Seismic retrofitting of unreinforced masonry buildings

by

Chase Young

B.S., Kansas State University, 2023

A REPORT

submitted in partial fulfillment of the requirements for the degree

MASTER OF SCIENCE

GE Johnson Department of Architectural Engineering and Construction Science
Carl R. Ice College of Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

2023

Approved by:

Major Professor
Bill Zhang

Copyright

Abstract

Masonry is one of the oldest building materials to exist. It is seen throughout history in many well-known and historical structures all around the world. Masonry is strong in compression but weak in tension. With it being such a prominent and old material, there are many existing structures that lack proper reinforcement. Because of this reason, many of these buildings are at risk of failure under earthquakes, which can cause serious structural damage, injury, and even fatalities. The failure of these structures can be prevented by newly developed technologies. There are many retrofitting techniques that can be used for unreinforced masonry. This report covers just a few of the methods that are available today, including grout and epoxy injections, carbon fiber reinforced polymers (CFRP), parapet bracing, out-of-plane wall bracing, shotcrete, and diaphragm ties. These methods have been proven to improve the stability and strength of unreinforced masonry buildings during a seismic event. This report seeks to summarize these methods and to discuss some technical details on how to use them to retrofit existing unreinforced masonry structures.

Table of Contents

List of Figures

List of Tables

Acknowledgements

I would like to thank all of my family, friends and professors that have helped me reach this point of my student career and for all of the support along the way.

Chapter 1: Introduction

History of Masonry

Masonry is one of the oldest building materials to exist, and ancient masonry buildings can be found across the world with some still standing to this day. Around 8000 BC in Mesopotamia, beehive domes were built which is the first known use of adobe. Masonry was seen throughout ancient Greece. Some of the most famous masonry structures still stand in the area, such as the Parthenon. In ancient Rome many historical structures were built out of masonry, including the Pantheon and the Roman Coliseum. The Pantheon is considered by many as one of the world’s most beautiful buildings. The dome has a 140-foot diameter that is constructed entirely from masonry with a hole at the top to allow for light to enter the structure. There are many other prominent historical structures that were also constructed from masonry, such as the Great Pyramids of Egypt (Figure 1), the Hagia Sophia, and the Great Wall of China. Masonry as a great construction material has stood the test of time throughout the world, and is still a common construction material now.

Figure 1: Great Pyramid of Giza

Source: (Jarus, 2022)

Prior to existence of modern building codes, there had been many unreinforced masonry buildings throughout the United States. One example is the Monadnock Building in Chicago, a 16-story building whose walls are 6-foot-thick at the base tapering up to one foot thick at the top. The tallest unreinforced masonry building in the US is the Philadelphia City Hall, with a staggering height of 548 feet. Construction of Philadelphia City Hall first started in 1871 and ended in 1901. Over 88 million bricks were used for the building. But this isn't even the tallest unreinforced masonry structure. The tallest structure is the Washington Monument (Figure 2), in Washington D.C., with a height of 555 feet and 5 inches. It has the title of the world's tallest unreinforced masonry structure. There are many unreinforced masonry buildings throughout the United States, which were constructed in the late 19th century to early 20th century, from big cities to small towns.

Figure 2: Washington Monument
Source: (U.S. Department of the Interior, 2022)

Historical Failures

With masonry being such a prominent building material, it has seen many failures throughout history, especially during major earthquake events. These failures can cause severe property damage, injuries, and even deaths. The damage that occurs from a seismic event is often so severe that the building must be demolished. This can be problematic for buildings of historic significance. This was seen after the 1989 Loma Prieta earthquake when damage in the Pacific Avenue Historic District in Santa Cruz was so extensive that the downtown area was removed from the National Register of Historic Places. In this Historic District, 52% of the old brick buildings were so badly damaged that they were quickly demolished, and another 16% were "red-tagged" (Reitherman et al., 2009). When a building is red-tagged it is no longer safe for occupation, this can result in the building having a long-term vacancy or being demolished. This is just one example of the historical earthquakes that have occurred and caused extensive damage, while Table 1 lists some other events.

Earthquake	**Magnitude**	**Damage**
1886 Charleston, SC	7.7	82% of buildings suffered more than minor damage, 7% collapsed or were demolished
1925 Santa Barbara, SoCal	6.2	40% of URM buildings were severely damaged or collapsed, was motivation California to adopt seismic design ideas from Japan
1933 Long Beach, SoCal	6.3	54% of URM buildings had damage that ranged from significant wall destruction to complete collapse
1983 Coalinga, Central Cal.	6.2	37 URM buildings only 1 escaped damage, 60% were damaged to the extent of half of their walls ruined, up to complete collapse
1983 Borah Peak, Challis, Idaho	7.3	Only earthquake-related fatalities happened when a URM wall fell on 2 children; 8 buildings required demolition; because of the size of the town, the damage constitutes a large disaster
1989 Loma Prieta, NorCal	7.1	374 of the 2400 URM buildings (16%) experienced damage severe enough to require that they be vacated
2003 San Simeon, Central Cal.	6.5	Of 53 URM buildings, none of the 9 that had been retrofitted experienced major damage, many of the others were damaged extensively that they were demolished

Table 1: Historical Failures of Unreinforced Masonry Buildings
Source: (Reitherman et al., 2009)

Masonry can fail under both in-plane loading and out-of-plane loading. There are several ways that masonry can fail under in-plane loading. There is sliding shear failure, diagonal shear, rocking, and toe crushing. Sliding shear failure (Figure 3a) occurs when the wall will either slide off the foundation or shear directly from the mortar joints and is caused from low vertical loads and poor mortar (Jamal, 2017). Diagonal shear (Figure 3b) usually causes cracking through the mortar joints in a diagonal pattern and is commonly seen when the wall is stacked using a common bond and the crack forms a diagonal line across the wall. The cracks occur “when the tensile stresses, developed in the wall under a combination of vertical and horizontal loads, exceed the tensile strength of the masonry material” (Jamal, 2017). Rocking (Figure 3c) and toe crushing (Figure 3d) are related when the wall rocks back and forth causing a compression stress at the “toe” of the wall, which is the bottom corners of the wall causing the unit in that location to crush due to the high compression forces concentrated on it.

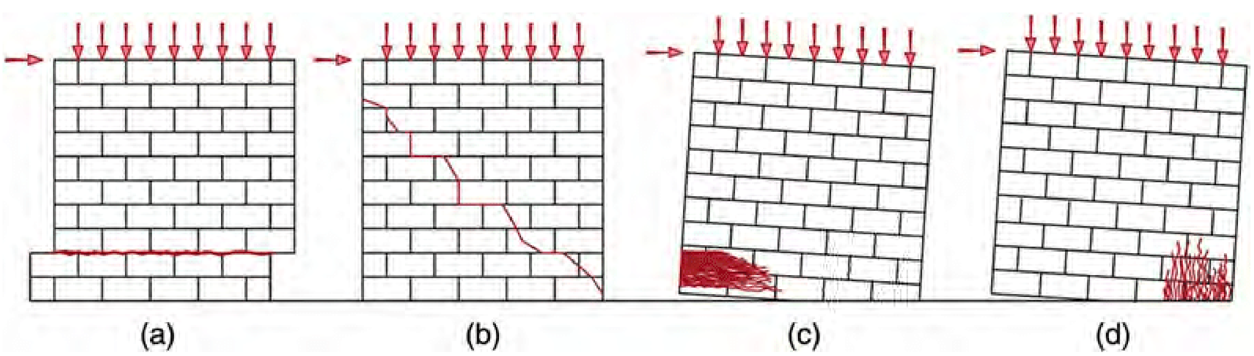


Figure 3: In-Plane Failures of Masonry (a) sliding shear failure (b) diagonal shear (c) rocking (d) toe crushing

Source: (Oyguc et al., 2017)

There are also several ways that masonry can fail under out-of-plane loading. There is failure of the mortar joint, failure of the bricks, and bending failure. Failure of the mortar joints is similar to sliding shear failure where the wall will crack through the mortar and cause the wall to slip inward or outward on the joints. Failure of the bricks is also similar to a diagonal shear failure, where cracks will form through the bricks and cause the wall to also slip inward or outward. Bending failure is the most common failure that occurs during out-of-plane loading. It happens because the flexural strength of the wall cannot resist the load that is applied because of the lack of tensile reinforcement. Figure 4 shows examples of these failures.

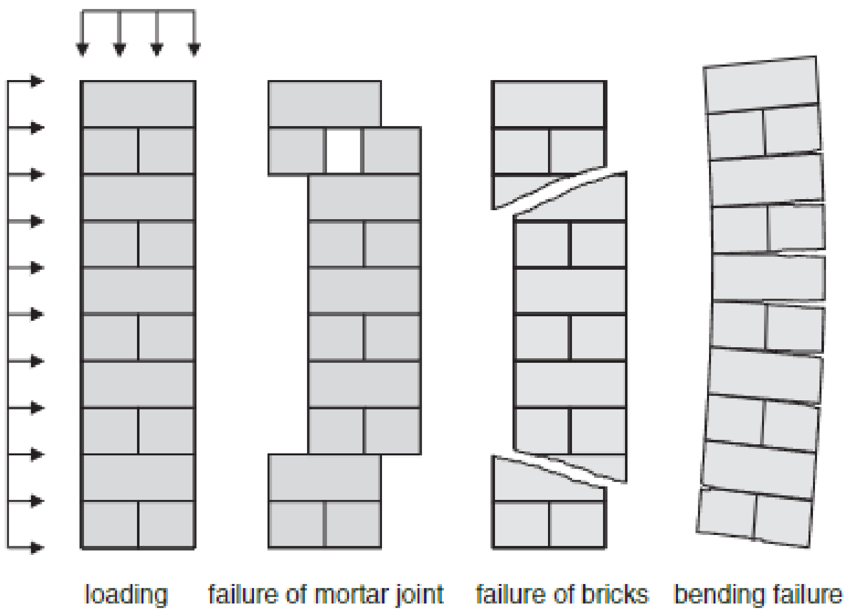


Figure 4: Out-of-Plane Failure Modes

Source: (Lönhoff et al., 2018)

Before modern building codes addressed seismic requirements, almost all masonry structures didn't have steel reinforcement. Because unreinforced masonry structures are vulnerable to seismic loads, there is a great need to retrofit many historical buildings. Over the past few decades, many retrofitting technologies have been developed for unreinforced masonry. This report aims to introduce the technical details and practical applications for some of these methods.

Structure of this report

This report seeks to inform the readers of the different methods of seismic retrofitting that are available to strengthen unreinforced masonry structures. First, a brief history of masonry as a building material, historical failures of unreinforced masonry structures during seismic events, and how unreinforced masonry fails are introduced. Then the different methods of retrofitting are presented, with an explanation of technical data and how they are applied. Then experimental studies of the different methods are discussed. Finally, different case studies of the retrofitting methods are used to show how they have been proven effective in seismic application.

Chapter 2: Methods of Retrofitting

Masonry, much like concrete, is great in compression but terrible in tension. The addition of steel reinforcement is needed to carry the tensile stresses. The main reason many of these historic buildings (unreinforced masonry) have failed during seismic events is because tensile stresses developed in masonry under ground shaking. This is why unreinforced masonry requires retrofitting to prevent this type of failure. These methods of retrofit generally increase the tensile strength of masonry, either by directly adding tensile reinforcement or other means. Masonry walls often rely on diaphragms for lateral support. Weak connection between diaphragms and walls can also cause premature failure of masonry structures. One of the retrofitting methods, diaphragm ties, is addressing this issue.

Diaphragm Ties

The first method of retrofitting is wall to diaphragm ties. The diaphragm is one of the most important components of a building's lateral force resistance system. Diaphragm also serves as lateral bracing for walls. It reduces the unbraced height of walls if proper connection is done. But it cannot be as effective without proper wall to diaphragm ties. There are typically two types of ties: shear ties (Figure 5a-b) and tension ties (Figure 5b). Tension ties take out of plane loads and transfer the loads from the face of the masonry into the diaphragm. Shear ties take loads from the diaphragm and transfer them into the walls of the building. Tension ties keep the wall from pulling away from the diaphragm while the shear ties keep the diaphragm from sliding along the wall.

The tie shown in Figure 5a is classified as a shear only tie. These ties will typically be drilled about 8 inches deep into the masonry and are rated for about 1,000 pounds at the ASD level. The tie shown in Figure 5b is a “combination” tie that can be used for both tension and shear. It will typically be drilled into the wall at a 22.5-degree angle with a minimum distance of 13 inches into the brick. Being angled will allow for the tie to engage with more bricks. The tie is rated for 1,200 pounds in tension and 1,000 pounds in shear at the ASD level. In Figure 6 a through bolt anchor with a plate is shown. The anchor utilizes an 8-inch sleeve that is used to take tension and shear forces and has the same values as the angled dowel. The standard detail of these ties shows the use of a ¾ inch diameter rod being placed in 1-inch diameter holes. “The typical installation approach is to drill the hole; clean it with a brush and compressed air; fill a nylon, carbon, or stainless-steel screen tube (which looks like a test tube made out of wire mesh) with adhesive; place the screen tube into the hole; and then push the rod into the screen tube forcing the adhesive out of the tube into the annulus between the tube and the masonry” (FEMA 547, 2006). Older ties would typically use grout and, because of this, the holes that were drilled into the masonry were required to be much larger so that the grout could be placed. Epoxy is now the most common adhesive used for modern day wall to diaphragm ties. The use of epoxy is also great for corrosion protection. The spacing of the anchors should not be placed closer than 12 inches on center.

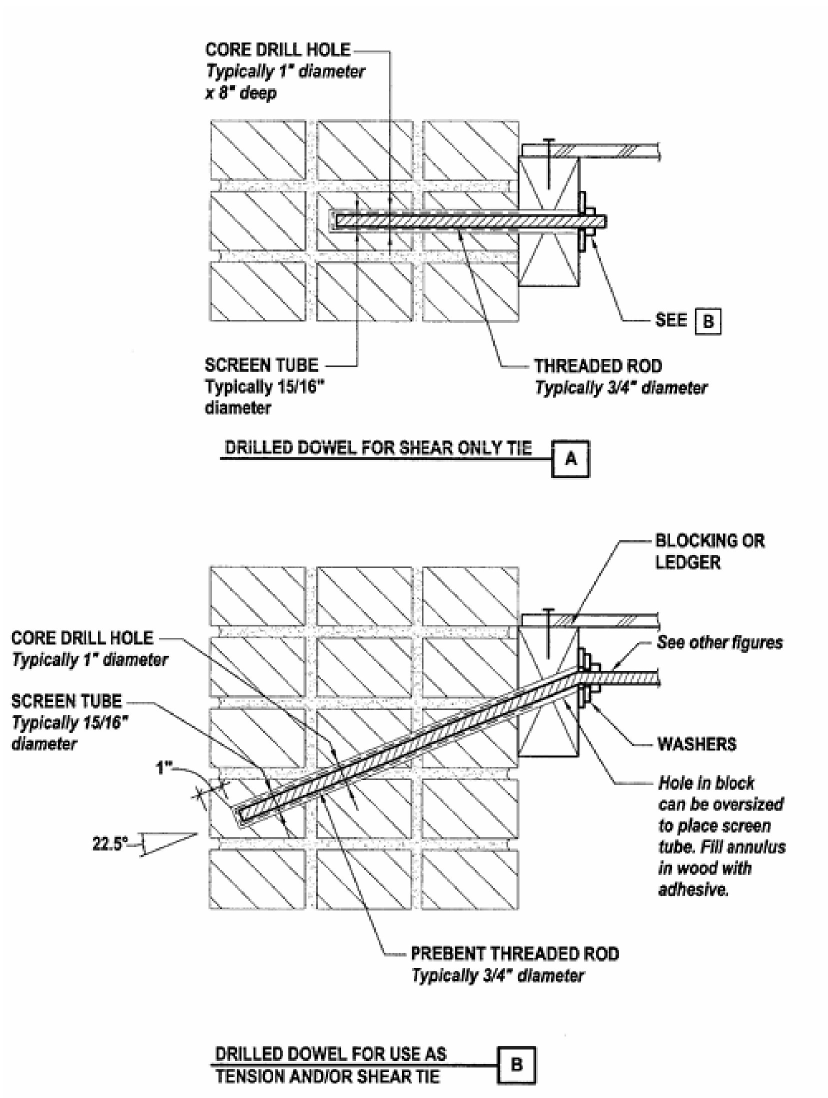


Figure 5: Diaphragm Ties (a) Shear tie (b) Shear and/or Tension tie

Source: (FEMA 547, 2006)

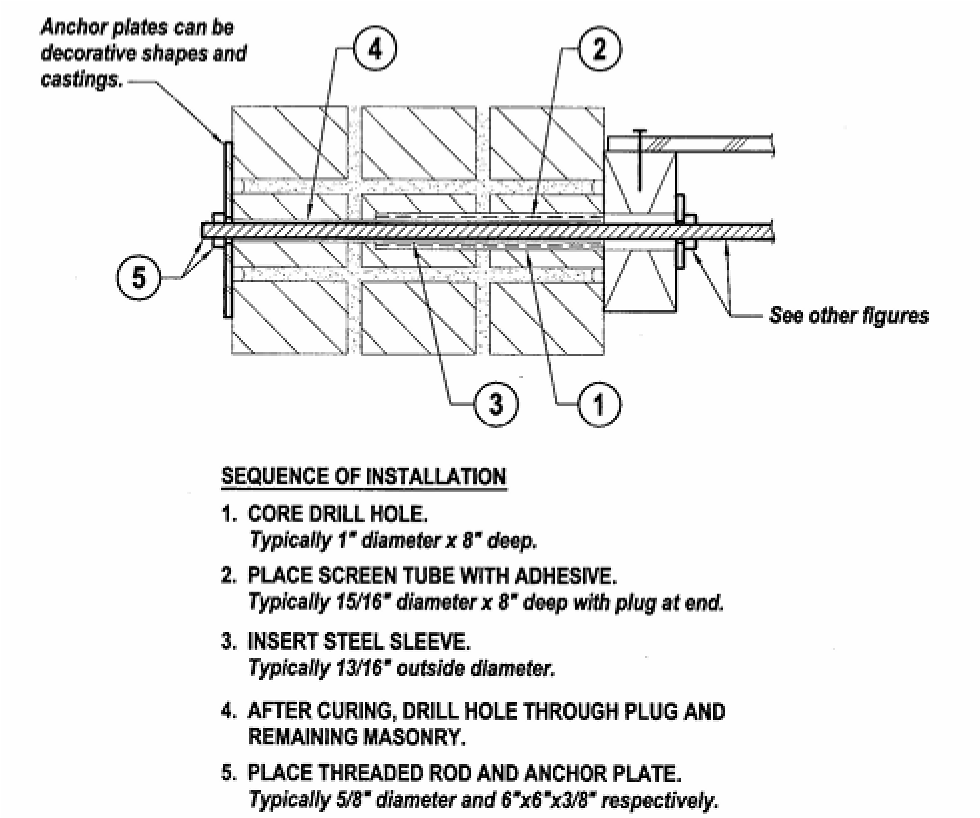


Figure 6: Through Bolt Anchor

Source: (FEMA 547, 2006)

The reasoning for these ties has to do with basic load transfer. The diaphragm that is a part of the elevated floors and roof can be used as a brace if the wall is properly tied into the diaphragm. This will shorten the unbraced length and reduce the out-of-plane moment in the wall allowing for less reinforcement. The tension ties are then used to keep the wall and the diaphragm from pulling away from each other. When a lateral load acts on the walls of a building, it creates a shear force in the diaphragm. This shear force will go through the shear ties that are connected to the walls. The walls then absorb the shear force and transfer it to the ground. Without proper connection the shear walls are effectively useless.

This method of retrofitting has several issues that need to be considered. The first issue is that the ties that are drilled completely through can have an effect on the aesthetics of the building because the bearing plate will be visible on the outside surface, (see Figure 6), although there are covers that can be purchased to add to the aesthetics of the building (Figure 7). Another consideration is where the ties need to be drilled. The ties can be installed from either the top or bottom of the diaphragm. "The choice of whether to install from above or below depends on whether there are finishes that need to be avoided, whether diaphragm strengthening is being done, and what type of diaphragm strengthening is planned. If there is a special plaster ceiling to be avoided, then access and installation would proceed from above. If there is no plaster ceiling and the floor or roof diaphragm is not being modified or is being enhanced by adding a wood structural panel overlay from above, then access and installation for wall diaphragm ties would be from below" (FEMA 547, 2006). The last issue to be covered is the disruption of installation that occurs. The disruption occurs because of the noise from drilling and that the floor or ceiling must be removed to install the dowels. This makes it impractical to install the dowels while a room is being occupied.

Figure 7: Decorative Anchor Covers
Source: (Areta, 2020)

Grout and Epoxy Injections

Grout and epoxy injection is the most popular technique for repairing cracked masonry units, primarily for its ease of installation and low cost. However, this is more of a repair method than retrofitting because it does not increase the strength of the URM walls significantly. This method's main purpose is to fill cracks and voids that are present in masonry that incurred from lateral loading or just pure deterioration of the masonry over time (Figure 8). Grout and epoxy injections aim is to restore the masonry to its original strength, without increasing the strength to an extent that exceeds the original static scheme. The reasoning behind not increasing this is for the potential of damage happening elsewhere around the structure due to stress redistributions.

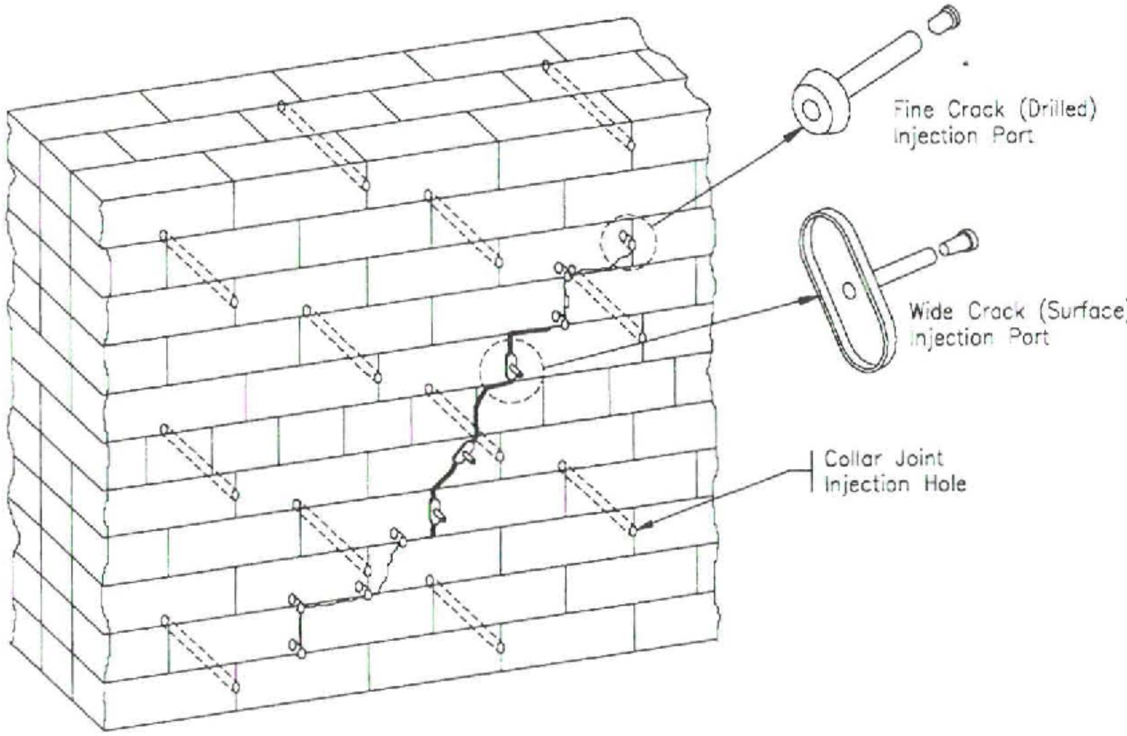


Figure 8: Injection Repair

Souce: (SehuJler Et al, 2015)

The reasoning behind this method is to stop the cracking from getting worse. It's not so much a retrofit method as much as it is a repair method. The grout will not have a very strong tensile capacity because it is like the original grout that is already in the wall. The epoxy resin will have a higher tensile strength than the grout and will be able to absorb some of the tension load that the wall experiences during a seismic event. This method could assist in the shear load that is present during the wall, as most of the need for this method is due to a shear failure, such as cracking through the masonry and grout. Epoxy is better for resisting shear loads because of its flowability and ability to reach into deeper voids and cracks and fill them.

This method also has an advantage of not having to alter buildings' architectural aesthetic features. The grout, which is cement based, will typically have a high water to cement ratio to ensure that the grout reaches to the deepest parts of the cracks. Grout is typically best when used to repair large cracks and voids. Epoxy resins are a flexible, durable, and highly effective material that can be used to fill small cracks that would not be suitable for the use of grout. The main disadvantage to using epoxy resins over the use of grout is that the cost of the resin will be higher than the cost of the grout. The repair process of a cracked masonry wall using grout or epoxy has the following steps: "selection and placement of the injection ports and sealing of the cracks to prevent leakage of the inserted material. Injection of water from the bottom of the wall to its top to identify which paths are active and clean them from dust and other loose particles. Injection of the filling material starting from the lowest port and continuing to the highest one. Low pressure should be used to protect the surrounding masonry" (Gkournelos, 2022). Because this method aims to not strengthen the masonry, it is not as much a retrofit as much as it is a repair. It aims to repair cracks and prevent further cracking. This method is great for buildings with historic significance as it does not alter architectural aesthetics.

Carbon Fiber Reinforced Polymer

The next method of retrofitting is to use carbon fiber reinforced polymer (CFRP). CFRP is a lightweight material that has a very high strength and stiffness. It can come in either bars or sheets, in this case it is sheets. Carbon fiber itself consists of very thin strands that are roughly 5 to 10 micrometers thick. They are paired together then to form a tow (Figure 9) which will then be woven into a fabric. Once the fabric is made it will be paired with a polymer resin which then makes it a carbon fiber reinforced polymer (Figure 10). It is about 4 times lighter than steel. Along with its great strength to weight ratio, it is also resistant to chemicals and corrosion, and is 100% recyclable. But these advantages don't come without their fair share of disadvantages. The main disadvantage of CFRP is the cost. In comparison to steel, CFRP costs roughly $15 per pound while steel costs roughly $0.42 per pound. But there are considerations to be made when comparing CFRP to steel, "…carbon fiber costs must include manufacturing process costs, skilled labor costs, as well as material costs. Comparatively speaking, producing carbon fiber is a very precise and time-consuming process, but it's worth it in the end. While carbon fiber components may cost a bit more, they are stronger, lighter, and built to last much longer than a steel counterpart" (Zoletk, 2021).

Figure 9: Carbon Fiber Tow

Source: (FibreGlast)

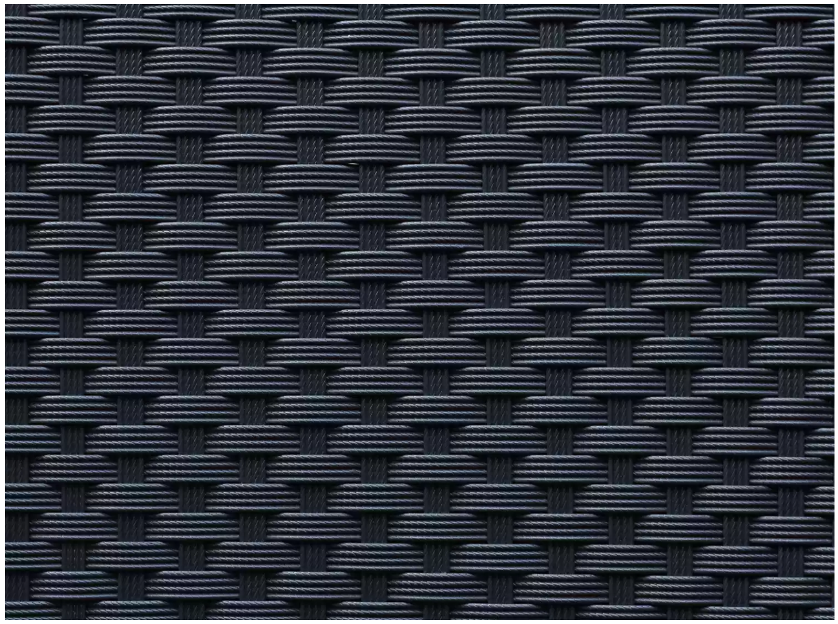


Figure 10: Carbon Fiber Reinforced Polymer

Source: (Johnson, 2019)

The main purpose of the addition of CFRP is to improve a walls in-place capacity, but it has been shown to increase the out-of-plane capacity. CFRP can be installed on one side of the wall or both. An epoxy is needed to bond the CFRP to the wall and another layer of epoxy will be added to the top of the CFRP to ensure a proper bond and also provide moisture protection to the CFRP. To install the CFRP sheets to the walls, the walls will be roughened up usually by being blasted with an abrasive material and then cleaned to ensure an adequate connection between the epoxy and the wall surface. When the epoxy has been added the CFRP sheets are set on top of that initial coating of epoxy and then the second layer of epoxy is added (Figure 11). This process can be disruptive to the occupants due to the noise of the blasting and the fumes from the epoxy. The use of this method is not preferred to be used on the exterior of the building if the architectural aesthetics are an issue (Figure 12).

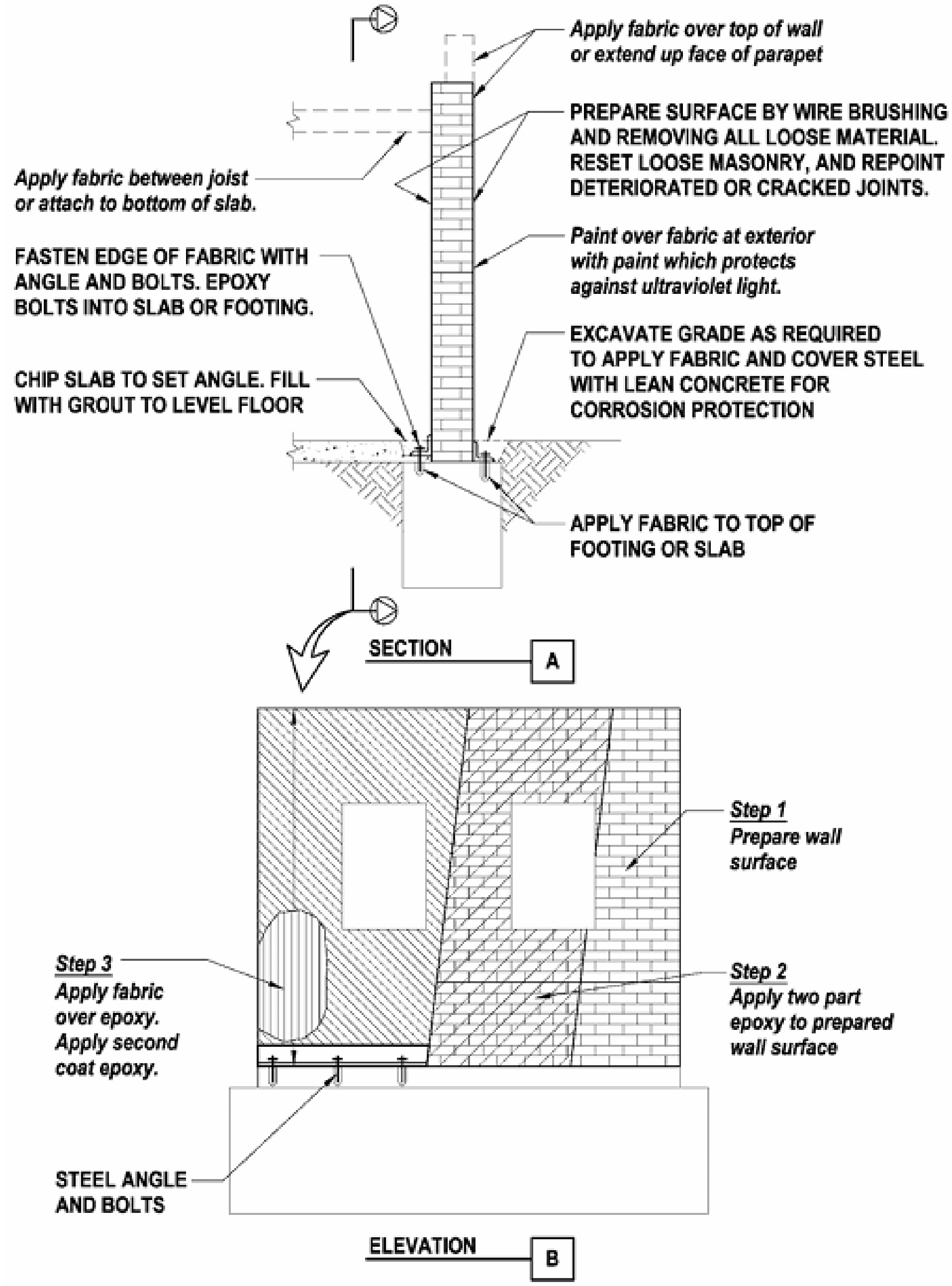


Figure 11: Fiber Composite Overlay on URM Wall

Source: (FEMA 547, 2006)

The addition of CFRP can improve both in-plane and out-of-plane capacity. This retrofitting method is typically used to increase the shear strength of the wall. CFRP can also increase the tensile capacity of the wall due to an out-of-plane load. The CFRP is used to take the tension and shear stresses that will be present on the wall during a seismic event. When a shear load happens, the CFRP can assist in preventing a sliding failure because of its stronger shear capacity, compared to that of masonry CFRP's shear capacity is nearly 36 times stronger.

Figure 12: CFRP on Exterior of Masonry Building
Source: (CST Stabilization, 2020)

Parapet Bracing

In past seismic events, the parapet and chimneys have been the first thing to fail because of inadequate bending strength and ductility. During a seismic event, the parapet will break off and fall to the ground causing injury and or fatalities to pedestrians, damage to other buildings, the property below the building, and even possibly further damaging the building itself. Because of this parapet bracing is a great necessity. The retrofit typically consists of using a steel member to brace the parapet to the roof (Figure 13). The steel member will typically be connected to the roof and parapet using a steel angle. The steel angle connection at the roof will be screwed to the blocking of the structure with a bolted connection to the steel member that is connected to the parapet. The existing roof structure may need to be strengthened in the connected area to take the reaction from the angle. This can include the addition of blocking or adding sheathing to the area. Also because of the connection at the roof causing penetrations through the membrane of the roof, proper waterproofing must be completed. The steel angle connection at the parapet will typically have a gusset plate that is connected to the steel support member by a weld. The angle that is connected to the parapet will be connected by either a through bolt connection with a bearing plate or a drilled dowel that is placed at an angle, similar to diaphragm ties. If the architectural aesthetics are in question, the better option is to use drilled dowels to avoid the bearing plate being a distraction.

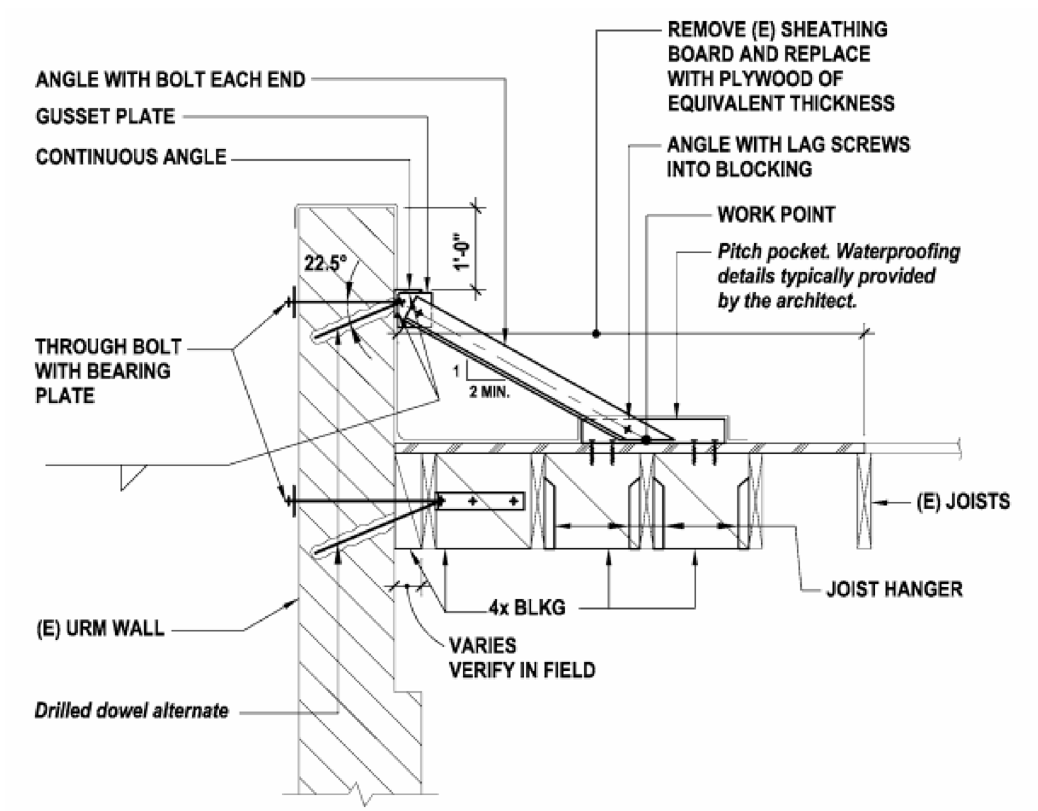


Figure 13: Typical Section of Parapet Bracing

Source: (FEMA 547, 2006)

Figure 14: Parapet Bracing

Source: (FEMA E-74, 2011)

The way the force is transferred and how this method works can also be attributed to proper diaphragm ties. Without the diaphragm and parapet being properly tied in this method would fail. Because a parapet is like a cantilever beam, it experiences a greater moment than a wall that is pinned at the top and the bottom. Because of this higher moment at the base, it has a higher chance of failure. The force that the parapet experiences will be transferred through the angle that is placed on the parapet itself and then through the member that is connected to the parapet and the diaphragm itself. Once the force reaches the diaphragm, the diaphragm absorbs that force and transfers it out to the lateral force resisting systems which will then carry the force to the ground.

This issue is so prominent that the city of San Francisco has implemented into their existing building code requirements for parapet bracing, and if the requirements are not met the parapet must be removed. “Parapets whose heights do not exceed 3 times their thicknesses need not be removed, stabilized or braced, provided they are located either immediately adjacent to a normally inaccessible court or yard or another building. In the case of an adjoining building, the top of the parapet of the building under consideration shall not be more than 12 inches (0.305 m) above the top of the parapet of the adjoining building. In order to qualify for this exception, the owner must execute an agreement with the Department to voluntarily abate any hazard that may arise as a result of changed conditions such as demolition of the adjacent building or development or occupancy of the adjoining court or yard. The owner must record the agreement with the County Recorder on a form satisfactory to the Department and supply a copy of the recorded agreement to the Department" (San Francisco Existing Building Code, 2022).

Out-of-Plane Wall Bracing

The next method of retrofitting discussed will be the addition of out-of-plane wall bracing. This method is when either a steel or wood member is placed on the inside of the building to absorb the tensile load that the building will experience. This method is typically used for walls that have a high h/t ratio. Where h is the height of the wall and t is the thickness of the wall. There are 2 types of bracing that can be used on an unreinforced masonry building: diagonal braces and vertical braces. The diagonal bracing will typically consist of steel square tubing that is surface mounted to the wall horizontally with a bracing rod connected from the horizontal member to a beam or joist above (Figure 15). The vertical bracing can consist of either wood or steel that will be connected to the center height of the wall via an anchor and will also be connected to both the floor above and below (Figure 15). Spacing requirements for both horizontal and vertical bracing are given in the 2003 International Existing Building Code. For vertical bracing the maximum spacing is 10 feet or half the unsupported height of the wall and for horizontal bracing the maximum spacing is 6 feet. If architectural aesthetics are an issue, a vertical brace can be recessed into the wall, although this will be costlier.

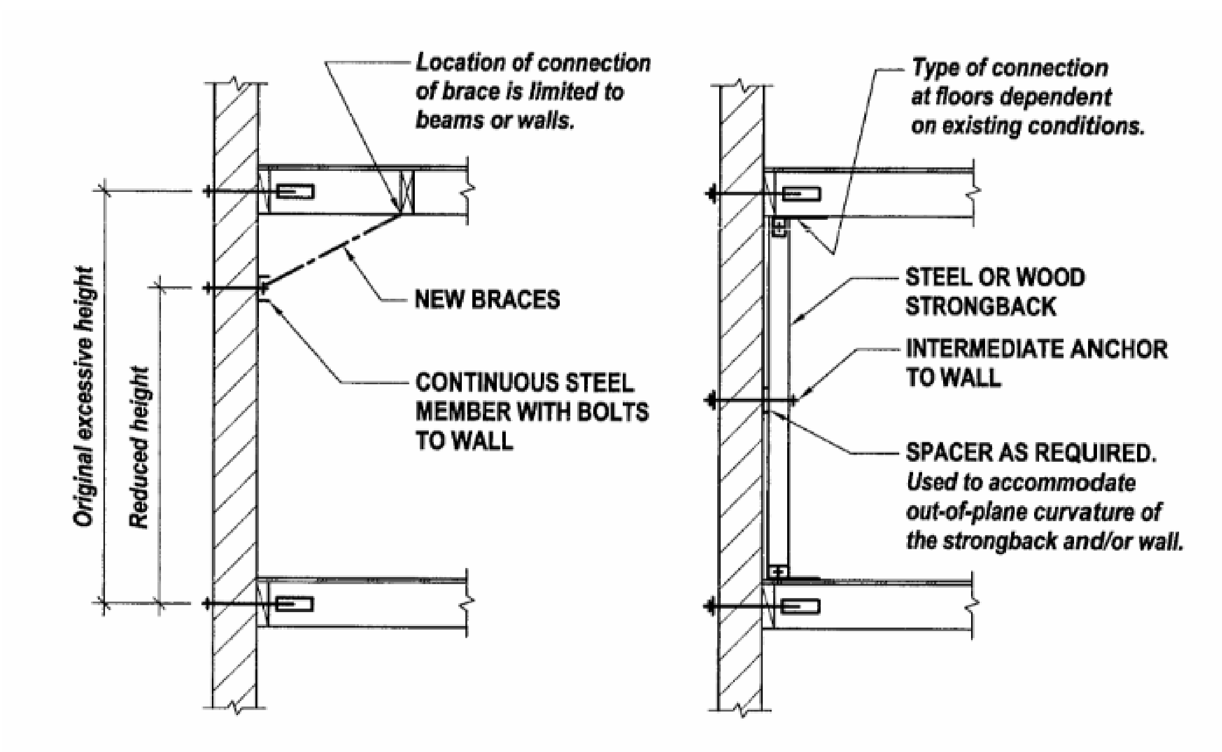


Figure 15: Out-of-Plane Wall Bracing

Source: (FEMA 547, 2006)

This method is great to be paired with wall to diaphragm ties because the load that will be absorbed by the bracing members. It will be transferred into the diaphragms of the building and, without proper anchorage to the diaphragms, the method would not be effective. The method works because the reduction in the wall's unbraced height will greatly reduce the moment that the wall experiences and have a less chance of failure. When the wall experiences an out-of-plane load, such as caused by seismic or wind, the unreinforced masonry wall will remain in compression and will rely on the bracing to absorb the tensile load, thus preventing a bending failure.

This method is more cost effective for out-of-plane load resistance than CFRP. To make it even more cost effective, the use of timber instead of steel can be used. Although diagonal bracing is typically the least expensive option, it is considered to be less reliable than vertical bracing. Also, if diagonal bracing is used, if the diaphragm deflects it can cause an inertial force on the brace which then will cause a force onto the wall itself. If the aesthetics of the building are a concern, the bracing can be covered up but at an additional cost. The installation of the bracing can be a disruptive process as well, due to the access to the perimeter of the building and the noise of drilling dowels and having access to the diaphragms of the building.

Shotcrete

The next method of retrofitting will be the use of shotcrete. This method is the act of having a mesh of steel rebar or welded wire reinforcement and placing it on the walls of the unreinforced masonry building by using drilled dowels. The drilled dowels will transfer the shear from the shotcrete and masonry and can also act as out-of-plane ties for the masonry. The dowels are typically spaced around 2 to 3 feet. Then once the reinforcement is added a sprayable concrete solution is used to overlay on top of the reinforcement (Figure 16). This method is far cheaper than the other overlay method of using carbon fiber reinforced polymers and more convenient because of constructability. Although, because of the weight of the steel reinforcement and the shotcrete itself, the inertial load that the building experiences during a seismic event will be greater than that of a building that is retrofitted with CFRP. This increased inertial load will also cause the wall to diaphragm tie spacing to decrease. Because this method is typically applied on the exterior of the structure, it is not suited for masonry structures that have prominent architectural features that cannot be covered up.

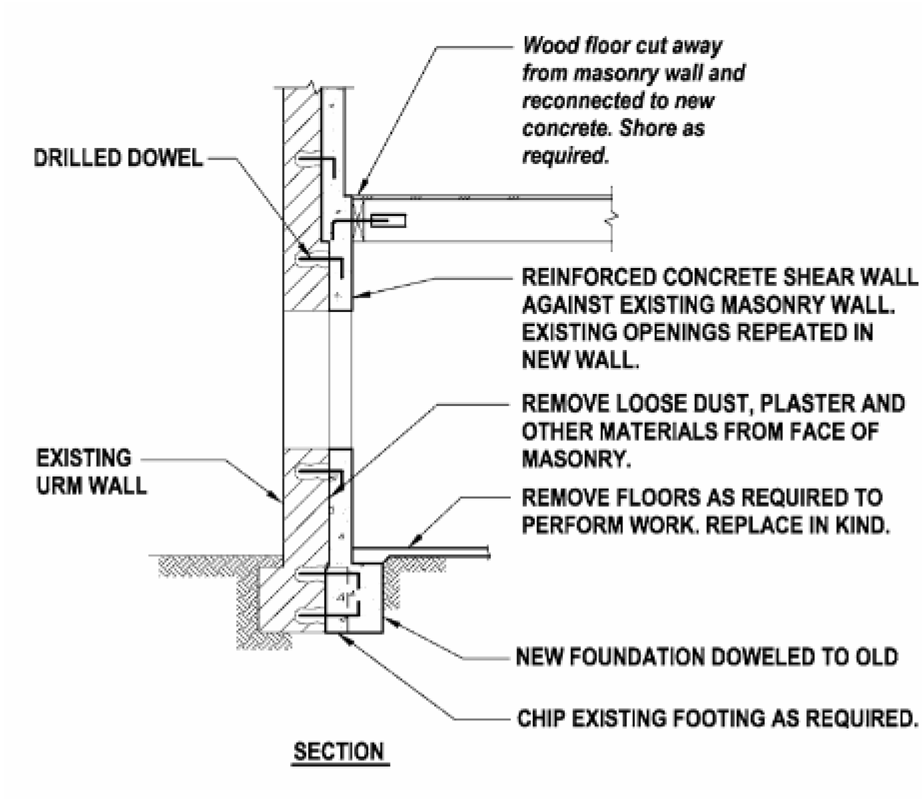


Figure 16: Shotcrete Wall Overlay
Source: (FEMA 547, 2006)

Because of the addition of steel reinforcement, when shotcrete is used, the tensile loads that a structure experiences can be properly transferred. When a seismic force hits, the tensile loads can cause the building to crack and fail because of the lack of this reinforcement. Because steel reinforcement is needed for the shotcrete to help adhere to the surface of the masonry wall, these tensile forces can be transferred properly back to the ground and the building will not fail as the building would if it was not reinforced. The force is transferred through the concrete first, then to the steel mesh, and from the mesh to the wall, via the drilled dowels that the mesh is attached to the masonry wall with. Once the masonry wall experiences the load, it will transfer it down through the foundation that the wall is placed on.

The process of applying shotcrete can be a disruptive process. This is because of the noise produced by drilling the dowels into the masonry wall, attaching the steel mesh, and the pumping of the concrete. Because shotcrete is best if it is shot downward so the concrete can build upon itself, the use of scaffolding is needed. This can cause space issues as well as a space for the truck to be placed to pump the concrete. If the wall that is being retrofitted is inside, it is necessary to seal the room off with plastic sheathing, to help assist with dust that will come up with spraying shotcrete.

Chapter 3: Experimental Studies

This chapter introduces experimental studies that have been performed for some of the retrofitting methods which were discussed in the previous chapter. The studies are about grout and epoxy injections, CFRP, parapet bracing, out-of-plane wall bracing, and shotcrete. The experiments proved that these retrofitting methods could increase the strength of unreinforced masonry significantly.

Grout and Epoxy Injections

Calvi et al., (1994) did an experimental study for grout and epoxy injection method of retrofitting masonry. The purpose of the study was to test the effect of using grout and epoxy injections to increase the shear strength of cracked unreinforced masonry walls. In this study, 6 walls were tested before and after being retrofitted with grout and epoxy injections. The walls "had width d=1.5m, thickness t= 0.38 m (three wythes, English bond), height h= 2 or 3m. They had been subjected previously to cyclic or monotonic shear tests with different values of vertical compression." (Calvi Et al, 1994). The walls were made with fired clay solid bricks and lime mortar, which is similar to historical brick masonry. The walls were placed in a press to resemble compression, with a hydraulic jack on the side to apply a shear force (Figure 17). Comparison was made of the cyclic behavior and the shear strength of the retrofitted walls.

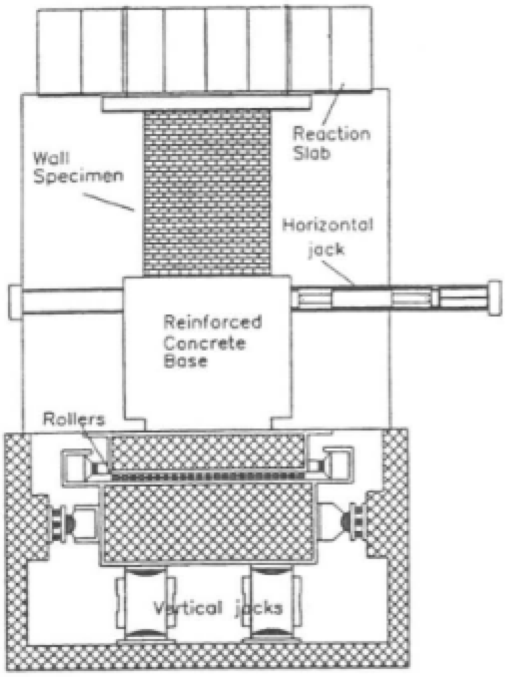
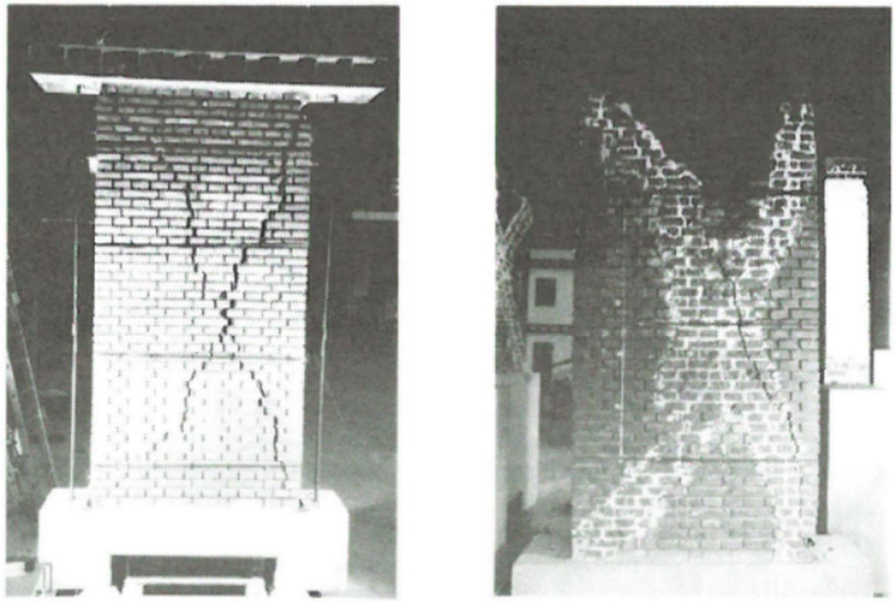


Figure 17: Grout and Epoxy Injection Press

Source: (Calvi et al., 1994)

The walls were initially built unreinforced and tested to crack as a reference (Figure 18a). The walls were then repaired with either a grout or epoxy injection (Figure 18b). For this study 4 walls were repaired with grout and 2 with epoxy. The procedure for repairing the walls with grout went as follows: First, holes were drilled for injection. Next, the cracks and holes were washed with water. Then, the injection ports were placed, and screens were placed along the surface to avoid grout leaks. The grout was injected. Once the grout cured, the screens were removed. The procedure for repairing the walls with epoxy is similar.



(a) (b)

Figure 18: (a) Wall Before Repair (b) Wall After Repair

Source: (Calvi Et al, 1994)

Specimen denomination	mortar type	height [m]	type of test before repair	crack pattern	crack width (mm)
MI1m	hydraulic lime	2.0	monotonic	SDM	≤ 10
MI1	hydraulic lime	2.0	cyclic	DDM	≤ 30
MI2	hydraulic lime	2.0	cyclic	DD	≤8
MI3	hydraulic lime	3.0	cyclic	dd	≤ 2
MI4	hydraulic lime	3.0	cyclic	DD	≤ 50
MA	common lime	2.0	cyclic	DD	≤ 30

SDM: single diagonal crack in one direction with minor adjacent cracks, mixed type failure (bricks and joints);
DDM: double diagonal cracks (X-shaped) in one direction with minor adjacent cracks, mixed type failure (bricks and joints);
DD: single diagonal cracks in both diections (X-shaped);
dd: diffused cracking, most crack widths less than 1 mm.

Table 2: Wall Characteristics and Crack Types

Source: (Calvi Et al, 1994)

The details of the wall specimens are listed in Table 2. The table shows the crack pattern and the crack width. Because walls MI2 and MI3 cracks were small they were repaired with epoxy while walls MI1m, MI3, MI4, and MA were repaired with grout. Testing was completed and it was found that all walls experienced an increase in strength except for wall MI4, this was due to pre-existing cracking. The results can be seen in Table 3, where F is the strength of the walls before being repaired and F' is the strength of the walls after being repaired.

Wall	**F [MPa]**	**F' [MPa]**	**F'/F**
MI1m (Grout)	0.284	0.317	1.12
MI1 (Grout)	0.264	0.325	1.23
MI4 (Grout)	0.196	0.159	0.81
MA (Grout)	0.134	0.184	1.37
MI2 (Epoxy)	0.308	0.661	2.15
MI3 (Epoxy)	0.157	0.614	3.91

Note: 1 MPa = 145 psi

Table 3: Repaired Walls Strength

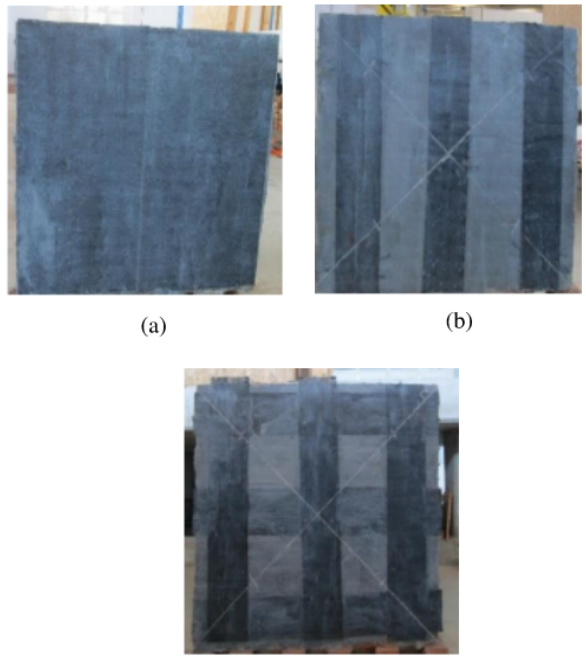
Source: (Calvi Et al, 1994)

"The walls repaired with epoxy resin increased the strength to such a level that the tests had to be stopped because the limits of the reaction system for the horizontal force had been reached, without any sign of cracking in the repaired zone at the center of the panels". (Calvi et al., 1994). The reasoning behind the epoxy resin having a higher increase in strength than grout is because the injectability of the resin allows the penetration into small voids and cracks and allows the structure to absorb the tensile stresses caused by the horizontal shear.

In conclusion, it was found that the use of either injection type was able to restore the original strength of unreinforced masonry walls which had cracked previously. The only case where the original strength was not met was due to cracks which were not filled by grout. One issue related to epoxy resin injections is that there is a possibility of leaks caused by voids and cracks that are not visually detectable. But this is still an extremely cost-effective method to restore masonry to its original strength.

Carbon Fiber Reinforced Polymer

Elmalyh et al. did an experimental study titled "*Shear Strength of Unreinforced Masonry Walls Retrofitted with CFRP*" (2020). The experimental study was to test the in-plane loading of a shear wall retrofitted with CFRP. The study consisted of four total walls with three that were retrofitted, one that was unreinforced to be used as a reference, one that was entirely covered with CFRP (Figure 19a), another wall that consisted of three vertical strips of CFRP (Figure 19b), and a fourth wall that had three vertical and three horizontal strips of CFRP (Figure 19c). The walls were 1200 x 1200 x 115 mm (47.2 x 47.2 x 4.5 inches) in size and were constructed of masonry units of 240 x 115 x 63 mm (9.5 x 4.5 x 2.5 inches) with 10 mm (0.6 inches) thick mortar joints. The preparation of the walls for the CFRP retrofitting started with the walls being cleaned using high pressure air to remove any debris from the surface, then primed and mortared. Once those preparations had been completed the walls got the initial coating of epoxy resin, then the different configuration of CFRP was added to the walls. After that the second layer of epoxy resin was placed on top of the CFRP. When the curing process of the walls was completed, they were placed into the testing machine at a 45-degree angle under monotonic compressive load to determine the diagonal tensile strength for each wall (Figure 20). The results of the loading of the walls where the initial cracking started to form is shown in Table 4 below.



(a) (b) (c)

Figure 19: (a) Wall Completely Covered w/ CFRP (b) Wall w/ 3 Vertical Strips of CFRP (c) Wall w/ 3 Vertical & Horizontal Strips of CFRP

Source: (Elmalyh et Al., 2020)

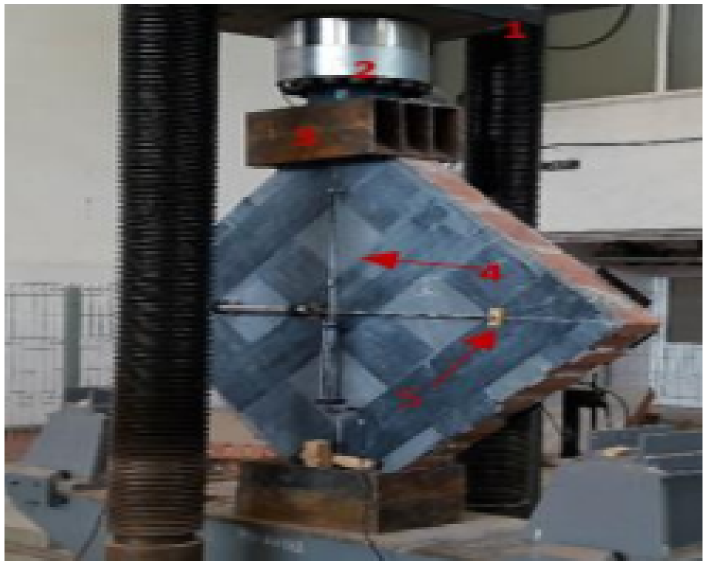


Figure 20: Test Setup

Source: (Elmalyh et al., 2020)

Configuration type	Side N°	Load (KN)	ΔV (mm)	ΔH (mm)	Shear stress (N/mm²)	Shear strain (mm/mm)	Modulus of Rigidity (MPa)
Initial Cracks							
URM	1	7.9	0.62	0.65	0.04	0.0026	15.38
	2		1.24	0.52		0.0035	11.43
W- CFRP-W 1	1	30.8	1.20	0.18	0.16	0.0028	57.14
	2		4.83	0.37		0.01	16.00
W- CFRP-W 2	1	27.9	0.97	0.08	0.14	0.0021	66.67
	2		1.8	0.14		0.0039	35.90
W-CFRP-W 3	1	24.7	1.02	0.7	0.13	0.0034	38.24
	2		1.46	3.02		0.009	14.44

Table 4: Initial Cracking of Walls

Source: (Elmalyh et al., 2020)

As seen in Table 4, for the strength at initial cracking, the unreinforced masonry wall could withstand the least amount of load at 7.9 kN (1.8 kips), while the wall that had been completely covered with the CFRP sheets withstood the greatest amount of load at 30.8 kN (6.9

kips). This was a 22.9 kN (5.2 kip) increase, or nearly a 400% increase in strength.

Configuration type	Side N°	Load (KN)	ΔV (mm)	ΔH (mm)	Shear stress (N/mm²)	Shear strain (mm/mm)	Modulus of Rigidity (MPa)
Failure points							
URM	1	9.3	0.62	0.65	0.05	0.0026	18.46
	2		1.24	0.52		0.0035	13.71
W- CFRP-W 1	1	32.3	1.2	0.18	0.17	0.0028	60.71
	2		4.83	0.37		0.01	17.00
W- CFRP-W 2	1	29.4	1.03	0.15	0.15	0.0023	65.22
	2		1.90	1		0.0058	25.86
W-CFRP-W 3	1	26.2	1.02	0.7	0.13	0.0034	38.24
	2		1.46	3.02		0.009	14.44

Table 5: Failure Points of Walls

Source: (Elmalyh et al., 2020)

The results were very similar for the strength at complete failure, as shown in Table 5. The unreinforced masonry wall was able to withstand the least amount of strength, with a load of 9.3 kN (2.1 kips), while the fully covered wall withstood a load of 32.3 kN (7.3 kips), a 23 kN (5.2 kips) difference or almost 400% increase. Although the other walls were not as strong as the wall that had been completely covered by the CFRP retrofit, they still exhibited a significant jump in strength than the unreinforced wall (Table 5).

Parapet Bracing

Giaretton et al. did an experimental study titled "*Shake Table Testing of Seismically Restrained Clay-Brick Masonry Parapets*" (2018). The study was used to test the effectiveness of parapet bracing using either timber or steel. The experimental study had 9 full scale clay brick masonry parapets tested using a shake table (Figure 21). The bracing that was used in this study included both steel and timber. In addition, tests were done for timber bracing paired with timber vertical strong-backs. All test specimens are listed in Table 6.

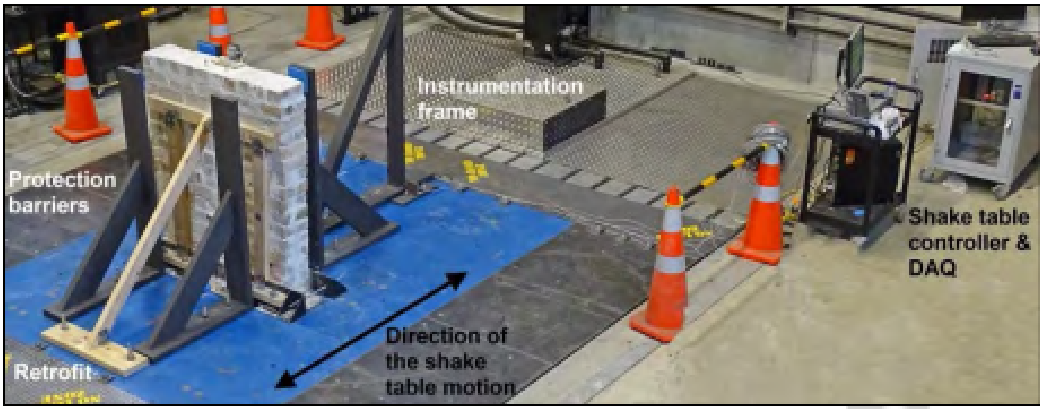


Figure 21: Test Setup

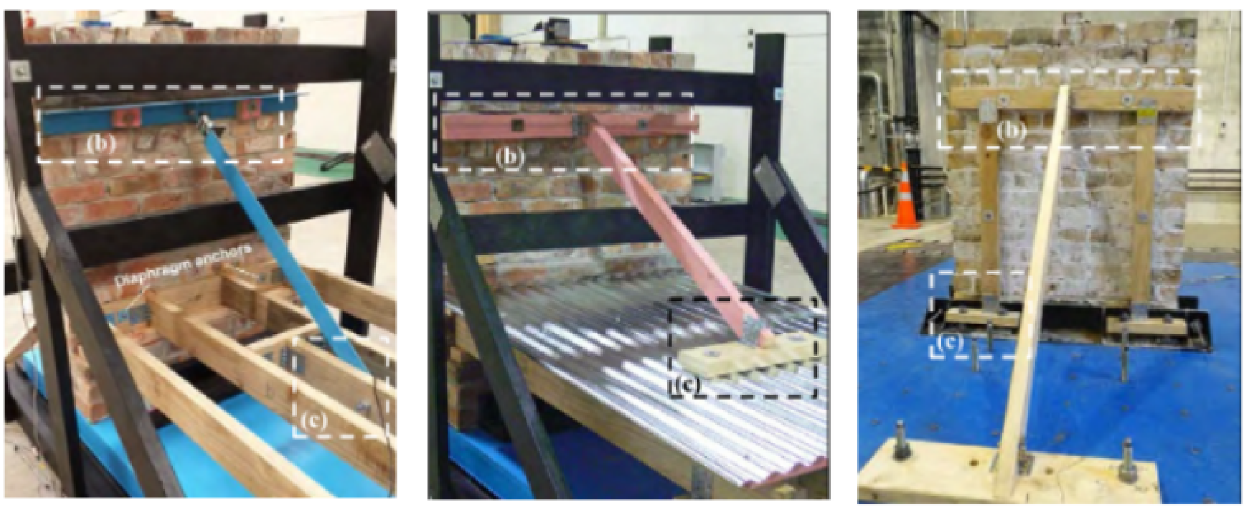
Source: (Giaretton et al., 2018)

Parapet ID	Parapet height (mm)*	Parapet thickness (mm)	Wall thickness (mm)	Mortar mix **	Retrofit type
P1-C(1605)TB	1605 (19)	230	350	0:1:3 (C)	Timber brace
P2-A(1180)SB	1180 (14)	230	350	1:2:9 (A)	Steel brace
P2-A(1180)TB	1180 (14)	230	350	1:2:9 (A)	Timber brace
P2-A(1180)PT	1180 (14)	230	350	1:2:9 (A)	Post-tensioning
P4-B(1180)TB ***	1180 (14)	230	230	1:3:12 (B)	Timber brace
P5-B(1180)SB	1180 (14)	230	350	1:3:12 (B)	Steel brace
P6-B(1180)TBS ***	1180 (14)	230	230	1:3:12 (B)	Timber brace and vertical strong-backs
P7-C(1180)TBS ***	1180 (14)	230	230	0:1:3 (C)	Timber brace and vertical strong-backs
P14-C(1180)TB	1180 (14)	230	350	0:1:3 (C)	Timber brace

Table 6: List of Parapet Designs

Source: (Giaretton et al., 2018)

"The parapets were constructed with the aim of simulating a central portion of the top-story façade of a common existing single or multi-story URM building" (Giaretton et al., 2018). The parapets were constructed with a makeshift timber diaphragm that was anchored to the masonry wall. Two of the parapets that were tested were retrofitted using the technique that was described in chapter 2, the way that is recommended by the Federal Emergency Management Agency (FEMA). It is recommended to use a steel spreader bar that is anchored to the parapet and with the steel brace connected to the bar and then connected to the diaphragm (Figure 22a). Two parapets were also tested by following FEMA guidelines, but with the use of timber instead of steel to test a more cost-effective technique (Figure 22b). With the use of timber there is some risk to be aware of. If the timber is left untreated it is susceptible to rot. The third technique that was tested was the use of timber braces but with the addition of 2 vertical timber strong backs (Figure 22c).



(a) (b) (c)

Figure 22: (a) Steel Braced Parapet (b) Timber Braced Parapet (c) Timber Braced w/ Vertical Strong-backs

Source: (Giaretton et al., 2018)

The test was conducted with a similar ground motion that occurred during the Christchurch earthquake in New Zealand on February 22, 2011. “The test protocol consisted of increasing the acceleration amplitude in increments of 10% of input motion (corresponding to an increasing acceleration of approximately 0.05g) up to failure. During Phase 2, a single-axis acceleration-controlled sinusoidal test transitioning from 0.5 Hz to 30 Hz (FEMA 461) was applied with increasing increments of table acceleration of approximately 0.05g every 10 seconds and constant amplitude of 50 mm" (Giaretton et al., 2018).

The results obtained for this study showed a great increase for braced parapets compared to as-built parapets. The addition of steel and timber diagonal bracing was able to improve the capacity of the parapets by almost 8-times the magnitude of the as-built parapets when they were near-collapse. If durability of the bracing is not of concern, timber bracing is an effective way to save cost. "The addition of vertical strong-backs further improved the performance of braced parapets, with failure occurring at a peak table acceleration that was twice the value recorded for timber-braced parapets without strong-backs." (Giaretton et al., 2018). The vertical strong-back connections need to be designed to transfer the base shear loads into the structure. In conclusion, the addition of any type of retrofit mentioned in this study was able to increase the strength of the parapets and be able to withstand a greater magnitude in comparison to a parapet without any retrofit at all.

Out-of-Plane Wall Bracing

Cassol et al did an experimental study titled "*Seismic Out-of-Plane Retrofit of URM Walls Using Timber Strong-backs*" (2021). The experimental study was to test out-of-plane wall bracing using timber strong-backs as a cost-effective way to retrofit unreinforced masonry walls. Five walls were constructed for this experiment and were subjected to semi-cyclic out-of-plane loading using airbags. Four out of the five walls were built using clay bricks, two that were double leaf (2-wythe) walls, two were single leaf walls, and one wall that was constructed using concrete masonry units (CMU). The strong-backs that were attached to the walls by mechanical screws were spaced at 600 mm (23.6 inches). The strong-backs were attached at each end by timber members that ran perpendicular at the top and bottom of each wall, very similar to a stud wall in timber construction with no blocking. The timber itself was New Zealand Radiata Pine, with a cross-section of 90 x 45 mm (3.54 x 1.77 inches). The timber members were placed on one side, and the loading was applied as if the timber was on the compression side or the tension side of the wall. The test was set up to mimic a single-story perimeter wall (Figure 23).

Figure 23: Test Setup

Source: (Cassol Et al, 2021)

All five walls were first tested without strongbacks to mimic an unreinforced masonry wall as a reference. It was observed that all walls, except one, cracked roughly 2/3 of the height of the wall with one wall cracking at the top constraint. Walls 1 and 2 were double leaf walls and were constructed similarly, but experienced different out-of-plane capacities. Wall 1, which was the wall that experienced the cracking at the top, had an out-of-plane capacity of 4.63 kN (1.04 kips). This was lower than wall 2, which experienced a capacity of 6.03 kN (1.4 kips). Wall 3, which was constructed of CMU, had a capacity of 3.14 kN (0.7 kips). This was lower than walls 1 and 2 because its weight was far less. Walls 4 and 5 were both single leaf and very similar in capacities, at1.73 kN (0.39 kips) and 1.51kN (0.34 kips).

Once these tests were completed, the timber strongbacks were added (Figure 24), and the walls were tested in a way that the strongbacks were on the compression side. Walls 1 and 2 were retrofitted with members that had different cross sections. Wall 1 had a 45 mm x 90 mm (1.77 x 3.54 inches) member and wall 2 had a 90 mm x 90 mm (3.54 x 3.54 in) member, but both were spaced at the same distance. Wall 1's out of plane capacity increased to 16.41 kN (3.7 kips) and wall 2 increased to 40.61 kN (9.1 kips) "which represent an improvement in capacity by approximately 254% and 242% above the capacities of the as-built walls." (Cassol Et al, 2021). The CMU wall (Wall 3) was retrofitted with 45 mm x 90 mm (1.77 x 3.54 inches) members spaced at 500 mm (19.7 inches), which resulted in a capacity of 8.77 kN (2 kips), "an improvement of approximately 179% to the value achieved for the as-built wall." (Cassol Et al, 2021). The two single leaf clay brick walls were retrofitted with the same member at different orientations. Wall 4 was retrofitted with 45 mm x 90mm (1.77 x 3.54 inches) oriented in strong direction, while Wall 5 with the same member oriented in the weak direction. The strongbacks were spaced equally at 450 mm (17.7 inches) for both walls. Wall 4 achieved a capacity of 10.39

kN (2.3 kips) and Wall 5 reached a capacity of 6.26 kN (1.4 kips), “representing improvements of approximately 501% and 315% above the capacities of the as-built walls.” (Cassol et al., 2021). Because of the different orientation of the timber, Wall 4 “was 66% more resistant than Wall 5 while strong-back failure occurred for smaller values of displacement.” (Cassol et al., 2021).

Figure 24: Timber Strongbacks on Wall

Source: (Cassol et al., 2021)

When the walls were tested with timber strongbacks on the tension side, only Walls 2 and 5 were tested. Walls were tested in the same manner as when strongback was placed on the compression side. Wall 2 was retrofitted with a 45 mm x 90 mm (1.77 x 3.54 inches) member and spaced at 450 mm (17.7 inches), the wall achieved a capacity of 33.95 kN (7.6 kips), "an improvement of approximately 463% and 65% above the capacities of the unreinforced Wall 2 and Wall 2 with compression strongbacks, respectively." (Cassol et al., 2021). Wall 5 was retrofitted with members that were 90 mm x 45 mm (3.54 x 1.77 inches) and spaced at 450 mm (17.7 inches), the wall achieved a capacity of 16.73 kN (3.76 kips), "corresponding to a strength increase of 1008% and 167% above the capacities of unreinforced Wall 5 and Wall 5 with compression strongbacks, respectively." (Cassol et al, 2021).

Shotcrete

ElGawady et al. did an experimental study titled "Retrofitting of Masonry Walls Using Shotcrete" (2006). The study was to test the in-plane shear capacities of walls that had been retrofitted. Three half-scale walls were tested where one of the walls was completely unreinforced to use as a reference wall, another wall was retrofitted with a 40 mm (roughly 1.5 inches) thick layer of shotcrete on a single side, and the last wall was retrofitted with 20 mm (roughly 0.75 inches) thick layers of shotcrete on both sides. The walls were constructed using a single wythe pattern with hollow clay brick masonry and a weak mortar. For the shotcrete to be installed, steel reinforcement was added first. The first step was the installation of 4 mm (0.16 inch) shear dowels that were placed into holes drilled into masonry spaced around 250 to 300 mm (9.8 to 11.8 inches). Next was the addition of the steel reinforcement. The reinforcement was a mesh of 4 mm diameter bars spaced at 100 mm (3.9 inches). The wall with shotcrete on both sides had one layer of mesh on each side, and the wall with shotcrete on one side had double layers of reinforcement on the same side. Thus, both walls had the same amount of reinforcement. Lastly, the walls were wetted and the shotcrete was applied (Figure 25).

Figure 25: Shotcrete Being Installed
Source: (ElGawady et al., 2006)

Testing was done on the walls by adding a superimposed dead load of 30 kN (6.7 kips) and the lateral loading was applied cyclicly through a head beam which then distributed the load to the top of the wall (Figure 26). The load was increased by roughly 5 kN (1.1 kips) per cycle. The first wall, the unreinforced reference wall, “had an average lateral strength of 35.5 kN (8 kips). It had mixed modes of failure, namely rocking, sliding, and toe crushing” (ElGawady et al., 2006). At 15.1 kN (3.4 kips) the first crack was formed. The next crack was observed after a load of 24.9 kN (5.6 kips) was reached. The wall did not crack again until a load of 29.7 kN (6.7 kips) and it failed at 35.5 kN (8 kips). The wall reinforced with shotcrete on its one side was tested next. For this wall, the lateral load was increased by 10 kN (2.2 kips) per cycle. The first visible crack wasn’t observed until the wall reached a load of 91.6 kN (20.6 kips) “approximately 75% of the specimen's ultimate strength” (ElGawady et al., 2006). Step cracks were formed after a lateral load of 101 kN (22.7 kips) was reached; the load was then increased to 122.9 kN (27.6 kips) when the wall failed. This is nearly a 350% increase from the reference wall. The last wall tested was the wall reinforced on both sides. Hair cracks first started to appear at a lateral load of 53.8 kN (12.1 kips) but the main visible cracks appeared at a load of 82.9 kN (18.6 kips). The wall finally failed once "an average peak force of approximately 126.2 kN (28.4 kips) and a drift of 0.78%, the toe and heel were heavily damaged and the reinforcement bars ruptured without any yield" (ElGawady Et al, 2006).

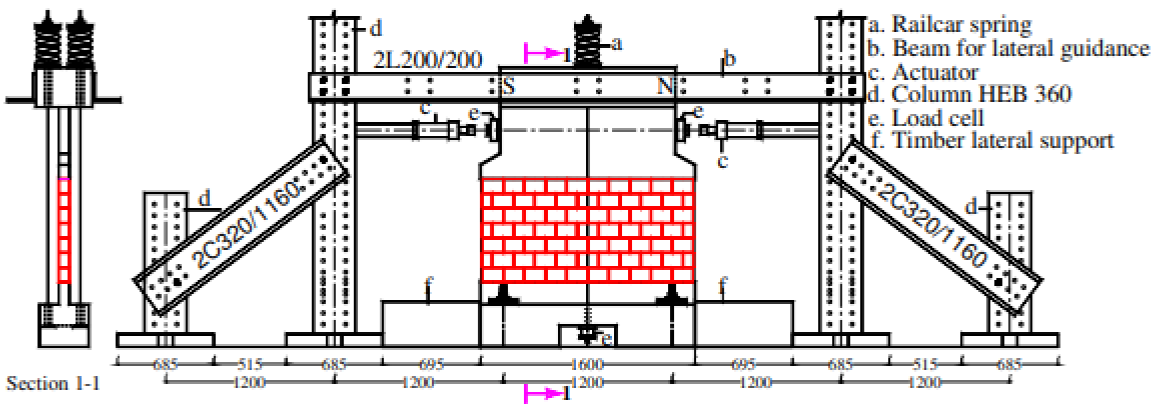


Figure 26: Test Setup

Source: (ElGawady et al, 2006)

In conclusion, the two walls that were reinforced both experienced a great increase in strength in comparison to the unreinforced wall. The method of retrofitting using shotcrete increased the lateral strength of the walls by a factor of around 3. The study proved that using this retrofitting method is a great way of reinforcing unreinforced masonry buildings for seismic events.

Chapter 4: Case Studies

This chapter will present real life examples where retrofitting methods were used on buildings. The chapter will discuss Gruening Middle School in Eagle River, Alaska and several unreinforced and retrofitted buildings that were affected by the 2010 Darfield Earthquake which struck Christchurch New Zealand. In 2018 an earthquake hit Gruening Middle School, causing the school to be closed for three years while retrofit took place. The 2010 Darfield Earthquake will cover the performance of buildings that were unreinforced and buildings that had been retrofitted prior to the earthquake.

Gruening Middle School

The information gathered for this case study comes from Hamel's article titled "*Seismic repair and retrofit in Alaska*" (2023). On November 30, 2018, an earthquake struck the town of Eagle River, Alaska. The earthquake had a magnitude of 7.1. The earthquake's epicenter was just 11 miles from Gruening Middle School. The earthquake caused significant damage to the school and forced the school to be shut down for nearly three years while seismic upgrades and repairs were done. The school was originally constructed in 1981 and is a two-story, 124,000 square foot building. The building utilizes CMU shear walls on the interior. The completed project can be seen below (Figure 27).

Figure 27: Gruening Middle School Completed
Source: (Hamel, 2023)

Gruening was first evaluated by engineers on December 2, 2018. In the following three weeks it would be visited three more times and would result in the building being red tagged. The damage that was observed was "…cracked masonry, leaning of the two-story CMU gym wall, stairway to column connection damage, bent gym curtain support beam, cracked drywall, acoustic ceiling tiles failures, and gypsum-board ceiling failures" (Hamel, 2023). Repairs of the minor masonry cracking and ceiling tiles began as soon as possible. The leaning two-story gym wall was shored.

Before any retrofit or upgrades took place, the first correction to be made was the removal of the exterior masonry veneer. The removal of the veneer reduced the seismic weight greatly. With the removal of the masonry veneer, a new metal siding was used. The veneer weight alone was 63 pounds per square foot, while the new siding weight was only five pounds per square foot. This provided a reduction of 1,200 tons of seismic weight. The addition of the new siding also gave the school an updated look.

The existing CMU walls were reviewed with current CMU wall design, and it was found that the reinforcement in the walls at the school were spaced at 32-inches vertical and 48-inches horizontal. This does not meet the minimum spacing requirements of the Masonry Standards Joint Committee (MSJC). The school utilizes special reinforced masonry shear walls, because it is a requirement for the seismic design category of the building. The MSJC states that a minimum spacing of 24-inch vertical and horizontal reinforcement is required. It was also found that the unbraced CMU walls that had a height greater than 17 feet were lacking proper out-of-plane bending capacity.

After assessing all these problems, it was determined to use a fiber reinforced cementitious matrix (FRCM). The engineers found that using FRCM would, "…upgrade existing stack bound CMU walls to meet current MSJC reinforcing requirements and strengthening tall walls to meet out-of-plane bending capacity requirements." (Hamel, 2023). FRCM is similar to shotcrete but instead of using a steel mesh it uses a carbon fiber grid. This method would not add significant weight or volume to the project. This was the first time that FRCM was used in the state of Alaska. The application of the FRCM went as follows: "Remove finishes from single (or both) sides of masonry shear walls as noted on plans. Remove paint from the surface. Fill the existing CMU flutes and place the fabric matrix. The anticipated added thickness to the CMU wall is a maximum of ½ inch. Trowel finish and paint as shown on Architectural Drawings" (Hamel, 2023). FRCM was used to cover around 36,000 square feet of wall. Most of the FRCM was single sided except for the tall walls. It was required on both sides to assist with out-of-plane bending reinforcing.

Although the school has not seen a significant earthquake since it reopened, there should be no doubt that it will not fail with the addition of its new retrofit.

2010 Darfield Earthquake

The information gathered for this case study comes from Dizhur's article titled "*Performance of unreinforced and retrofitted masonry buildings during the 2010 Darfield earthquake*" (2010). On September 4th, 2010, a 7.1 magnitude earthquake struck 40 km away from Christchurch City, New Zealand. Due to this earthquake, many of the URM buildings in the area were severely damaged with many being red-tagged. In this case study there are six examples of the damage that these URM buildings suffered. There are also six examples of buildings that had been retrofitted prior to the earthquake and how they performed.

After inspection, it was observed that "Of the 595 URM buildings that were assessed in Christchurch following the Darfield earthquake, 21% were assessed as unsafe and tagged red, 32% were assessed as yellow – limited accessibility, requiring further inspection, and 47% were assessed as green – safe for public use" (Dizhur et al., 2010). It was noted that out of the 595 buildings, 395 of them were rated as having 10% or less damage. The rest experienced damage greater than 10%. It was found that during the earthquake, many parapets failed causing further damage due to falling bricks and large masonry sections. The falling parapets caused the canopy braces which hold the canopies above the street front of the building, to pull through the masonry.

The following buildings had not been retrofitted prior to the earthquake. The two-story Caxton Press building experienced severe damage. The parapets on the façade wall and the top of the gabled side walls had failed due to out-of-plane loading. It was also observed that the façade developed shear cracks through the spandrel over the openings of the building and that the façade had pulled away from the side walls due to insufficient anchorage. The 135-139 Manchester Street building (Figure 28a) is a three-story “L” shaped building. The front façade had entirely collapsed out-of-plane. There was damage due to collapsed parapets except for two bays where the parapet still remained. The collapsed façade revealed severe water damage to the floor joists and roof rafters. The steel through-anchors around the gable were seen with the brickwork pulling away from the anchor plates. The parapet landed on the canopies below resulting in the canopy collapsing. The Welstead House (Figure 28b) is a two-story rectangular building. The damage observed was a complete out-of-plane collapse of the façade walls. Due to the extensive damage of this building, it was eventually demolished. The 192 Madras Street building is a two-story URM building. The spandrels on the first floor and the roof had extensive cracking. The façade appeared to move at the diaphragm. The side walls were shown to have suffered diagonal shear failures. The parapet had remained attached because of the support from masonry columns that extended up to the corners of the building. The Cecil House (Figure 28c) is an “L” shaped building with two-stories. The damage observed was that the parapet had toppled around the street front. The parapet landed on the canopies below, which caused the tension braces to overload and caused a punching shear failure in the masonry wall. There was no evidence of through-anchors connecting the roof diaphragm to the wall that could be observed. The 463-469 Colombo Street building is a two-story “L” shaped building. The parapet had completely collapsed. The external veneer had collapsed and revealed wire ties that were in good condition with just a few ties appearing to have rust.

(a) (b) (c)

Figure 28: (a) 135-139 Manchester Street (b) Welstead House (c) Cecil House

Source: (Dizhur et al., 2010)

Many buildings had been retrofitted prior to the earthquake. It was observed that the buildings that had been retrofitted perform very well with the only prevalent damage was either partial or complete collapse of parapets and chimneys. Because of the heritage of the URM buildings that had been retrofitted, the methods of retrofit had been the minimally invasive methods. The most common was the addition of a secondary structural system, with just a few using alternate solutions such as steel strapping and the use of fiber reinforced polymers (FRP).

The Malthouse building was retrofitted using grout injections, installation of new wall to diaphragm anchors, and strengthening the diaphragm by adding blocking and sheathing. The building performed well and there were no signs of damage that could be observed. The Smokehouse building is a two-story building. The building was retrofitted in 2007 using steel moment resisting frames. The retrofit also "…won the New Zealand Architectural Award in 2008 for initiative in retention, restoration and extension of a significant building and its adaption to new uses." (Dizhur et al., 2010). The building performed well with no damage observed on the exterior of the building. But there were some minor cracks that spanned vertically at the wall corners and also one minor crack that spanned horizontally above the base of a pier at the second floor. The X Backpackers building is a four-story building. The building was retrofitted in 2001 using steel moment resisting frames (Figure 29b) along with the parapet being tied to the roof structure. There were minor internal cracks, and large cracks on the interior at the top of the façade which was found to be due to poor wall to diaphragm anchorage. The TSB Bank Building (Figure 29a) is a three-story building that is isolated. The building had been retrofitted just the year prior to the earthquake in 2009. The retrofit was the inclusion of secondary frames with the façade being strengthened by concrete columns and beams while the side walls were strengthened with a steel frame with diagonal braces that are anchored into the masonry. The diaphragms of the building had been strengthened with new plywood sheets. The walls of the building are supported with new concrete beams at the basement level. The building performed very well with the only damage being minor vertical cracks in the basement beams. Joe's Garage Café building is a two-story isolated building. The building was retrofitted in 2004 using steel portal frames only in the transverse direction, along with the diaphragms that were not strengthened. Because of the steel frames being placed in one direction, there was a strong torsional response which caused diagonal shear cracks in the rear wall of the building. There was some localized punching of the bricks due to the steel frame pounding against the wall. The Shirley Community Center building is a single-story building. Seven walls were strengthened using surface FRP (Figure 29c). Also, steel strong backs were added to assist in the out-of-plane stability around the perimeter wall. The veneer façade was secured to the main wall using veneer ties. The building performed well with no significant damage. There was minor cracking that had been observed at the ceiling and wall to ceiling levels. No cracking was observed on the FRP sheets. A minor step crack was found in the veneer on the exterior of the building.

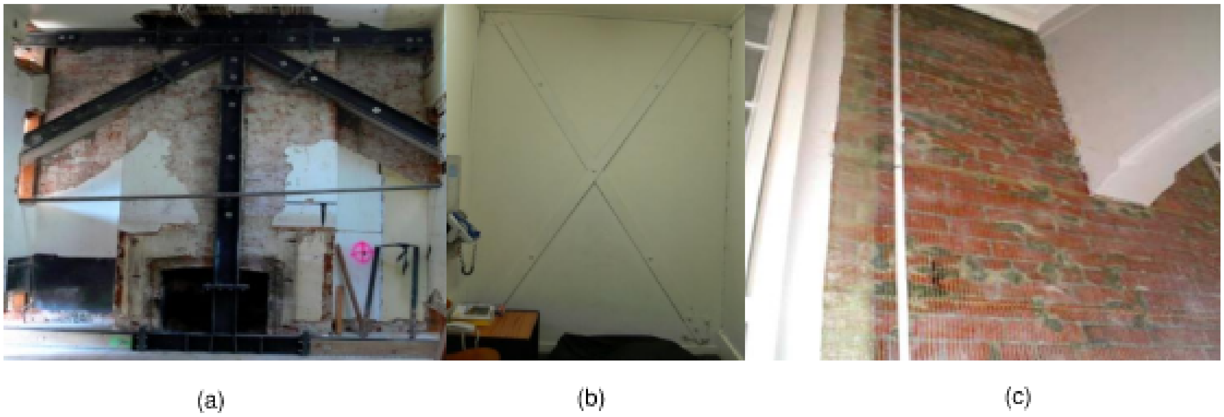


Figure 29: (a) TSB Bank Building Steel Brace Frame (b) X Backpackers Steel Brace Frame (c) Shirley Community Center FRP

Source: (Dizhur et al., 2010)

In conclusion, it was found that the URM buildings that had not been retrofitted had performed as expected during a seismic event. The most common failures being: "…parapet failure, chimney failure, out-of-plane facade wall failure and in-plane damage." (Dizhur et al., 2010). On the other hand, the buildings that had been retrofitted saw very little damage and performed very well.

Chapter 5: Conclusion

Masonry is one of the oldest and most popular building materials to exist. It can be seen all around the world in many historical cities and historical landmarks. With the steel reinforcement required for seismic design of masonry buildings that came around in the 1940s, many masonry buildings lack proper reinforcement and are vulnerable to earthquakes. There were many historical failures that have caused injuries and even deaths because of the lack of reinforcement. Many retrofitting methods have been developed to strengthen historical unreinforced masonry buildings.

This report discusses multiple methods of retrofitting. Many unreinforced masonry buildings lack proper diaphragm ties. Adding diaphragm ties is critical to complete lateral load path. With the addition of diaphragm ties, the unbraced length of masonry walls can also be reduced, resulting in a smaller out-of-plane moment and assisting in stability of the wall during a seismic event. Two different ties were discussed: tension ties and shear ties. The tension ties are used to keep the diaphragm attached to the wall, while the shear ties are designed to transfer lateral loads to shear walls. The diaphragm ties can also work with other retrofitting methods.

Carbon fiber reinforced polymer (CFRP) is another effective method of retrofitting URM's. CFRP is an extremely strong and lightweight material. In the experimental study, the walls that had been retrofitted with CFRP showed a significant increase in initial cracking and ultimate strengths compared to unreinforced masonry walls, with a minimum of 200% increase. The main drawbacks are the cost and the altering of architectural aesthetics. Factoring in the high maintenance cost and initial installation of the CFRP will add to the total cost.

During an earthquake, parapets are prone to failure and can cause secondary failure of the building. Some local governments require unbraced parapets to be retrofit in their building codes. Bracing parapets is very important and effective retrofitting method for URM's. It is typically done with a steel angle, or wood for a more cost-effective method. The bracing will be connected to the roof diaphragm of the building and to a spreader bar attached to the parapet. In an experimental study, steel, wood bracing, and wooden vertical strong backs were tested. It was shown that a significant increase in strength was achieved in braced parapets compared to as-built parapets. The addition of the bracing was able to increase the strength of the parapets by as much as a factor of 8 compared to unbraced parapets.

Another method of reparing URM's is grout and epoxy injections. This method has great popularity because it's easy to implement and of low cost. The method is primarily used to fill cracks and voids. It has been proven to be effective in repairing masonry structures and returning them to a stronger state. This method will not alter the aesthetics of a masonry building. Since it does not add reinforcement, this method does not increase URM wall strength significantly.

Out-of-plane wall bracing can be added to URM walls. This method consists of placing either a horizontal or vertical member on the wall. A horizontal member is typically a steel channel or square tube that is attached to the wall and then has a diagonal member that connects to the diaphragm above. A vertical member can be either steel or timber, with timber being a more cost-effective method. It can be placed on either the inside or outside of the wall. The member will assist in absorbing either the compression or tension load that the building will experience.

Shotcrete can be used to greatly strengthen a URM wall. Shotcrete is installed on one side or both sides of the wall after placing a steel mesh on the side(s). The mesh is connected to the wall by drilled dowels spaced evenly along the surface of the wall. The shotcrete is then applied on the face of the wall and fully covering the mesh. This method is great for the addition of steel reinforcement because the reinforcement is near the surface where the tensile stresses are the maximum. A drawback of this method is that it adds a significant amount of seismic mass causing the seismic load to be higher.

Real life examples were discussed where retrofitting methods were utilized, including Gruening Middle School in Alaska and several buildings located in Christchurch New Zealand. Gruening Middle School underwent repairs and retrofit after a 7.1 magnitude earthquake. The retrofit method done to the CMU shear walls was the use of fiber reinforced cementitious matrix (FRCM). The study on unreinforced masonry buildings in Christchurch, New Zealand compared the performance of multiple un-retrofitted and retrofitted URM structures during the Christchurch earthquake in 2010. The study showed that the un-retrofitted buildings had either serious damage or had to be demolished after the earthquake while the retrofitted buildings had very little to no damage at all. It proved the effectiveness of these retrofitting methods.

References

Applied Technology Council. (2011). 6.3 Architectural Components. In *Reducing the risks of nonstructural earthquake damage: A practical guide* (Vol. FEMA E-74). essay.

Areta. (2020, September 1). *Holding together historical buildings: The anchor plates of Lviv*. Forgotten Galicia. Retrieved April 14, 2023, from https://forgottengalicia.com/anchor-plates-lviv/

Calvi, G., & Magenes, G. (1994). Experimental results on unreinforced Masonry shear walls damaged and repaired. kipdf.com. Retrieved October 20, 2022, from https://kipdf.com/experimental-resul-ts-on-unreinforced-masonr-y-shear-w-alls-damaged-and- rep-aire_5ad2c2997f8b9a30038b464d.html

Carbon fiber structural repair exterior block wall. CST Stabilization. (2020, May 1). Retrieved March 2, 2023, from https://cststabilization.com/project/carbon-fiber-structural-repair-exterior-block-wall/

Carbon Fiber vs Steel: Who does it better? ZOLTEK. (2021, November 23). Retrieved February 8, 2023, from https://zoltek.com/carbon-fiber-vs-steel/

Cassol, D., Giongo, I., Ingham, J., & Dizhur, D. (2021). Seismic out-of-plane retrofit of URM walls using timber strong-backs. *Construction and Building Materials*, *269*, 121237.

Dizhur, D., Ismail, N., Knox, C., Lumantarna, R., & Ingham, J. M. (2010). Performance of unreinforced and retrofitted masonry buildings during the 2010 Darfield earthquake. *Bulletin of the New Zealand Society for Earthquake Engineering*, *43*(4), 321–339. https://doi.org/10.5459/bnzsee.43.4.321-339

ElGawady, M. A., Lestuzzi, P., & Badoux, M. (2006, March). Retrofitting of masonry walls using shotcrete. In *2006 NZSEE Conference, Paper* (Vol. 45).

Elmalyh, S., Bouyahyaoui, A., Cherradi, T., Rotaru, A., & Mihai, P. (2020). Shear Strength of Unreinforced Masonry Walls Retrofitted with CFRP. Advances in Science, Technology and Engineering Systems Journal, 5(2), 351-359.

Giaretton, M., Dizhur, D., & Ingham, J. (2018). Shake table testing of seismically restrained clay-brick masonry parapets. Earthquake Spectra, 34(1), 99-119.

Gkournelos, P. D., Triantafillou, T. C., & Bournas, D. A. (2022). Seismic upgrading of existing masonry structures: A state-of-the-art review. Soil Dynamics and Earthquake Engineering, 161, 107428.

Hamel, E. (2023, March). *Seismic repair and retrofit in Alaska*. STRUCTURE magazine. Retrieved April 10, 2023, from https://www.structuremag.org/?p=23005

Jamal, H. (2017, April 26). Haseeb Jamal. Shear, Diagonal, Non-Structural, Overturning Failure. Retrieved November 13, 2022, from https://www.aboutcivil.org/failure-modes-masonry-structures.html

Jarus, O. (2022, October 6). Pyramids of giza and the Sphinx: Facts about the ancient Egyptian monuments. LiveScience. Retrieved March 2, 2023, from https://www.livescience.com/22621-pyramids-giza-sphinx.html

Johnson, T. (2019, April 1). What are CRFP composites and why are they useful? ThoughtCo. Retrieved April 21, 2023, from https://www.thoughtco.com/understanding-cfrp-composites-820393

Lönhoff, M., & Sadegh-Azar, H. (2018, September). *Seismic out-of-plane behavior of unreinforced masonry walls*. Wiley Online Library. Retrieved March 22, 2023, from https://onlinelibrary.wiley.com/doi/epdf/10.1002/cepa.844

Mark, J. J. (2016). Great Pyramid of Giza. World History Encyclopedia. Retrieved November 12, 2022, from https://www.worldhistory.org/Great_Pyramid_of_Giza/

Oyguc, Resat & Oyguc, Evrim. (2017). The 2011 Van Earthquakes: Lessons from Damaged Masonry Structures. Journal of Performance of Constructed Facilities. 31. 10.1061/(ASCE)CF.1943-5509.0001057.

Reitherman, R., & Perry, S. (2009). Unreinforced masonry buildings and earthquakes: Developing successful risk reduction programs. (K. Yawitz, Ed.). Federal Emergency Management Agency, NEHRP

Retrofitting masonry walls using CFRP. Retrofitting Masonry Walls Using CFRP. (n.d.). Retrieved November 13, 2022, from https://www.rhinocarbonfiber.com/retrofitting-masonry-walls-using-cfrp

SehuJler, M.P., Atkinson, R.H., & BorgsrniJler, J.T. (2015). INJECTION GROUTING FOR REPAIR AND RETROFIT OF UNREINFORCED MASONRY.

U.S. Department of the Interior. (2022, December 22). History & culture. National Parks Service. Retrieved March 2, 2023, from https://www.nps.gov/wamo/learn/historyculture/index.htm

U.S. Dept. of Homeland Security, FEMA, NEHRP. (2006). Ch 21 Building Type URM: Unreinforced Masonry Bearing Walls. In *Techniques for the seismic rehabilitation of existing buildings: FEMA 547--october 2006*. essay.

World's largest free standing masonry building: The Philadelphia City Hall sets world record. 2023 World Records: UNLIMITED categories, FASTEST 24/7/365 services, LIFETIME listing! (n.d.). Retrieved January 26, 2023, from https://www.worldrecordacademy.org/2022/05/worlds-largest-free-standing-masonry-building-the-philadelphia-city-hall-sets-world-record-422194

24K Carbon Tow. (n.d.). FibreGlast. photograph. Retrieved from https://www.fibreglast.com/product/24K_Carbon_Tow_2293.