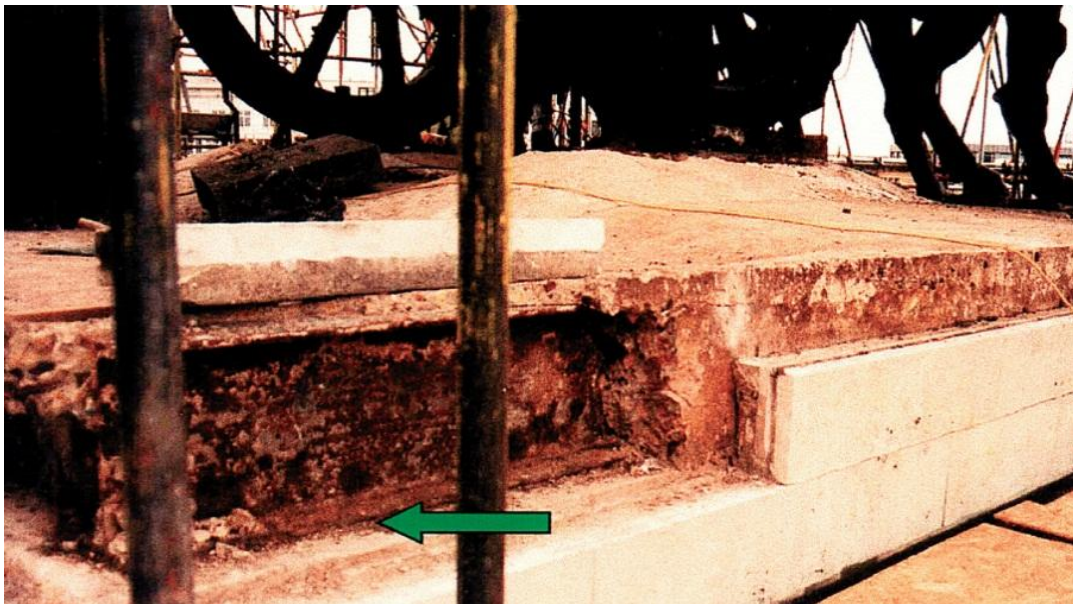


Figure 5.7: Corroded steel I-beam within the Wellington Arch; Not wanting to replace the historic steel structure, English Heritage chose to install an ICCP system to provide sufficient long-term corrosion prevention. (Source: Farrell and Davies. "Cathodic Protection of Iron and Steel in Heritage Buildings in the United Kingdom". *APT Bulletin* 36, 2005)



Figure 5.8: A view of the restored terra cotta cornice at the National Holocaust Museum in Washington, D.C. (Source: Kelly Page, ed., “Cathodic Protection of Historic Terra Cotta Cornice”, *Concrete Repair Bulletin* (May/June 2009): 20-23, accessed 27 January 2012, www.icri.org)

historic integrity of the building as well as prevent further damage, the project team developed a solution to install an Impressed Current Cathodic Protection (ICCP) system¹³².

Contractor selection and training was a crucial because of the limited ICCP installations on historic buildings in the United States. The installation crew also was trained on project-specific procedures and quality assurance for this structure. Once the contractor began to remove the mortar joint above the modillions, anodes were inserted into areas to deliver corrosion protection. Joints were repointed as the installation was occurring, which masked all visible evidence of the system and prevented prolonged exposure to oxygen and moisture (see figures 5.9 and 5.10). The anodes were then connected to a DC power supply to deliver low-voltage electric currents. The system has been successfully monitored and running without issue since January 2007 and is controlled by a specialized control unit specially designed for steel frame corrosion in heritage buildings (see figure 5.11)¹³³. Although primarily concerned with steel corrosion, the cathodic protection system at the National Holocaust Museum

¹³² Kelly Page, ed., “Cathodic Protection of Historic Terra Cotta Cornice”, *Concrete Repair Bulletin* (May/June 2009): 20-23, accessed 27 January 2012, www.icri.org.

¹³³ Page, “Cathodic Protection of Historic Terra Cotta Cornice”.



Figure 5.9: Discrete anode connected to power supply and ready for placement into mortar joint. (Source: Kelly Page, ed., "Cathodic Protection of Historic Terra Cotta Cornice", *Concrete Repair Bulletin* (May/June 2009): 20-23, accessed 27 January 2012, www.icri.org)



Figure 5.10: Placement of lime-based mortar after anode and reference electrode installation. (Source: Kelly Page, ed., "Cathodic Protection of Historic Terra Cotta Cornice", *Concrete Repair Bulletin* (May/June 2009): 20-23, accessed 27 January 2012, www.icri.org)



Figure 5.11: Monitoring equipment installed inside the building allows for remote monitoring and system adjustment. (Source: Kelly Page, ed., “Cathodic Protection of Historic Terra Cotta Cornice”, *Concrete Repair Bulletin* (May/June 2009): 20-23, accessed 27 January 2012, www.icri.org)

demonstrates a successful CP installation in the United States and that the benefits of the system are threefold: the nature of the technology is non-destructive, cost-savings associated with the system are often greater versus material replacement, and the ICCP system provides a long-term solution to corrosion related damage.

Conservation Recommendations

In his *Maintenance Handbook* for the conservation of historic iron and steelwork, Mandeno outlines six factors that must be considered when preserving historic ironwork: access (transport and condition), deterioration, climate & environment, appearance, hazardous material and budget constraints¹³⁴. With the last factor being of utmost importance at a structure that is managed by a non-profit organization and maintained as a public museum, a range of suggestions based on priority and budget are provided for selecting the appropriate treatments of the wrought iron tie bars at the Old Powder Magazine:

¹³⁴ Mandeno, *Conservation of Iron and Steelwork*, 19-20.

Monitor & Maintain (Low Budget/Low Priority): No structural remediation is immediately required, but monitoring should be undertaken for future condition changes; any work required to correct problems related to structural movement should be carried out within 5 years. Protective coatings should be maintained on a regular basis; the exterior bearing plates should be cleaned and repainted in the near future to prevent further rusting.

Material Testing (Average Budget/Important): Work is required to determine material limits related to structural movement and accelerating decay; suggested techniques include hardness testing at the exposed areas and x-ray analysis using portable machinery on site.

Stabilization (Extended Budget/Urgent): Full replacement of the tie rods is not recommended unless completely necessary. All action should first be taken to reverse corrosion and prevent further deterioration. Should conditions pose a considerable threat that may become an immediate threat if not stabilized or corrected within the next year; a cathodic protection system is recommended as a non-destructive method to reduce the rate of corrosion of embedded iron.

This system of conservation recommendations addresses the building needs with budgetary constraints in mind before concerns associated with aesthetics. The recommendations relate directly to correcting sources of problems for the sake of retaining the Old Powder Magazine in Charleston for its historic significance, rather than ignoring problems for the short-term gain. However, conservation policy should also always favor the principle of minimum intervention, where any repairs should preserve the historically important features of the architecture. In regards to the Old Powder Magazine, the conservation techniques applied to the wrought iron tie bars should be undertaken in a way that prevents any further deterioration of surrounding materials, principally the historic masonry walls.

CHAPTER SIX

CONCLUSION

Long an object of fascination with sightseers and Charleston residents alike, much of the story behind “earthquake bolts” continues to be hidden behind the heavy, masonry walls of Charleston’s historic buildings. Through the endeavors of this thesis, the historic availability, characteristics, and use of the tie rod technology in Charleston has been expanded and clarified. This thesis addresses the *when* and *where* of iron ties, and considers the research questions concerning *how* and *why* using speculative approaches to style, typology and classification. With an archaeological approach to buildings, this thesis draws attention to a feature whose significance is not typically recognized by vernacular architectural approaches. It provides insight into a subject that has been previously lacking in knowledge – the use of the iron tie rod reinforcement in Charleston’s building tradition.

Inquiries into the use of iron ties in Charleston revealed some unexpected observations. Considering that wrought iron structural tie bars are a historic building technology that have served both utilitarian and stylistic purposes worldwide since ancient times, it is interesting that the majority of tie rod applications in Charleston were imposed as a method of retrofit and repair. Ironwork repairs represent many things when they are discovered in a building. They highlight the historic failure of the frame, be it from a defect of design, overloading, material deterioration, or simple mishap. The use of iron tie reinforcement in some of the earliest structures in Charleston makes it apparent that builders were not completely out of touch with the performance of masonry structures and the latest advances in masonry technology. Yet, even though knowledge existed of the inherent weakness of load bearing masonry, tie rods were only employed once failures were exposed.

Research has demonstrated that contrary to the popular belief that almost all tie rods in Charleston are “earthquake bolts” and were installed following the great earthquake in 1886, the tie rod is a structural reinforcement method that has been incorporated into buildings in Charleston long before the earthquake. Case studies of buildings that exhibit surviving evidence of tie rod reinforcement vary in their quality and usefulness, but all make it evident that the

choice of tie rod construction is not an environmentally determined choice. Rather, tie rod construction is a choice bound by time, place, and intended use.

Examination of tie rod reinforcement in Charleston has further established that, like buildings themselves, iron ties are subject to changes in form, design and material. Most of the early structural ironwork of Charleston is rather plain in form, essentially quadrilateral in section, and surviving examples tend to be fairly sturdy & heavy in appearance when compared to later work. The anchor plates themselves provide perhaps the most usefulness in measuring chronology. Early plate forms are most easily recognized as purely decorative in form: curvilinear shapes represent stylistic tastes and Dutch influence of those who built with them; letter and number-form wall anchors represent builder initials and key dates of construction. The appearance of anchor plates that are more uniform in shape, like the abundant circular form most popular following the 1886 earthquake, demonstrates the growing knowledge of engineering concepts as they relate to structural performance in the 19th century. Still in use today, modern anchor plates exhibit familiar shapes inspired by their predecessors. Yet, dimensions and specifications are meticulously calculated for each project to produce a design – a purposeful design - that accounts for structural capacity. Ideally, one should be able to follow this evolution of form and date structural repairs by closely examining the iron reinforcement. The quality of its material, its design, and its method of attachment are all chronological indicators of the connection between style, function, and period of tie rod reinforcement.

Examination of iron tie rod reinforcement in Charleston raises a number of additional questions and future research opportunities associated with the subject matter within this thesis. An analysis of underlying geology and substrates could be conducted in the lowcountry to determine if the explanation of unstable substrates is justifiable in the early development of iron tie rods in Charleston. Do buildings that exhibit tie rod reinforcement have a tendency to be found on unstable ground? The flexibility of the buildings also needs further study. Inquiries into the transmission of craft are an area that deserves close examination as well. Where exactly did the material and technique come from for the production of these tie rods? What is the relationship between Charleston and the place from which they came? How did the people who produced them and those who built with them identify themselves?

The seismic effectiveness of these now historic tie rods is a fascinating topic that deserves further in-depth examination as another earthquake in the Charleston vicinity is inevitable. The state of South Carolina is not known for its earthquakes, yet it does experience between two and five felt earthquakes a year. Seismically, South Carolina has been active since the first settlers arrived around 1670, as we know based on personal accounts, evidence of damage, as well as seismographs in the later years. The city of Charleston is undoubtedly best known for the earthquake of August 31, 1886, which measured a magnitude between a 6.9 and a 7.3 on the Richter scale, and devastated Charleston and Summerville causing massive structural damage. In the direct aftermath and subsequent years following the earthquake of 1886, numerous masonry structures were repaired and reinforced with iron tie rods – spawning the term “earthquake bolt”. Are buildings utilizing tie rod reinforcement more likely to survive than other contemporary seismic strengthening solutions? Or were they a brilliant scheme by those looking to profit in a damaged and devastated city, which has kept many important buildings in Charleston standing since 1886? Scam or scheme, their effectiveness during another large seismic event is very much open to question. Even in 1886, engineers had their uncertainties:

“Where anchor or tie rods have been put in, some will prove very efficient, especially so where they run through from wall to wall. But very many have been put in with small round plates in the cracked parts of walls, extending in only three or four feet and hooked on to the joist. These, in our judgment, are of little good, and in case of a fire would prove detrimental, because they would assist in throwing down a wall by the burning of the timbers, whereas, without them, the walls might stand.”¹³⁵

However, the effectiveness of earthquake bolts has never been conclusively determined, nor have they been tested in a significant seismic event. Actual earthquake events are so rare, and when they do occur, the forces can be so large that some structural damage is expected even in new structures. As a result, the line between effective and non-effective performance of historic tie rods is certain to always be vague and fluid.

Those outside of the preservation realm often make the narrow-minded assumption that “new is always better”. They come to believe that older forms of construction practice must be more dangerous simply because they were designed before building and seismic codes were

¹³⁵ *Record of Earthquake Damages, 1886*. On microfiche, Historic Charleston Foundation Archives.

adopted, or before current engineering knowledge about building performance had developed. While it is impossible to ignore these modern advances and argue a return to pure traditional construction practice, the debate over such alternatives always turns to one question: how much life safety protection is enough? When existing historic construction techniques remain in use, can they be relied on to continue to perform adequately? Can the preservation of unreinforced masonry buildings be justified when modern building systems can potentially afford a greater degree of life safety? A study of how current preconceptions tend to shape response to the hazards presented by older buildings warrants further exploration. Unless professionals understand the importance of the original structural fabric of the historical buildings, and incorporate this understanding into their plans, they will continue to do what they are used to doing with newer structures – gutting for strengthening.

Sometimes the small finesses of early structural reinforcement are even misunderstood by restoration professionals. One remarkable example is South Hall at the University of California, Berkeley. In the mid-1980s, the late 19th century load bearing masonry structure was gutted to undergo seismic strengthening as part of the University's campus-wide program. The retrofit plans included the demolition and replacing of the timber floors with steel and concrete. In the process of carving channels into the walls, it was discovered that the original builders had installed bond iron in the masonry - continuous bars of wrought iron which extended from corner to corner above and below the windows in all of the building's walls. Iron anchors were also discovered securing the floors to the walls. At the corners, the bond iron bars were secured by large cast iron plates which formed part of the architecture of the building. Despite the appearance of these great cast iron ornamental plates on the exterior, designers never thought to investigate the structural history of the building, including whether or not these plates served a structural purpose. Unfortunately, the existence of the bond iron was not known until the demolition for the retrofit, and all of the bond iron was cut as a result. In addition, as historically significant and advanced as this original reinforcement system was, no recording of it was ever conducted. Ironically, one of the engineers stated that had they known of the existence of the bond iron and the anchors, their retrofit designs may have been different and less extensive.

Upon discovery, however, it was too late to change the retrofit plan, and the early seismic technology was destroyed¹³⁶.

The case of USC's South Hall begs the question, "why is it so important to preserve what has been hidden in the historic wall when it is not visible anyway?" The purpose of historic preservation is not simply to conserve structures and objects as if frozen in time. It should also preserve the slow evolution of building traditions - traditions which just may provide a structure's most effective and lasting defense against time and natural disaster in the end. It is important to recognize the traditions of building which must be preserved, and this thesis is the foundation of a movement towards a more comprehensive understanding of iron tie rod reinforcement and its tradition in Charleston.

¹³⁶ Randolph Langenbach, "Architectural issues in the seismic rehabilitation of masonry buildings", in *Disaster Management Programs for Historic Sites*, eds. Dirk Spenneman and David Look (San Francisco: Association for Preservation Technology and the Johnstone Institute Centre, Charles Sturt University, 2004), 79.

APPENDIX A

Earthquake Reinforcement Survey Form

79-81 Church Street



G. Duger
House

Presence of Visible Reinforcement:

North Façade:

Façade not visible (attached to adjacent structure)

South Façade:

2 anchor plates between 2nd & 3rd floors

East Façade:

None visible

West Façade:

Façade not visible (attached to adjacent structure)

**Date of
Construction:
(Unknown)**

Notes:

The structure that occupies 79-81 Church Street exhibits two anchor plates on the south façade that are placed in between the second and third floors. The north façade is attached to an adjacent structure and not visible, but presumably these tie rods only extend into the floor frame 2-3 joist bays.

The plates themselves are cross-shaped, an early form. The earthquake damage assessment indicates that the structure was already repaired before the assessors made their observations. Thus, it is difficult to determine whether these are in fact “earthquake bolts” or earlier reinforcement.

The stucco beneath the plates shows some discoloration from rust build up. The anchor plates themselves appear to be in good condition.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good
South Wall: Good
East Wall: Good
West Wall: Good

Remarks & Recommendations:

Has been repaired

Earthquake Reinforcement Survey Form
79-81 Church Street: Photographs



Earthquake Reinforcement Survey Form

82 Church Street



Wm. Mills
House

Presence of Visible Reinforcement:

North Façade:

Façade not visible (attached to adjacent structure)

South Façade:

None visible

East Façade:

Façade not visible (attached to adjacent structure)

West Façade:

1 anchor plate between 1st & 2nd floors
4 anchor plates between 2nd & 3rd floors

**Date of
Construction:
(1782 w. 19th
century
alterations)**

Notes:

The structure that occupies 82 Church Street exhibits five anchor plates total, all on the west façade. There is 1 plate between the first and second floors & 4 anchor plates between the second and third floors. Both the north and east façades are attached to adjacent structures and not visible, so presumably these tie rods only extend into the floor frame 2-3 joist bays. The plates themselves are all of the same shape and form, indicating they were probably all installed at the same time. The circular anchor plate with moulded profile is typical of post-1886 repair work.

The earthquake damage assessment indicates that both the west and south walls were damaged and cracked during the earthquake. As a result, anchoring was specifically recommended for the east and west walls, which appears to have been done.

There are no visible signs of deterioration on either of the anchor plates or the stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Division
South Wall: Badly cracked
East Wall: Division
West Wall: Slightly cracked

Remarks & Recommendations:

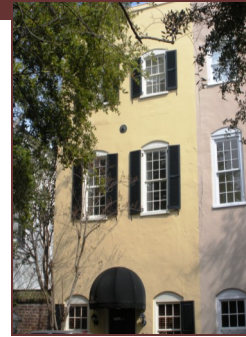
Anchor east and west at floor of each story

Earthquake Reinforcement Survey Form
82 Church Street: Photographs



Earthquake Reinforcement Survey Form

84 Church Street



H. Viohl
House

Presence of Visible Reinforcement:

North Façade:

1 anchor plate between 2nd & 3rd floors

South Façade:

Façade not visible (attached to adjacent structure)

East Façade:

Façade not visible

West Façade:

1 anchor plate between 2nd & 3rd floors

**Date of
Construction:
(Unknown)**

Notes:

The structure that occupies 84 Church Street exhibits two anchor plates. One plate appears centered on the front (west) façade in between the second and third floors. The second plate appears on the side (north) façade in between the second and third floors.

The earthquake damage assessment makes no indication of any damage or repair work done at 84 Church. The anchor plate on the north façade is circular in shape with a moulded profile, presumably post-1886. Whether or not the tie rod was installed before or after the damage assessment was done cannot be determined. The anchor plate on the front façade is a simple, circular plate with a flat profile. It has a form that could be from a mid to late 19th century installation.

There are no visible signs of deterioration on either of the anchor plates or the stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good

South Wall: Good

East Wall: Good

West Wall: Good

Remarks & Recommendations:

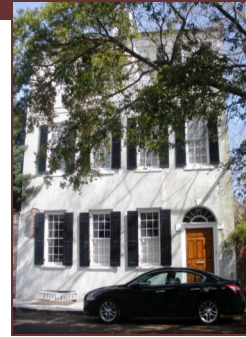
OK

Earthquake Reinforcement Survey Form 84 Church Street: Photographs



Earthquake Reinforcement Survey Form

88 Church Street



Presence of Visible Reinforcement:

North Façade:

2 anchor plates between 1st & 2nd floors

South Façade:

None visible

East Façade:

Façade not visible

West Façade:

3 anchor plates between 3rd floor & roof structure

88 Church
Street

**Date of
Construction:
(Unknown;
possibly post-
1886)**

Notes:

The structure that occupies 88 Church Street exhibits three anchor plates on the front (west) façade in between the third floor and roof structure. Each plate on this façade is centered between window openings and is circular in shape. The north side façade exhibits a total of 4 plates, 2 in between the first and second floors, and 2 in between the second and third floors. Each of these are the same shape, size and profile of the plates on the west façade.

There is no entry in the 1886 Record of Earthquake Damages for 88 Church Street, which more than likely indicates there was no structure on the property at the time of assessment. The house on the property could have possibly been constructed post-1886 and reinforced at a later date. The anchor plates are certainly of a style still in use even after the earthquake of 1886.

There are no visible signs of deterioration on either of the anchor plates or the stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

Property not included in 1886 damage assessment.

Earthquake Reinforcement Survey Form 88 Church Street: Photographs



Earthquake Reinforcement Survey Form

90 Church Street



Peter Leger
House

Presence of Visible Reinforcement:

North Façade:

3 anchor plates between 1st & 2nd floors
2 anchor plates between 2nd & 3rd floors

South Façade:

Façade not visible (blocked by piazza)

East Façade:

Façade not visible

West Façade:

4 anchor plates between 1st & 2nd floors

**Date of
Construction:**
(1759, restored
1927)

Notes:

The structure that occupies 90 Church Street exhibits a total of 8 anchor plates on the front (west) façade, 4 each in between the first and second & second and third floors, respectively. In between each floor, the plates are centered evenly between the window openings and the ends of the walls. A large amount of reinforcement is also visible on the north side façade where 3 anchor plates are centered in between the first and second floors and 2 more in between the second and third floors.

The 1886 damage assessment illustrates that the west wall was badly damaged, and it was specifically recommended that this wall be anchored. Each of the round plates on this façade are slightly smaller than usual with a simple, stepped profile. The circular plates on the north façade are a little larger than those on the west, but retain the same profile. The 1886 assessment makes no mention of damage to the north wall. Therefore, the wall could have possibly been reinforced before the earthquake or several years later following some other cause of wall failure.

There are no visible signs of deterioration on either of the anchor plates or the brick surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good
South Wall: Good
East Wall: Good
West Wall: Badly cracked

Remarks & Recommendations:

Rebuild chimneys; Repair west wall and anchor it to floor beams; Kitchen OK

Earthquake Reinforcement Survey Form 90 Church Street: Photographs



Earthquake Reinforcement Survey Form

92 Church Street



Alexander
Christie
House

Presence of Visible Reinforcement:

North Façade:

None visible

South Façade:

Façade not visible (blocked by piazza)

East Façade:

Façade not visible

West Façade:

2 anchor plates between 2nd & 3rd floors

**Date of
Construction:**
(1805, renovated
1908 & late
1980s)

Notes:

The only visible anchor plates on the front (west) façade of the structure at 92 Church Street are two that are centered in between the second and third floors. The plates are centered evenly between the window openings and the ends of the walls, and are placed within the brick belt course on the façade, perhaps indicating a post-construction application. The rear (east) façade is not visible and the south façade is hidden by the piazza.

The 1886 damage assessment illustrates that the west wall was badly cracked at the corner, yet no specific repair recommendations were made at the time. The circular plates exhibit a flat, non-decorative profile.

There are no visible signs of deterioration on either of the anchor plates or the brick surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good
South Wall: Good
East Wall: Good
West Wall: Badly cracked at corner

Remarks & Recommendations:

Repair kitchen wall & rebuild the main house chimneys from under the roof

Earthquake Reinforcement Survey Form 92 Church Street: Photographs



Earthquake Reinforcement Survey Form

100 Church Street



**Bank of the
United States
Building**

Presence of Visible Reinforcement:

North Façade:

- 1 anchor plate between 1st & 2nd floors
- 3 anchor plates between 2nd & 3rd floors

South Façade:

- Façade not visible (attached to adjacent structure)

East Façade:

- 4 anchor plates between 1st & 2nd floors; bond iron present
- Bond iron present between 2nd & 3rd floors

West Façade:

- None visible

**Date of
Construction:
(1785-1790)**

Notes:

The structure at 100 Church Street exhibits a variety of visible reinforcement. On the side (north) façade four cross-shaped anchor plates can be seen—one between the first and second floors, and three between the second and third floors. The group of anchor plates are concentrated towards the west end of the building. On the rear (east) façade all of the reinforcement is painted with the stucco, but still visible. There are three vertical, rectangular plates centered across the façade between the ground and second floors. One large cross shaped plate also appears at this level on the northern corner of the building. In between the second and third floors, a long bond iron strap extends almost the full length of the wall.

The 1886 damage assessment illustrates that 100 Church was one of the most severely damaged structures on the block. It was recommended that it be pulled down completely. However, the large, cross anchor plate on the rear façade is an early plate indicating that the structure was not completely rebuilt following the earthquake, and in fact had been previously reinforced. The remaining anchor plates and bond iron on the rear façade could be post-1886. Interestingly, in 1886 it was recommended to take the west wall down. Today, this wall does not show any visible reinforcement, which could indicate that the wall was indeed completely rebuilt.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

- North Wall:** Badly sprung
- South Wall:** Badly cracked
- East Wall:** Cracked
- West Wall:** Must come down

Remarks & Recommendations:

- Building should come down.

Earthquake Reinforcement Survey Form 100 Church Street: Photographs



Earthquake Reinforcement Survey Form

79-81 East Bay Street

(Also 0-4 Tradd Street)



Lindsay-
Bulwinkle
Property

Presence of Visible Reinforcement:

North Façade:

3 anchor plates between 1st & 2nd floors

South Façade:

3 anchor plates between 1st & 2nd floors

1 anchor plate between 2nd floor & roof structure

East Façade:

5 anchor plates between 2nd floor & roof structure

West Façade:

Façade not visible (blocked by piazza)

**Date of
Construction:**
(1845, 1778-
1785)

Notes:

The structure at 79-81 East Bay (also 0-4 Tradd) Street exhibits one consistent form and shape of anchor plate. On the front (east) façade, five cross-shaped anchor plates can be seen spaced evenly across the façade between the second floor and the roof structure above. The same shape of plate appears on the south side façade. These plates are unevenly spaced and do not appear to be applied in any specific location pattern. Three plates are featured between the first and second floors, while only one plate appears between the second floor and roof structure above.

The consistent size and shape of the anchor plates on the structure seem to indicate that they were all installed at the same time and during the same reinforcement campaign. The 1886 damage assessment report indicates that these repairs were more than likely made following the earthquake.

There are no visible signs of deterioration on either of the anchor plates or the painted stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: 4 Tradd: Badly cracked

South Wall: 4 Tradd: Badly cracked

East Wall: 4 Tradd: Cracked

West Wall: 4 Tradd: Good

Remarks & Recommendations:

79-81 East Bay: Not in an insurable condition unless rebuilt from ground.

2 Tradd: A wreck

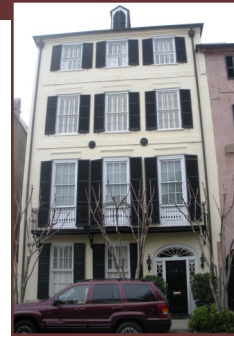
4 Tradd: Repair roof and rebuild chimneys from below roof; anchor and rebuild cracked portions of walls

Earthquake Reinforcement Survey Form
79-81 East Bay Street: Photographs



Earthquake Reinforcement Survey Form

83 East Bay Street



Wm. Stone
House
(Rainbow
Row)

Presence of Visible Reinforcement:

North Façade:

Façade not visible (attached to adjacent structure)

South Façade:

2 anchor plates between 2nd & 3rd floors

East Façade:

2 anchor plates between 1st & 2nd floors

2 anchor plates between 2nd & 3rd floors

West Façade:

Façade not visible

**Date of
Construction:**
(1784, restored
1941)

Notes:

The structure at 83 East Bay Street exhibits 4 anchor plates on the front (east) façade, all of the same circular shape and size—2 between the first and second floors, and 2 between the second and third floors. Each pair of anchor plates is centered horizontally between the window openings and vertically between their respective floors. The plates between the first and second floors are partially hidden by the placement of the iron balcony. On the south side façade, one circular anchor plate of the same style appears between the second and third floors closer to the front corner of the building. Further back towards the rear wall, the faint ghosting of an S-shaped plate underneath the stucco surface appears.

The 1886 damage assessment specifically recommends that the structure be anchored, more than likely between the east and west walls. The circular plates visible today are of a quality and form consistent with post-earthquake bolts. The S-shaped plates are an earlier form, and the presence of one at 83 East Bay may signify earlier reinforcement of the structure.

There are no visible signs of deterioration on either of the anchor plates or the painted stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good
South Wall: Good
East Wall: Badly cracked between openings
West Wall: Badly cracked between openings

Remarks & Recommendations:

Should be well anchored; rear building in good condition.

Earthquake Reinforcement Survey Form 83 East Bay Street: Photographs



Earthquake Reinforcement Survey Form

85 East Bay Street



Smith,
Desaussure &
Darrell
Building
(Rainbow
Row)

Presence of Visible Reinforcement:

North Façade:

- I anchor plate between 2nd & 3rd floors
- I anchor plate between 3rd & 4th floors
- I anchor plate between 4th floor & roof structure

South Façade:

Façade not visible (attached to adjacent structure)

East Façade:

4 anchor plates between 2nd & 3rd floors

West Façade:

Façade not visible

Notes:

The structure at 85 East Bay Street exhibits one consistent shape and form of anchor plate on both the north and east (front) façades. On the east façade, 4 circular, plain profile anchor plates are spaced evenly across the wall between the second and third floors. The north wall is anchored by one bolt in between each of the second, third, fourth floor, and roof structures. The bolts are all placed in the northeast corner of the structure, seemingly running directly behind the front façade wall.

The 1886 damage assessment report seems to suggest that the structure could only be completely taken down and rebuilt. Yet, the presence of reinforcement suggests that this wasn't the case and the building was in fact saved by reinforcing the remaining walls.

There are no visible signs of deterioration on either of the anchor plates or the painted stucco surface beneath the plates.

**Date of
Construction:**
(1785-1787,
restored 1930s,
1996-1997)

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

- North Wall:** Division good
- South Wall:** Must come down
- East Wall:** Must come down
- West Wall:** Must be taken down to second story & replaced

Remarks & Recommendations:

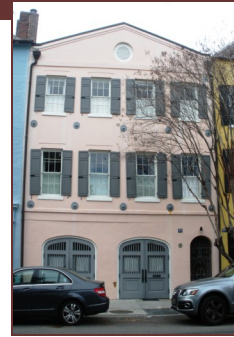
Only by rebuilding, now valueless.

Earthquake Reinforcement Survey Form 85 East Bay Street: Photographs



Earthquake Reinforcement Survey Form

91 East Bay Street



Inglis Arch
House
(Rainbow
Row)

**Date of
Construction:**
(1782, restored
1938)

Presence of Visible Reinforcement:

North Façade:

Façade not visible (attached to adjacent structure)

South Façade:

Façade not visible (attached to adjacent structure)

East Façade:

5 anchor plates between 1st & 2nd floors

5 anchor plates between 2nd & 3rd floors

1 anchor plate between 3rd floor & roof structure

West Façade:

Façade not visible

Notes:

The front (east) façade of the structure at 91 East Bay Street exhibits 11 anchor plates total. There are 5 plates each in between the first and second floors & the second and third floors, respectively. The circular plates feature a decorative, moulded profile and are spaced evenly across the façade between their respective floors. In between the third floor and the roof structure, one flat, circular plate appears in the upper right-hand corner of the façade.

The 1886 damage assessment for the structure at 91 East Bay Street provides little information regarding the post-earthquake repairs to the building. Whether the building simply suffered little damage or just was not assessed until after repairs were already made can not be determined. However, the difference in form and attachment method between the types of plates on the façade may indicate two separate reinforcement campaigns.

In general, the anchor plates appear to be in good condition. There is slight visible evidence discoloration due to rusting beneath the plates between the first and second floors on the stucco surface.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good

South Wall: Good

East Wall: Good

West Wall: Good

Remarks & Recommendations:

Rebuild chimneys

Earthquake Reinforcement Survey Form
91 East Bay Street: Photographs



Earthquake Reinforcement Survey Form

95 East Bay Street



Othniel Beale
Houses
(Rainbow
Row)

**Date of
Construction:**
(1740, restored
1932-1936)

Presence of Visible Reinforcement:

North Façade:

Façade not visible (attached to adjacent structure)

South Façade:

Façade not visible (attached to adjacent structure)

East Façade:

4 anchor plates between 2nd & 3rd floors

4 anchor plates between 3rd & 4th floors

1 anchor plate between 4th floor & roof structure

West Façade:

Façade not visible

Notes:

The front (east) façade of the structure at 95 East Bay Street exhibits 9 anchor plates total. There are 4 plates each in between the second and third floors & the third and fourth floors, respectively. The circular plates are of a relatively plain, flat profile and are spaced evenly across the façade between their respective floors. The plates between the second and third floors have been installed within the belt course on the stucco facade. In between the fourth floor and the roof structure, one small circular plate (same in profile as the others) appears in the upper left-hand corner of the façade. It can be seen just above the left window.

The 1886 damage assessment indicates that the structure was damaged and recommends repairs to the east and west walls, but does not specifically recommend that the structure be anchored. However, the circular plates visible today are of a quality and form consistent with post-earthquake bolts.

There are no visible signs of deterioration on either of the anchor plates or the painted stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Division good
South Wall: Division good
East Wall: Badly cracked over openings
West Wall: Badly cracked

Remarks & Recommendations:

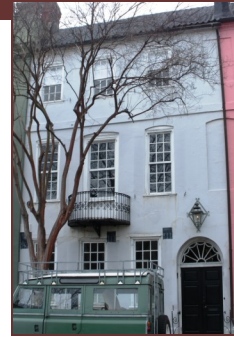
Rebuild chimneys; repair east and west walls

Earthquake Reinforcement Survey Form
95 East Bay Street: Photographs



Earthquake Reinforcement Survey Form

97 East Bay Street



Othniel Beale
Houses
(Rainbow
Row)

Presence of Visible Reinforcement:

North Façade:

Façade not visible (attached to adjacent structure)

South Façade:

Façade not visible (attached to adjacent structure)

East Façade:

2 anchor plates between 2nd & 3rd floors

West Façade:

Façade not visible

**Date of
Construction:
(1740, restored
1932-1936)**

Notes:

The front (east) façade of the structure at 97 East Bay Street shows evidence of some anchoring between the second and third floors. Contrary to the typical anchor plates, however, all that is visible is two small bolts in the stucco belt course. The bolts may suggest the use of some variation on bond iron to reinforce the structure. Or perhaps the anchor plates themselves are hidden beneath the belt course in an attempt to preserve the aesthetic of the façade and hide reinforcement.

The 1886 damage assessment indicates that the structure was damaged and recommends repairs to the east wall, but does not specifically recommend that the structure be anchored.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Division good
South Wall: Division good
East Wall: Badly cracked
West Wall: Good

Remarks & Recommendations:

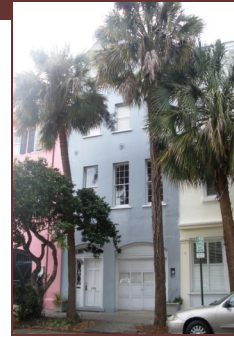
Rebuild chimneys; repair roof and east wall and plaster

Earthquake Reinforcement Survey Form
97 East Bay Street: Photographs



Earthquake Reinforcement Survey Form

103 East Bay Street



Joseph Dulles
House

Presence of Visible Reinforcement:

North Façade:

Façade not visible (attached to adjacent structure)

South Façade:

Façade not visible (attached to adjacent structure)

East Façade:

3 anchor plates between 2nd & 3rd floors

4 anchor plates between 3rd & 4th floors

West Façade:

Façade not visible

**Date of
Construction:**
(1787, renovated
1930s)

Notes:

The front (east) façade of the structure at 103 East Bay Street exhibits 9 anchor plates total. There are 3 plates in between the second and third floors & 4 plates between the third and fourth floors. All of the circular plates are of a relatively plain, flat profile with the exception of the center plate between the second and third floors. This plate has the only moulded profile. Each row of anchor plates is spaced evenly across their respective walls.

The 1886 damage assessment indicates that the structure was damaged, especially the east and west walls. The assessment recommended that both walls be replaced entirely. If the walls were indeed replaced, the visible earthquake bolts could have been added as additional reinforcement to the new walls.

There are no visible signs of deterioration on any of the anchor plates or the painted stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Badly Sprung
South Wall: Badly Sprung
East Wall: Gables out, now boarded
West Wall: Gables out, now boarded

Remarks & Recommendations:

East and west walls should be replaced; kitchen not insurable

Earthquake Reinforcement Survey Form
103 East Bay Street: Photographs



Earthquake Reinforcement Survey Form

107 East Bay Street



John Blake
Building

Presence of Visible Reinforcement:

North Façade:

- 2 anchor plates between 1st & 2nd floors
- 2 anchor plates between 2nd & 3rd floors
- 1 anchor plate between 3rd floor & roof structure

South Façade:

Façade not visible (attached to adjacent structure)

East Façade:

None visible

West Façade:

- 1 anchor plate between 3rd floor & roof structure
- 2 anchor plates between roof structure and gable

**Date of
Construction:**
(1792, present
façade circa 1887
-1890)

Notes:

The front (east) façade of the structure at 107 East Bay Street exhibits 8 anchor plates total. On the side (north) façade, there are 2 anchor plates each between the first & second and the second & third floors, respectively. There is 1 plate between the third floor and roof structure on this façade. On the rear (west) façade, there is 1 anchor plate between the third floor and the roof structure, and 2 plates between the roof structure and parapet wall. All of the circular plates are of a relatively plain profile. However, the plates on the north wall have a slight convex shape, while the ones on the rear façade are completely flat. There seems to be no effort to place the plates in any particular pattern on the façade.

The 1886 damage assessment for 107 East Bay does not have an remarks for each specific wall. However, the comments indicate that the structure had been previously anchored but remained unsafe. The absence of reinforcement on the front (east) façade supports the claim that the wall dates to post-1886.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall:

South Wall:

East Wall:

West Wall:

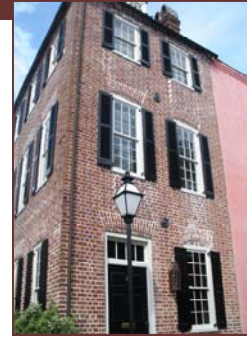
Remarks & Recommendations:

Has been anchored but is not safe; should come down

Earthquake Reinforcement Survey Form
107 East Bay Street: Photographs



Earthquake Reinforcement Survey Form 4 Elliott Street



David Saylor
House

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

1 anchor plate between 1st & 2nd floors
1 anchor plate between 2nd & 3rd floors

East Façade:

Façade not visible (attached to adjacent structure)

West Façade:

None visible

**Date of
Construction:**
(Pre-1779,
restored 1986)

Notes:

The structure that occupies 4 Elliott Street exhibits two anchor plates total. Visible on the front (south) façade, there is 1 plate each between the first and second & second and third floors, respectively. Each plate is centered on the wall between the windows on their respective floor. The two plates are identical with a decorative, moulded profile—typical of the post-1886 bolts.

The 1886 damage assessment indicates that both the north and south walls were badly damaged in the earthquake. It was specifically recommended that the south wall be anchored. Although the north wall is not visible because it faces the back of the property, it is possible that the anchors visible on the south extend all the way through.

There are no signs of deterioration of the anchor plates or the brick surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Slightly cracked
South Wall: Badly cracked
East Wall: Division
West Wall: Good

Remarks & Recommendations:

Anchor south wall at each floor and repair cracks; rear building not insurable

Earthquake Reinforcement Survey Form
4 Elliott Street: Photographs



Earthquake Reinforcement Survey Form

10 Elliott Street



News &
Courier
Printing
Office

Presence of Visible Reinforcement:

North Façade:

None visible

South Façade:

None visible

East Façade:

2 anchor plates between 2nd floor & roof structure

West Façade:

Façade not visible (attached to adjacent structure)

**Date of
Construction:
(Unknown)**

Notes:

The structure that occupies 10 Elliott Street exhibits two anchor plates total, both on the east façade. One tie rod is installed between the first and second floors, and the other sits between the second and third floors. The west façade is attached to an adjacent structure and not visible, so presumably these tie rods only extend into the floor frame 2-3 joist bays. The plates themselves are a circular form with a slightly convex shape, plain in profile.

The 1886 damage assessment indicates that all four walls of the structure were badly damaged in the earthquake. All walls were badly cracked and uninsurable. Thus, the walls were presumably rebuilt, at least partially. It is difficult to determine without further investigation if the plates on the south façade belong to an early reinforcement campaign or 1886 earthquake bolts.

There are no signs of deterioration of the anchor plates or related deterioration of the brick surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Bad
South Wall: Bad
East Wall: Bad
West Wall: Bad

Remarks & Recommendations:

Walls on all sides badly cracked; not insurable

Earthquake Reinforcement Survey Form
10 Elliott Street: Photographs



Earthquake Reinforcement Survey Form

20 Elliott Street



George Gibbs
Tenement

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

1 anchor plate between 1st & 2nd floor
1 anchor plate between 2nd & 3rd floor

East Façade:

Façade not visible (attached to adjacent structure)

West Façade:

Façade not visible (attached to adjacent structure)

**Date of
Construction:**
(Circa 1802,
restored mid-
20th century)

Notes:

The structure that occupies 20 Elliott Street exhibits two anchor plates total, both on the front (south) façade. One tie rod is installed between the first and second floors, and the other sits between the second and third floors. Both tie rods are installed on the far right-hand corner of the façade. The rear (north) façade is not visible, thus it is impossible to determine if the rods go wall-to-wall or only extend 8-10 feet into the structure. Both the east and west walls are attached to an adjacent structure and not visible. The plates themselves are a circular form with a very flat, plain profile.

The 1886 damage assessment indicates that both the north and south walls were down after the earthquake and anchoring was specifically recommended for repair. Thus, the reinforcement visible today was more than likely put in place post-1886. Interestingly though, the plates at 20 Elliott are slightly larger than most other earthquake bolts and typical of earlier reinforcement forms.

There are no signs of deterioration of the anchor plates or related deterioration of the stucco surface surrounding the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Down
South Wall: Down
East Wall: Division
West Wall: Division

Remarks & Recommendations:

Repair north and south walls and anchor; put on new tin roof; rear buildings old & dilapidated, not insurable.

Earthquake Reinforcement Survey Form
20 Elliott Street: Photographs



Earthquake Reinforcement Survey Form

22 Elliott Street



Gibbs-
Blackwood
Tenement

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

3 anchor plates between 1st & 2nd floor
3 anchor plates between 2nd & 3rd floor

East Façade:

Façade not visible (attached to adjacent structure)

West Façade:

None visible

**Date of
Construction:**
(1793-1800,
restored 1947,
1990)

Notes:

The structure that occupies 22 Elliott Street exhibits six anchor plates total, all on the front (south) façade. Three tie rods each are installed between the first and second & second and third floors, respectively. On the facade, the anchor plates are concentrated towards the mid-section of the structure in between their respective floors. The rear (north) façade is not visible, thus it is impossible to determine if the rods go wall-to-wall or only extend 8-10 feet into the structure. The east wall is attached to an adjacent structure and not visible. The plates themselves exhibit one of the less common shapes—a rectangular shaped plate oriented vertically.

The 1886 damage assessment indicates that both the north and south walls were damaged after the earthquake and anchoring was specifically recommended for repair. Thus, the reinforcement visible today was more than likely put in place post-1886. The rear (north) façade is not visible, thus it is impossible to determine if the rods go wall-to-wall or only extend 8-10 feet into the structure.

Some discoloration due to corrosion is visible beneath the plates installed between the second and third floors. However, no serious signs of deterioration are evident.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Badly cracked, top off
South Wall: Slightly cracked
East Wall: Division
West Wall: Division, gable down

Remarks & Recommendations:

Rebuild west gable; repair north and south walls and anchor at each floor;
take off tile and put on tin; rebuild chimneys

Earthquake Reinforcement Survey Form
22 Elliott Street: Photographs



Earthquake Reinforcement Survey Form

60 Meeting Street



60 Meeting
Street

Presence of Visible Reinforcement:

North Façade:

None visible

South Façade:

3 anchor plates between 2nd & 3rd floor

East Façade:

None visible

West Façade:

3 anchor plates between 2nd & 3rd floors (2 on main house,
1 on addition)

**Date of
Construction:
(1771 w. 1893
alterations)**

Notes:

The structure that occupies 60 Meeting Street exhibits six anchor plates total. Three anchor plates can be seen on the south side façade between the second and third floors. These plates are placed within the stucco belt course and centered in between window openings. On the west facade, two anchor plates can be seen on the main house between the second and third floors, one in each corner of the floor structure. Because corresponding plates are not seen on the opposite walls, it is assumed that each of these tie rods extend only partially into the structure. The plates themselves are circular in shape with a plain, flat profile—typical of post-1886 anchor plates.

The 1886 damage assessment specifically recommends anchoring the damaged walls. Interestingly, the west wall was noted as in good condition. Yet, this wall exhibits three out of the six anchor plates visible today.

Some stucco discoloration due to corrosion is visible beneath the plates installed on the south façade. No other signs of deterioration are visible on the other plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Slight cracked over openings

South Wall: Badly cracked

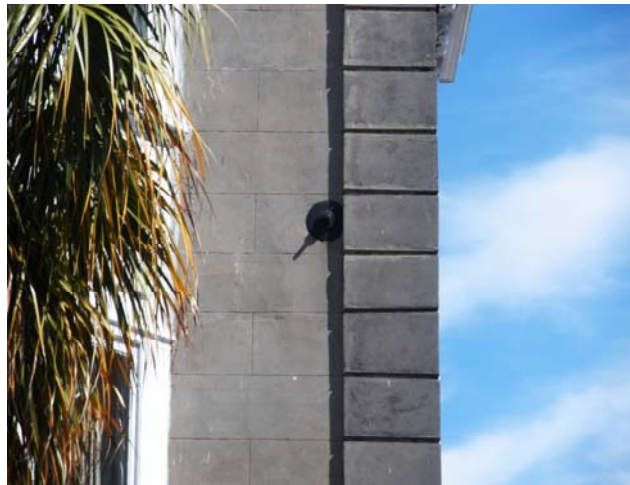
East Wall: Badly cracked

West Wall: Good

Remarks & Recommendations:

Should be securely anchored

Earthquake Reinforcement Survey Form
60 Meeting Street: Photographs



Earthquake Reinforcement Survey Form

8 St. Michael's Alley



Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

None visible

East Façade:

None visible

West Façade:

2 anchor plates between 1st & 2nd floor

James L.
Petigru Law
Office

**Date of
Construction:**
(1848, restored
1918, 1983-84)

Notes:

The structure that occupies 8 St. Michael's Alley exhibits only two anchor plates total, both on the side (west) façade between the first and second floors. Centered horizontally in the wall, both plates are circular in shape with a slightly moulded profile. Interestingly, the plates are placed at a location on the façade that aligns with the bottom of the window opening on the same floor. If the bolts are truly still installed in the wall, they may be visible above the floor on the interior. It is difficult to determine more about the installation of these bolts without further investigation.

The 1886 damage assessment indicates that the north and south walls were damaged and specifically recommends anchoring. Interestingly, the west wall was noted as good. Yet, this wall exhibits the only visible reinforcement today.

Some stucco discoloration due to corrosion is visible beneath the plates installed on the west façade. The plates themselves also seem to show a significant amount of surface corrosion and are in need of repainting and sealing.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Cracked
South Wall: Cracked
East Wall: Good
West Wall: Good

Remarks & Recommendations:

Rebuild parapet and anchor walls

Earthquake Reinforcement Survey Form
8 St. Michael's Alley: Photographs



Earthquake Reinforcement Survey Form

11 St. Michael's Alley



11 St.
Michael's
Alley

Presence of Visible Reinforcement:

North Façade:

2 anchor plates between 1st & 2nd floors

South Façade:

None visible

East Façade:

1 anchor plate between 1st & 2nd floors (limited visibility)

West Façade:

3 anchor plates between 1st & 2nd floors

**Date of
Construction:
(1780s)**

Notes:

The structure that occupies 11 St. Michael's Alley exhibits six anchor plates total. All of the anchor plates are installed between the first and second floors—3 on the west wall, 2 on the north wall, and 1 on the east wall. The porch and entrance gate limits visibility on the east, thus more reinforcement could in fact be present. All of the plates are of the same shape & form—the less common, star-shape. This uniform size & shape also indicates the likelihood that all were installed during the same reinforcement campaign.

The 1886 damage assessment indicates that all four walls were cracked as a result of the earthquake. It was also noted that the walls were already anchored. This could mean that the walls were either anchored post-earthquake but before the assessment, or were in fact reinforced pre-1886 and subsequently only cracked in the earthquake. It is difficult to determine more without further investigation.

There are no signs of deterioration of the anchor plates or related deterioration of the stucco surface surrounding the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Cracked
South Wall: Cracked
East Wall: Cracked
West Wall: Cracked

Remarks & Recommendations:

Rebuild south and east walls from foundations; walls anchored

Earthquake Reinforcement Survey Form
II St. Michael's Alley: Photographs



Earthquake Reinforcement Survey Form

6 Tradd Street



John Fabre Jr.
House

**Date of
Construction:**
(1788; restored
before 1920)

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

2 anchor plates between 2nd & 3rd floors

East Façade:

2 anchor plates between 1st & 2nd floors

1 anchor plate between 2nd & 3rd floors

2 anchor plates between 3rd floor & roof structure

West Façade:

2 anchor plates between 1st & 2nd floors

2 anchor plates between 2nd & 3rd floors

2 anchor plates between 3rd floor & roof structure

Notes:

The structure that occupies 6 Tradd Street exhibits eleven anchor plates total. Two anchor plates are visible on the front (south) façade, centered on the wall between the second and third floors, and appear to extend only 8-10 feet into the structure. The side facades (east and west) appear to have corresponding anchor plates between both the first and second & second and third floors, respectively, indicating that these are full-length, wall-to-wall tie rods. All of the plates are of the same shape & form—small, circular, and flat in profile. This uniform size & shape also indicates the likelihood that all were installed during the same reinforcement campaign.

The 1886 damage assessment indicates that the entire structure was badly cracked as a result of the earthquake. It was also noted that the east and west walls were already anchored. This could mean that the walls were either anchored post-earthquake but before the assessment, or were in fact reinforced pre-1886 and subsequently only cracked in the earthquake. It is difficult to determine more without further investigation.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Badly cracked
South Wall: Badly cracked
East Wall: Division good
West Wall: Slightly cracked

Remarks & Recommendations:

East and west walls have been anchored; repair cracked portions of north and south walls; rebuild chimneys and repair kitchen

Earthquake Reinforcement Survey Form
6 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

12 Tradd Street



Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

2 anchor plates between 2nd & 3rd floors

East Façade:

None visible

West Façade:

Façade not visible (attached to adjacent structure)

Lightwood-
Sommers
House

**Date of
Construction:**
(1748-50; rebuilt
1789-96)

Notes:

The structure that occupies 12 Tradd Street exhibits only two anchor plates. Both plates are visible on the front (south) façade, centered on the wall between the second and third floors, and presumably extend only 8-10 feet into the structure. Both plates on the front façade are of the same standard shape—circular with a relatively plain profile. The attachment method is the same indicating these plates were installed together at the same time.

The 1886 damage assessment indicates that at the time of observation all four walls were in good condition and the structure has been repaired. This could mean that the walls were either anchored post-earthquake but before the assessment took place, or were in fact reinforced pre-1886 and subsequently undamaged in the earthquake. It is difficult to determine more without further investigation.

There are no signs of deterioration of the anchor plates or the brick surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good

South Wall: Good

East Wall: Good

West Wall: Good

Remarks & Recommendations:

Has been put in good repair

Earthquake Reinforcement Survey Form
12 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

14 Tradd Street



Humphrey
Sommers
Tenement

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

3 anchor plates between 1st & 2nd floors
3 anchor plates between 2nd & 3rd floors

East Façade:

Façade not visible (attached to adjacent structure)

West Façade:

Façade not visible (attached to adjacent structure)

Date of
Construction:
(Circa 1770)

Notes:

The structure that occupies 14 Tradd Street exhibits a total of six anchor plates, all visible on the front (south) façade. Three plates each are centered on the wall between the first and second floors & the second and third floors, respectively. Presumably, these façade tie rods extend only 8-10 feet into the structure. Each of the anchor plates are of the same standard shape—circular with a relatively plain and slightly convex profile. This likely indicates that these tie rods were all part of the same reinforcement campaign.

The 1886 damage assessment indicates that at the time of observation all four walls were in good condition and the structure has been repaired. This could mean that the walls were either anchored post-earthquake but before the assessment took place, or were in fact reinforced pre-1886 and subsequently undamaged in the earthquake. It is difficult to determine more without further investigation.

There are no signs of deterioration of the anchor plates or the stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good
South Wall: Good
East Wall: Good
West Wall: Good

Remarks & Recommendations:

Has been put in good order, OK

Earthquake Reinforcement Survey Form
14 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

16 Tradd Street



Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

2 anchor plates between 1st & 2nd floors
2 anchor plates between 2nd & 3rd floors

East Façade:

Façade not visible (attached to adjacent structure)

West Façade:

Façade not visible (attached to adjacent structure)

Humphrey
Sommers
Tenement

Date of
Construction:
(Circa 1770)

Notes:

The structure that occupies 16 Tradd Street exhibits a total of four anchor plates, all visible on the front (south) façade. Two plates each are installed between the first and second & the second and third floors, respectively. The anchor plates are approximately six feet apart and aligned with the left side of the façade. Presumably, these façade tie rods extend only 8-10 feet into the structure. Each of the anchor plates exhibits the same shape & form as those found on number 14 Tradd Street—circular with a relatively plain and slightly convex profile. This likely indicates that these tie rods were all part of the same reinforcement campaign.

The 1886 damage assessment indicates that at the time of observation all four walls were in good condition and the structure has been repaired. In the damage assessment, numbers 14 and 16 Tradd are assessed together. For the purpose of this survey, the structures have been surveyed individually.

There are no signs of deterioration of the anchor plates or the stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good
South Wall: Good
East Wall: Good
West Wall: Good

Remarks & Recommendations:

Has been put in good order, OK

Earthquake Reinforcement Survey Form
16 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

26 Tradd Street



Robert Ewing
House

**Date of
Construction:**
(Post-1778,
renovated 1990s)

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

1 anchor plate between 2nd & 3rd floors

East Façade:

None visible

West Façade:

Façade not visible (attached to adjacent structure)

Notes:

The structure that occupies 26 Tradd Street exhibits only one visible anchor plate. The anchor plate can be seen in the center of the front (south) façade in between the second and third floors. Presumably, this tie rod only extends 8-10 feet into the structure. The plate itself is of an unusual cross-shape. The small vertical arm is threaded onto the rod first and sits flush with the wall surface. The horizontal arm is threaded onto the rod second and formed to fit flush on top of the other arm.

The 1886 damage assessment indicates that at the time of observation all four walls were in good condition and the structure had been repaired. This could mean that the walls were either anchored post-earthquake but before the assessment took place, or were in fact reinforced pre-1886 and subsequently undamaged in the earthquake. It is difficult to determine more without further investigation.

There are no signs of deterioration of the anchor plates or the stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good

South Wall: Good

East Wall: Good

West Wall: Good

Remarks & Recommendations:

Has been repaired, OK

Earthquake Reinforcement Survey Form
26 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

28 Tradd Street



Dr. Peter J.
Moore
Tenement

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

2 anchor plates between 1st & 2nd floors
2 anchor plates between 2nd & 3rd floors

East Façade:

Façade not visible (attached to adjacent structure)

West Façade:

2 anchor plates between 1st & 2nd floors
3 anchor plates between 2nd & 3rd floors

Date of
Construction:
(Circa 1790)

Notes:

The structure that occupies 28 Tradd Street exhibits nine anchor plates total. On the front (south) façade, there are two anchor plates each between the first and second & the second and third floors. The plates are centered on the wall in between the window openings. All four plates on the south façade are of the same shape & form—circular, with a decorative, moulded profile—indicating they were installed at the same time. On the side (west) façade, there is one anchor plate each in between the first and second & second and third floors. The upper plate is of the same shape & form as those on the front. However, the lower anchor plate is a more plain circular form with flat profile. On this façade, the plates are installed close the front wall of the structure.

The 1886 damage assessment indicates that most of the walls were in good condition following the earthquake, except for the south (front) façade. At this time it was only recommended to repair the south wall, an indication that the front façade anchor plates are indeed post-1886.

There are no signs of deterioration of the anchor plates or the brick surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Good
South Wall: Top part down
East Wall: Good
West Wall: Good

Remarks & Recommendations:

Repair top of south wall; rebuild chimneys & repair kitchen walls

Earthquake Reinforcement Survey Form
28 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

32 Tradd Street



Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

3 anchor plates between 1st & 2nd floors
3 anchor plates between 2nd & 3rd floors
1 anchor plate between the 3rd floor & roof structure

East Façade:

None visible

West Façade:

Façade not visible (attached to adjacent structure)

32 Tradd
Street

Date of
Construction:
(Unknown)

Notes:

The structure that occupies 32 Tradd Street exhibits six anchor plates total, all visible on the front (south) façade. There are three plates each between the first and second & the second and third floors. Between the third and fourth floors, there is one single anchor plate centered on the wall underneath the center window opening. All anchor plates on the south façade exhibit the same shape & form—a relatively small, circular form with a stepped profile—an indication that they were likely installed at the same time.

The 1886 damage assessment indicates that both the north and south walls were damaged in the earthquake. It was recommended to completely rebuild both walls. It is difficult to determine if the south wall, which shows visible reinforcement today, was in fact rebuilt. However, it can be assumed that the tie rods were either installed when the wall was rebuilt as additional reinforcement, or were installed to repair the existing wall to prevent a rebuild.

There are no signs of deterioration of the anchor plates or the stucco surface beneath the plates.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Badly cracked
South Wall: Top part down
East Wall: Division
West Wall: Division

Remarks & Recommendations:

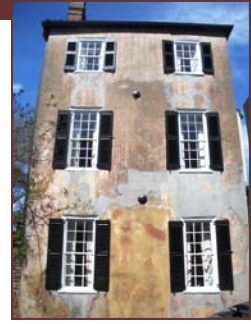
Rebuild south wall & repair cracks; rebuild north wall; repair roof and rebuild chimneys; rear building is a wreck

Earthquake Reinforcement Survey Form
32 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

46 Tradd Street



James
Vanderhorst
House

**Date of
Construction:**
(1770, renovated
1927)

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

I anchor plate between 1st & 2nd floors
I anchor plate between 2nd & 3rd floors

East Façade:

None visible

West Façade:

None visible

Notes:

The structure that occupies 46 Tradd Street exhibits two anchor plates total. Visible on the front (south) façade, there is I plate each between the first & second, and the second & third floors, respectively. Each plate is centered on the wall between the windows on their respective floor. The two circular plates are identical with a relatively plain, convex profile—common of the post-1886 bolts.

The 1886 damage assessment indicates that both the north and south walls were badly cracked following the earthquake. It was specifically recommended that the north and south walls be anchored. Thus, although the north wall is not visible because it faces the back of the property, it is possible that the anchors visible on the south extend all the way through.

There are no signs of deterioration of the anchor plates or the stucco surface beneath the plates

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Cracked at opening
South Wall: Cracked over openings
East Wall: Good
West Wall: Good

Remarks & Recommendations:

Anchor north and south walls

Earthquake Reinforcement Survey Form
46 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

54 Tradd Street



William
Vanderhorst
House

**Date of
Construction:**
(1770, restored
1930s, 1996)

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

- 1 anchor plate between 2nd & 3rd floors
- 1 anchor plate between 3rd floor & roof structure

East Façade:

- 2 anchor plates between 2nd & 3rd floors
- 2 anchor plates between 3rd floor & roof structure

West Façade:

- 2 anchor plates between 2nd & 3rd floors
- 2 anchor plates between 3rd floor & roof structure

Notes:

The structure that occupies 54 Tradd Street exhibits ten anchor plates total. Two of them are visible on the front (south) façade—one plate between the first and second floors & one plate between the second and third floors, respectively. Each plate is installed between the two left-most windows on the respective floor. On both side façades there are also two plates each between the first and second & the second and third floors, respectively. The placement of the anchor plates indicates that these east-west ties are full length, wall-to-wall tie rods. The façade ties more than likely only extend 8-10 feet into the structure and anchor to a floor joist.

The 1886 damage assessment indicates that both the north and south walls were badly cracked following the earthquake, and that the east and west walls were good. Recommendations for anchoring were made, but certain walls were not specified. This could mean that the east and west walls were either anchored post-earthquake but before the assessment, or were in fact reinforced pre-1886 and subsequently only cracked in the earthquake. It is difficult to determine more without further investigation. However, the consistent form & shape of the anchor plates seems to indicate that they were all installed as a part of the same reinforcement campaign.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Cracked
South Wall: Cracked at openings
East Wall: Good
West Wall: Good

Remarks & Recommendations:

Repair walls at openings and properly anchor

Earthquake Reinforcement Survey Form
54 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

58 Tradd Street



Cleland-Wells
House

**Date of
Construction:**
(1760, renovated
1935-36)

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

2 anchor plates between 1st & 2nd floors
2 anchor plates between 2nd & 3rd floors

East Façade:

2 anchor plates between 1st & 2nd floors
2 anchor plates between 2nd & 3rd floors

West Façade:

Façade not visible (blocked by piazza)

Notes:

The structure that occupies 58 Tradd Street exhibits eight anchor plates total. Visible on the front (south) façade, there are two plates each between the first and second & the second & third floors, respectively. Each plate is centered on the wall between the windows on the respective floor. On the side (east) façade there are also two plates each between the first and second & the second and third floors, respectively. The anchor plates correspond to two tie rods that are installed directly behind & parallel to the front façade. The other two ties appear to be installed closer to the rear wall. All of the circular anchor plates are identical with a relatively plain, convex profile—common of the post-1886 tie rods.

The 1886 damage assessment indicates that both the north and south walls were badly cracked along with the east wall following the earthquake. It was specifically recommended that the north and south walls be anchored. Thus, although the north wall is not visible because it faces the back of the property, it is possible that the anchors visible on the south extend all the way through, though likely not. The west wall is also not visible because it is blocked by the piazza, preventing the confirmation of the east-west tie rods as full length, wall-to-wall tie rods.

Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Badly cracked
South Wall: Badly cracked
East Wall: Cracked
West Wall: Good

Remarks & Recommendations:

Walls need anchoring north and south

Earthquake Reinforcement Survey Form
58 Tradd Street: Photographs



Earthquake Reinforcement Survey Form

60 Tradd Street



Dr. William
Cleland
House

Presence of Visible Reinforcement:

North Façade:

Façade not visible

South Façade:

2 anchor plates between 1st & 2nd floors
2 anchor plates between 2nd & 3rd floors

East Façade:

2 anchor plates between 1st & 2nd floors
2 anchor plates between 2nd & 3rd floors

West Façade:

2 anchor plates between 1st & 2nd floors

**Date of
Construction:
(Circa 1727)**

Notes:

The structure that occupies 60 Tradd Street exhibits twelve anchor plates total. Four of them are visible on the front (south) façade—two plates between the first and second floors & two plates between the second and third floors, respectively. Each plate is centered on the wall between the windows on the respective floor. On both side façades there are also two plates each between the first and second & the second and third floors, respectively. The placement of the anchor plates indicates that these east-west ties are full length, wall-to-wall tie rods. The façade ties more than likely only extend 8-10 feet into the structure and anchor to a floor joist. All of the circular plates are identical with a relatively plain, convex profile—common of the post-1886 bolts.

The 1886 damage assessment indicates that both the north and south walls were badly cracked following the earthquake. However, the only recommendation was to anchor the south wall. The consistent shape & form of the plates seems to indicate that all of the ties were part of the same reinforcement campaign. But it is difficult to determine based on the damage assessment report alone.

There are no signs of deterioration of the anchor plates or the stucco surface beneath the plates.

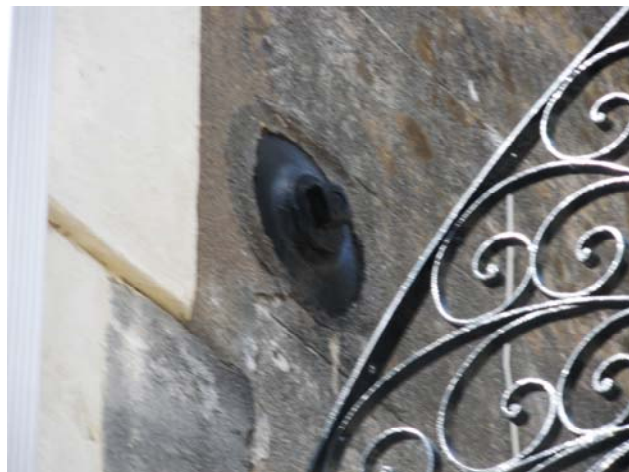
Summary of 1886 Record of Earthquake Damages & Repair Recommendations:

North Wall: Badly cracked
South Wall: Badly cracked
East Wall: Good
West Wall: Good

Remarks & Recommendations:

Anchor and repair south wall

Earthquake Reinforcement Survey Form
60 Tradd Street: Photographs



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