Application of Traditional Materials in Non-Traditional Ways for Improved Housing Construction

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Abstract

The role of alternative materials, such as precast concrete and masonry, for housing structures is discussed in this paper. Following a brief discussion on the potential advantages of concrete and masonry and a summary of the current state of the art with regards to these materials, research conducted at the University of Auckland and North Carolina State University on post-tensioned masonry is discussed in detail. Research results indicate that structures designed with post-tensioning benefit from a self-corrective nature, which is beneficial for natural hazard conditions due to earthquakes and hurricanes. The paper concludes with a discussion on future research needs in the area of post-tensioning for housing structures.

Keywords: Post-tensioned masonry, confined masonry, seismic design, cyclic wall response, in-plane wall response.

Introduction

It can be argued that future innovations in structural engineering may largely be based on development of new materials. In the case of housing structures, the objectives of the PATH program would seem to indicate that the ideal housing structure should be (1) Strong, (2) Ductile, (3) Fast, safe, and inexpensive to construct, (4) Durable, (5) Low maintenance, (6) Energy efficient, and (7) Inexpensive to purchase. This is a rather impressive set of goals that might lead one to believe that a system and material that possesses all of these characteristics is currently not available. While research on new materials is essential to advance the state of the art, it is important that traditional materials not be overlooked, particularly if they can be utilized in a manner where their strengths are mobilized, and their deficiencies addressed.

While wood-frame construction is by far the most common form for residential construction, it is the opinion of the authors of this manuscript that the future of housing structures rests not in timber systems, but rather structures designed almost entirely from precast concrete, masonry, and steel-based materials. This statement is offered in the spirit of this NSF workshop to stimulate discussion, and there are certainly many arguments against the use of these materials for housing. Chief among them is potential cost of materials, and a labor force that is not accustomed to dealing with residential structures whose load-bearing elements are constructed from materials other than wood. Indeed, widespread application of concrete, masonry and steel would be impossible without training of the labor force, although in the case of precast elements, similarities with wood structures abound.

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While there are several practical issues related to implementation of precast concrete and masonry for housing structures, this paper will focus on the structural performance issues. Specifically, performance under extreme events such as hurricanes, tornadoes, and earthquakes. It also focuses largely on the application of post-tensioning to concrete and masonry housing structures. Through the use of post-tensioning, the deficiencies of concrete and masonry may be addressed. In addition, there is a potential significant enhancement in performance as well.

Described first in this paper is a discussion regarding the current state of the art in concrete and masonry housing structures. This is followed by a discussion of how the application of post-tensioning to materials such as masonry can significantly improve performance. Following this, research from around the world is reviewed, including the testing conducted at NC State on clay brick and concrete block structures. Lastly, the paper concludes with a discussion of future needs in this area.

Current State of The Art

Currently, the typical role that concrete and masonry play in housing structures is largely aesthetic. Typically, applications of masonry are as façade elements over wood frame construction which forms the primary load bearing elements. In the west coast, stucco is favored over masonry as the façade material. In any case, the construction must employ cavity wall details in order to ensure proper performance under moisture demands. Also, the performance of cavity wall systems during earthquakes can be questionable. An alternative to this is to employ the pre-cast concrete or masonry as the load bearing and aesthetic elements. In so doing, one greatly simplifies the construction process. Furthermore, by application of post-tensioning, the possibility exists to significantly improve lateral load performance as well as uplift performance during high wind events.

Historic use of Post-Tensioning in Housing Structures

Though the concept of prestressed masonry has been around for decades, the codification of prestressed masonry as a standard construction technique has only begun in more recent years. This does not suggest an absence of prestressed masonry applications earlier in the twentieth century. Many projects using clay brick and calcium silicate masonry, typically in Europe, have been documented for example see Ganz (2003). The use of post-tensioned masonry in residential projects is mostly limited to low seismic areas of the US and typically involved one of three readily available prestressing systems. The PROTO-II™ wall system uses post-tensioning to provide out-of-plane strength for fences and retaining walls and according to company literature the system has been used in over ten thousand miles of wall over the past two decades (Proto II™ Wall Systems, 2003). The Dur-O-Wal Sure-Stress™ system is a post-tensioning system that can be used with any mass-produced concrete masonry unit and applied to any situation where the benefits of prestressing are required (Dur-O-Wal, 1999). The Integra wall system is probably the most widely used concrete masonry post-tensioning system in the US and has been implemented in approximately 15,000 homes since production began in 1984 (Integra Wall Systems, 2003).

Though it would seem that the number of projects involving post-tensioned masonry is vast, the use of prestressed masonry in highly seismic areas, such as California or New Zealand, is much more limited. One such example was the use of post-tensioned concrete masonry (PCM) in New Zealand by Hanlon (1970), who began experimenting by building several one and two storey structures before constructing a six-storey apartment building in Christchurch.

Research On Post-Tensioned Masonry Housing Elements

As discussed in the introduction, the motivation for the use of post-tensioning in masonry walls revolves around enhanced construction efficiency, cost, and most importantly, lateral force performance. Although the emphasis here is on seismic behavior, the details used could be readily employed in regions of low to moderate seismicity, as well advancing the behavior due to other lateral loads such as wind.

PCM Research at the University of Auckland, New Zealand

A research team from the University of Auckland has been investigating the in-plane seismic performance of PCM walls since 1995. This investigation has demonstrated the favorable characteristics that such walls have over their conventionally reinforced equivalents, such as increased in-plane strength and the absence of residual post-earthquake wall displacements. Wall damage typically involves masonry crushing of the wall bottom corners, and this can be easily repaired, thereby reinstating original wall strength and stiffness (Wight et al., 2002). A number of additional performance enhancers have been trialed with encouraging results. For example the use of confinement plates improves the strain capacity of the masonry, resulting in increased maximum lateral wall drifts. High strength masonry units in the lower wall corners also enables greater drifts to be achieved before masonry crushing, and the addition of reinforcing bars provides additional hysteretic damping to a rocking system that would otherwise have very low levels of damping (Laursen and Ingham, 2003).

The current research phase involves developing a wall system that can be used in residential structures. A series of meetings were held between the research team and a number of industry personnel to identify the key requirements of a wall system for the residential market. The New Zealand residential market is very price driven, but currently there are clients who are willing to pay a small premium for the benefits that masonry houses offer, such as improved thermal characteristics. It is not known at this stage if prestressed concrete masonry will be cost equivalent to conventionally reinforced concrete masonry, however early indications suggest it will be similar due to the reduction of grout and mild steel required. It is anticipated that the house design will utilize the strength of prestressed panels in strategic locations in the external walls of the house, providing the bracing requirements. All other panels will be detailed to displace to the same levels as the prestressed panels, while not necessarily being relied upon to provide bracing capacity.

A joint study between the University of Auckland and North Carolina State University (NCSU) is currently underway to verify the dynamic performance of the proposed wall system. This involves shake table testing of a number of wall details typically found in a residential structure, for example rectangular panels, window and door openings and corners. A brief summary of the findings thus far are contained within the section entitled NCMA Sponsored Research on PCM at NCSU.

PATH Research at NCSU

As part of the NSF-Sponsored PATH study, a series of five large scale cantilever walls were tested in the Constructed Facilities Laboratory (CFL) at North Carolina State University (NCSU). All five walls were double wythe walls 1.22m long, 305mm thick and 2.44m tall to the center of seismic force. The walls were constructed from standard cored clay brick masonry units measuring 57mm high, 92mm wide, and 194mm long. All of the walls were post-tensioned with 3 equally spaced 25mm diameter dywidag bars. Test unit one represented the ‘control’ specimen and contained a fully grouted wall cavity. The tendons were unbonded, and no additional reinforcement was placed in the wall. Test unit two was the same as test unit one with the exception that confinement plates were placed in the compression toe regions of the wall to enhance the compression strain capacity. Test unit three was also the same as test unit one, with the exception that 4#4 grade 60 ASTM A706 mild reinforcing bars were placed near the end regions of the wall to provide some additional energy dissipation capacity. These mild reinforcing bars extended 914mm above the footing level and were unbonded over a length of 152mm below the footing interface and 457mm above the footing interface. Special considerations were needed in the design of this wall as too little reinforcing steel would not provide sufficient benefit, and too much would result in a large amount of residual deformation after cyclic loading. Test unit four was also the same as test unit one with the exception that the post-tensioning bars were bonded to the wall. The final test specimen was different from the first in that the wall was ungrouted, and as a result, the post-tensioning bars not laterally restrained. All five walls were post-tensioned with a target total force of 1000kN.

Test Results Summary

Test unit one represents the 'control' specimen, and contained unbonded post-tensioning. The wall was grouted, and unconfined. During the course of this test, it was observed that the behavioral mechanism by which such a wall deforms is one of 'rocking', which is consistent with observations made by Laursen and Ingham (2000a,b) on post-tensioned concrete masonry walls. A photo of the test unit at 6.5% drift is shown in Fig. 2a, while the force-displacement hysteretic response is shown in Fig. 3a. From Fig. 2a, note the rather extensive damage to the compression toe regions, however, the remainder of the wall above this region remained largely crack free. The integrity of the masonry above the plastic hinge region is very important from the perspective of achieving a stable compression strut as will be noted in the discussion of tests four and five

From the force-displacement hysteretic response of Fig. 3a, there are two characteristics that must be noted. First, the unbonded post-tensioned system is largely self correcting as residual displacement is close to zero for the majority of the load history. This observation is consistent with the observations made for post-tensioned concrete (Priestley et al., 1999) and post-tensioned concrete masonry (Laursen and Ingham, 2000a,b). Second, the amount of energy dissipation is rather limited, however, this is not a considered to be a fatal flaw as the lateral strength is generally dependable without strength degradation until a drift ratio of 3.75%. Furthermore, test unit three will investigate the use of mild steel as a means for supplemental energy dissipation.

Test unit two was identical to test unit one, with the exception of the addition of galvanized steel plates to the mortar bed joints in the bottom five courses of masonry. Application of confinement plates to masonry was first proposed by Priestley and Bridgeman (1974). The initial behavior of the test specimens was identical to that of test unit one. Initial crushing was noted at the same drift ratio of 0.5%. Beyond initial crushing, however, the behavior of the wall changed dramatically. Whereas unit one started significant strength degradation after 3.75% drift, the confinement of unit two stabilized the compression toe region allowing the wall to achieve a drift ratio of 10% without significant strength degradation. By stabilizing the compression toe region, the diagonal compression strut was able to sustain much higher levels of lateral deformation until finally, the compression toe region degraded, forcing the compression strut resultant outside of the confined region at a drift ratio of 11.3%. Fig. 2b represents the test specimen at a drift ratio of 6.75%. Compared to test unit one, very little damage is noted in the compression toe regions. The test specimen at 11.3% drift ratio is shown in Fig. 2c. The force-displacement hysteretic response is shown in Fig. 3b. Note the very stable loops with little residual deformation or loss of strength. The overall energy dissipation is rather low, consistent with the observation of test unit one.

The purpose of test unit three was to investigate the possibility of increasing the energy dissipation while retaining the desirable characteristics of little residual deformation of walls one and two. This test unit was identical to test unit one with the exception of the presence of mild steel to act as supplemental energy dissipation. Such an approach has been utilized for post-tensioned concrete structures (Priestley et al., 1999) and post-tensioned steel structures (Christopoulos et al., 2002), (Ricles et al., 2001) with some success. Fig. 2d represents a photo of the test specimen at a drift ratio of 6.5% which compares favorably to Fig. 2a for the control wall. The force-displacement hysteretic response is shown in Fig .3c. When compared to the control wall, it is noted that significant strength degradation did not occur until a drift ratio of 5.0%. It is also noted that the energy dissipated in the walls is greater than that from the control specimen. Since the wall was unconfined, once the masonry starts to crush, there is a rather rapid degradation of strength. Application of confinement would certainly enhance the performance of this configuration, however, the added benefit may not out way the added effort for designing and constructing this configuration.

The goal of test four was to most closely mimic the behavior of a conventionally reinforced wall. In doing so, the PT bars were fully bonded along the length of the wall after applying the PT force. Although it was expected that this wall would be damaged more than the unbonded walls, it was still expected to achieve similar deformation capacity. Unfortunately, that was not the case. Unlike the previous three walls, in addition to the base crack, minor flexural cracks due to the bond stresses between PT steel and masonry occurred at a drift ratio of 0.2%. These cracks were spaced at 250mm. Further loading resulted in masonry crushing at 1.0% drift. Soon after, at a drift ratio of 1.75%, vertical cracks were noted at the positions of the PT bars. These cracks occurred due to exceedance of the bond stress capacity between PT duct and surrounding grout. As the bond stresses increased resulting in vertical cracks, the behavioral mechanism started to shift away from flexural deformation back to the rocking mechanism observed in the previous three tests. Significant strength degradation was noted after 3% drift ratio, and at a drift ratio of 3.75% (Fig. 2e) damage was extensive in the wall. Due to this rather extensive damage, the diagonal compression strut was unable to stabilize, resulting in wall failure. The force-displacement hysteretic response for the wall is shown in Fig. 3d, and although there is higher energy dissipation than the control wall, the level of damage, reduced deformation capacity, and quick loss of strength rendered the performance of this wall poor.

The final test of the series was the simplest to construct, and the motivation for this test stemmed from simplicity of the configuration. Unfortunately, this turned out to be the poorest performing configuration. Although it behaved initially just as tests one through three, this configuration suffered a diagonal shear failure on a plane parallel to diagonal compression strut at drift ratio of only 1% (Fig. 2f). Due to the lack of a grouted core, it became impossible for a diagonal compression strut to form in the wall, and as a result, the rocking deformation was not achievable. Laursen and Ingham observed similar behavior in partially grouted concrete masonry walls (Laursen and Ingham, 2000a,b). The force displacement hysteretic response is shown in Fig. 3e. It is clear that such a configuration is not for anything but the lowest seismic regions.

NCMA Sponsored Research on PCM at NCSU

This research is funded by a grant from the National Concrete Masonry Association (NCMA) and involves the shake table testing of a number of wall specimens. This section provides a brief summary of the tests conducted to date. The wall designations follow the convention: D refers to dynamic shake table testing. L1.8 and H2.4 signify that the wall had a length of 1.8 m and a height of 2.4 m and the final four letters indicate the type of wall tested, where ‘Rect’ refers to a rectangular panel and ‘Door’ refers to a wall containing a door opening.

The wall materials used were the same for all specimens and had the following properties:

?? 150 mm (6 in.) concrete masonry units giving an actual width of 143 mm (5 5/8 in.).

?? The post-tensioning tendons used throughout this testing series were Dywidag threadbar, from the Dywidag (DSI) Formtie product range, with a nominal diameter of 16 mm, an effective cross-sectional area of 177 mm^2, a specified tensile rupture strength of 195 kN and a yield strength of 163 kN.

?? Grout used in all walls was batched at the laboratory using a mechanical mixer, and provided an average 28 day masonry prism strength of 22.7 MPa. Dramix ZL 30/50 zinc-plated steel fibres were used which had a length of 28 mm and hooked ends and Sika Grout Aid was added to the mix for all walls except the first, providing a slow controlled expansion prior to grout hardening.

?? A-706 mild reinforcing longitudinal steel was used in the bond beam, which is the seismic grade mild reinforcing steel in the US, and had a specified yield stress of 414 MPa (60 ksi). The stirrups used in the bond beam were A-615, 6 mm round wire stirrups, with the same specified yield stress.

Fig. 4 shows the three walls tested thus far, where the hatched area indicates cells that were grouted and the prestressing ducts are depicted as a clear space inside a hatched cell. It is noted that due to the dimensions of wall D:L2.2-H2.4-Door, the two side panels had to be fully grouted, since this system requires the cell containing the tendon and the two end cells to be grouted. Since testing of the door frame resulted in damage confined to the bond beam and lower corners, the bond beam was removed, the lower corners repaired and the larger panel tested as wall D:L0.8-H2.4-Rect. Therefore the only partially grouted wall tested dynamically thus far was wall D:L1.8-H2.4-Rect. The bond beam in all walls consisted of 2×No.5 (16 mm) bars, with one bar located in each of the top two block courses on opposite sides of the centrally located post-tension duct. Stirrups were spaced at 600 mm centres except when the bond beam formed a lintel as was the case for D:L2.2-H2.4-Door, where they were spaced at 165 mm on centre over the opening.

Test Results Summary

All walls were subjected to a variety of ground excitations of increasing intensity on the single degree of freedom shake table at NCSU. Fig. 5(d) shows the original ground acceleration spectra for the final run for each of the walls. To provide a means of comparison the design elastic spectra from the IBC (International Code Council, 2000) for San Jose, California and for flexible soil in a high seismic area in New Zealand (NZS 4203 1992) are also included. The additional seismic mass applied to the wall top was typically 2880 kg, which represents a loading of 3 kPa over a tributary length of 5 m. Fig. 5(a,b,c) show the force-displacement backbone curves for the three walls, which were derived using the equation of motion for a SDF system subjected to an external force. Recognizing that the velocity at the peak displacement must be zero, the force at peak displacement is simply the seismic mass multiplied by the total acceleration.

By plotting the force at the peak displacement of each wall oscillation throughout the duration of an earthquake record, a backbone curve can be fitted which encompasses all these points. Fig. 5(a,b,c) contain backbone curves for all ground accelerations run for each wall. It should be noted that any base sliding was subtracted from the top of wall displacements prior to plotting, and this was seen to be small compared with the top of wall displacements for all dynamic tests.

Initially a smaller seismic mass of 960 kg was installed on wall D:L1.8-H2.4-Rect and eight ground accelerations run. The wall sustained negligible damage and reached a peak drift of only 0.29%, therefore additional mass was added bringing the total up to 2880 kg. A further nine records were run, with the only damage sustained is shown in Fig. 6(a), being fine cracking of the mortar joint in the lower left wall corner. The force-displacement backbone curves for the nine runs with the larger mass are shown in Fig. 5(a), and indicate that all nine runs followed the same bilinear curve suggesting that there was no loss in wall strength or stiffness. Testing ceased due to mechanical problems with the shake table. This wall had an initial period of 0.11 sec, therefore Fig. 5(d) suggests that the wall was subjected to accelerations in excess of both design spectra.

For wall D:L2.2-H2.4-Door, the seismic mass was installed in two equal amounts of 1440 kg, one on top of each of the two wall panels, ensuring that the mass block did not stiffen the door lintel. The wall was subject to eleven records before testing was ceased due to considerable crushing of the wall panel lower corners, as shown in Fig. 6(b). Fig. 5(b) shows that the wall stiffness reduced throughout all the runs as a result of the accumulation of lintel cracking, but there was no significant loss of strength until the final run in the negative direction when the lower corner was crushed.

Wall D:L0.8-H2.4-Rect was subjected to thirteen ground excitations before testing ceased due to splitting of the repair mortar in the lower wall corners. Fig. 5(c) shows that the wall maintained initial strength and stiffness out to a drift limit of approximately 1% before corner crushing resulted in reduced stiffness for the final three ground excitations. Wall strength was not seen to decrease until the wall reached a drift of 2.0% in the negative direction and corner splitting occurred.

Future Research Directions

For a complete discussion of Phase I research, the reader is referred to (Rosenboom and Kowalsky, 2002), (Kowalsky et al., 2003). The second phase of the program is currently underway and involves the following components: (1) Shake-table testing of the most promising configuration from Phase I, namely, grouted, confined, unbonded walls. (2) Large-scale (cyclic and dynamic) testing and analysis of post-tensioned clay masonry walls with window openings. (3) Large-scale cyclic testing and analysis of "L" and "T" cross-section walls. (4) Development of cyclic section analysis software for analysis and design of post-tensioned walls. The third phase of the research will concentrate on practical applications and design details, namely, floor and roof diaphragm to wall connections and integration of non-structural components. The shake table testing of additional concrete masonry wall specimens is currently underway at NCSU. A rectangular wall, walls containing a window and a shrinkage control joint, and a small simple structure are to be tested in the first half of 2004. It is anticipated that the first post-tensioned concrete masonry home in New Zealand will be designed and construction commenced in late 2004.

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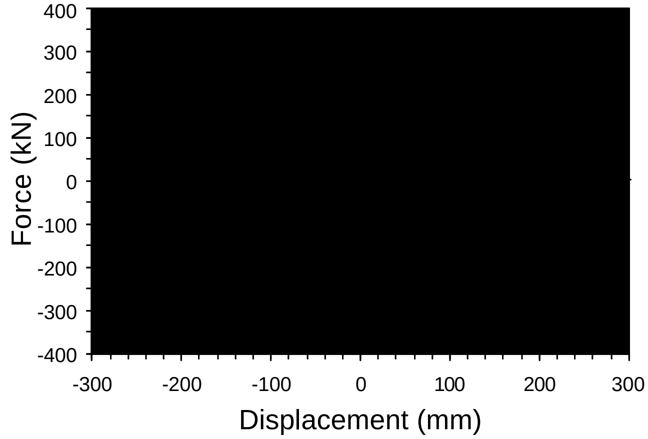
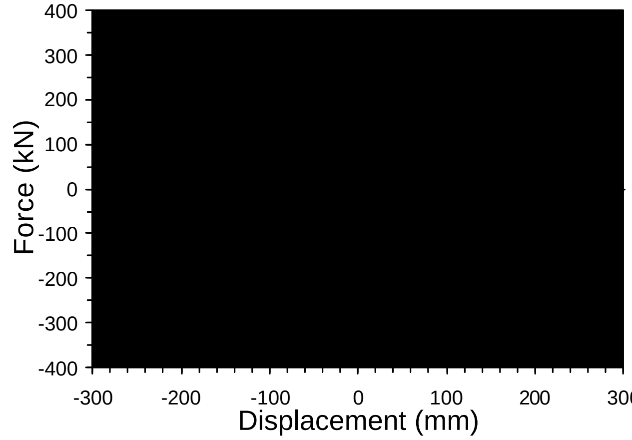
(a) Test 1: 6.5% Drift Ratio (b) Test 2: 6.5% Drift Ratio (c) Test 2: 11% Drift Ratio

(d) Test 3: 6.5% Drift Ratio (e) Test 4: 3.75% Drift Ratio (f) Test 5: 1% Drift Ratio

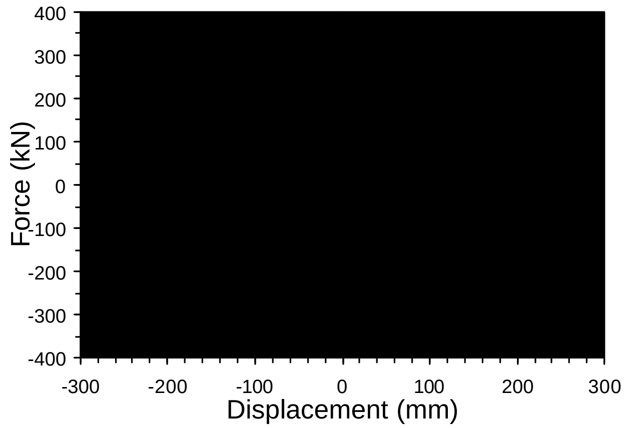
Fig. 1 Photos of test specimens during testing at various drift ratios



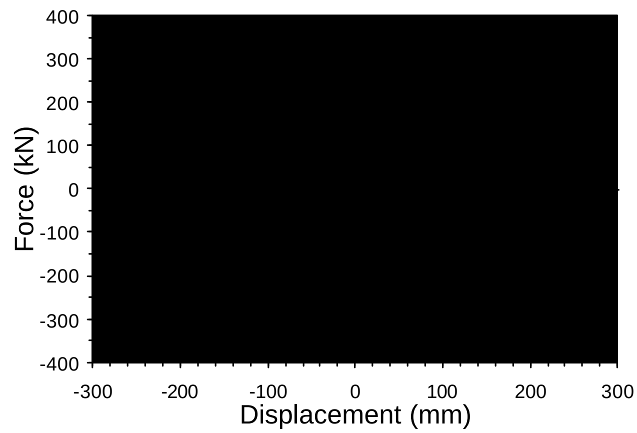
(a) Test 1: Grouted, Unbonded, Unconfined



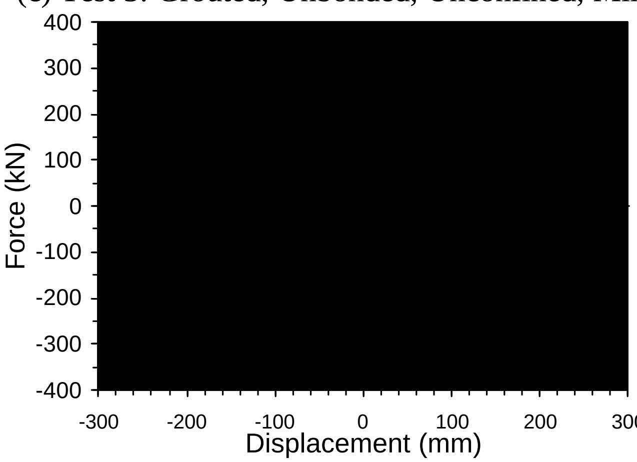
(b) Test 2: Grouted, Unbonded, Confined



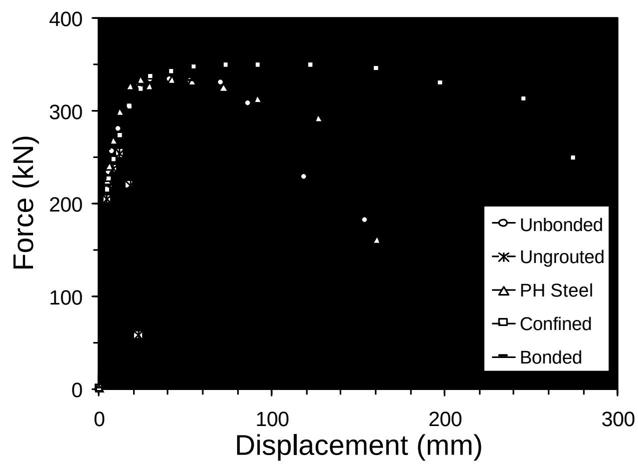
(c) Test 3: Grouted, Unbonded, Unconfined, Mild Steel



(d) Test 4: Grouted, Bonded, Unconfined

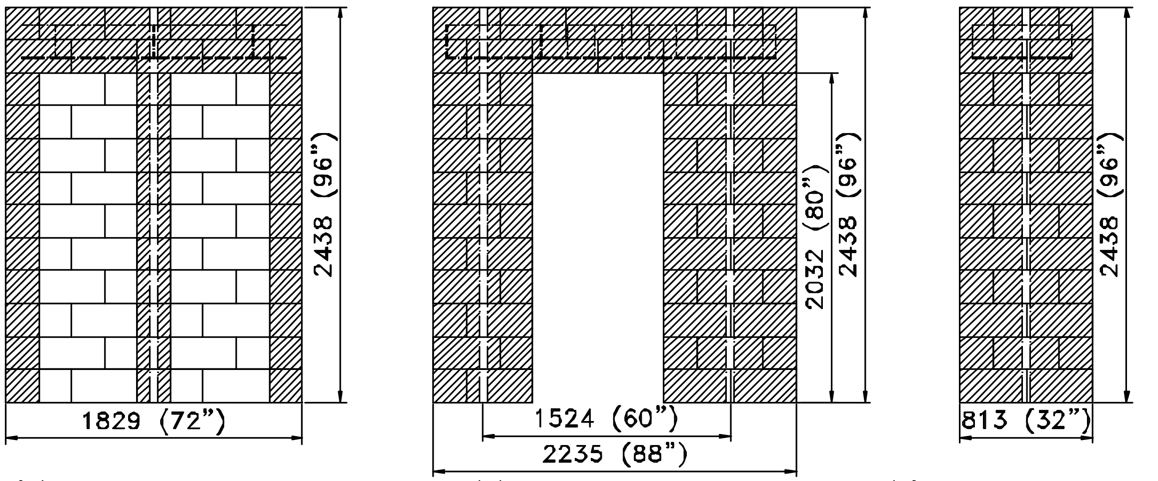


(e) Test 5: Ungrouted, Unbonded, Unconfined



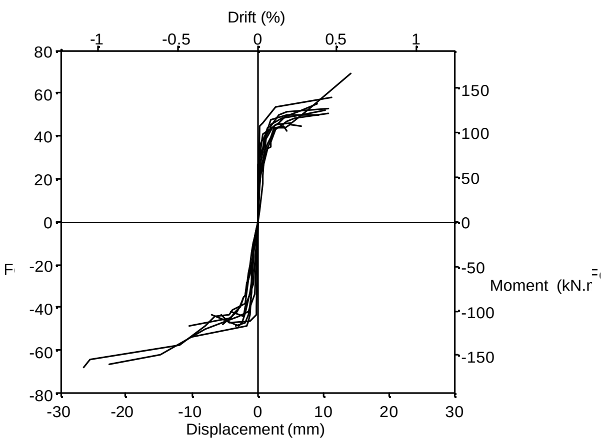
(f) Envelope responses (Average)

Figure 2. Force-displacement responses

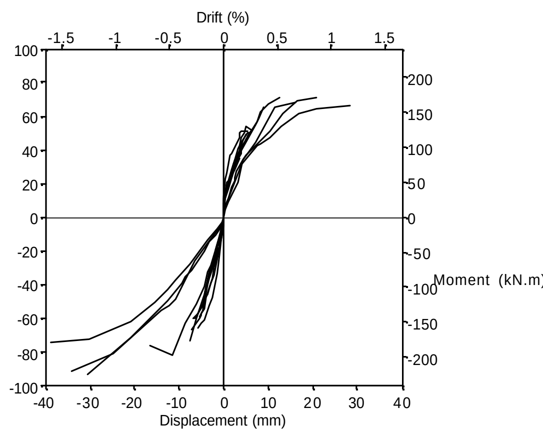


(a) D:L1.8–H2.4–Rect (b) D:L2.2–H2.4–Door (c) D:L0.8–H2.4–Rect

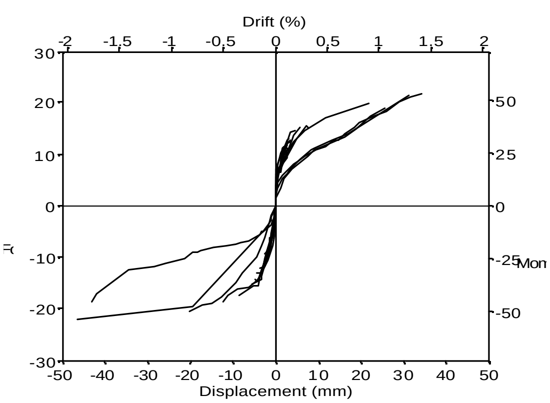
Figure 3. Dimensions of walls tested dynamically



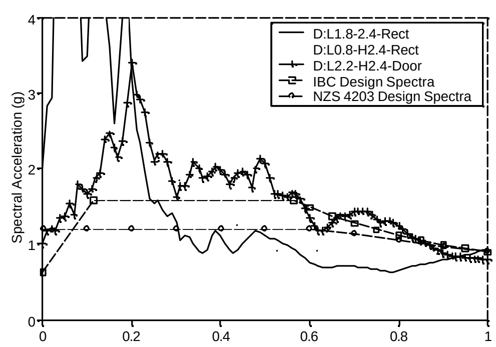
(a) D:L1.8-H2.4-Rect



(b) D:L2.2-H2.4-Door



(c) D:L0.8-H2.4-Rect



(d) Acceleration Spectra

Figure 4. Force-Displacement Histories and Ground Acceleration Spectra for Walls

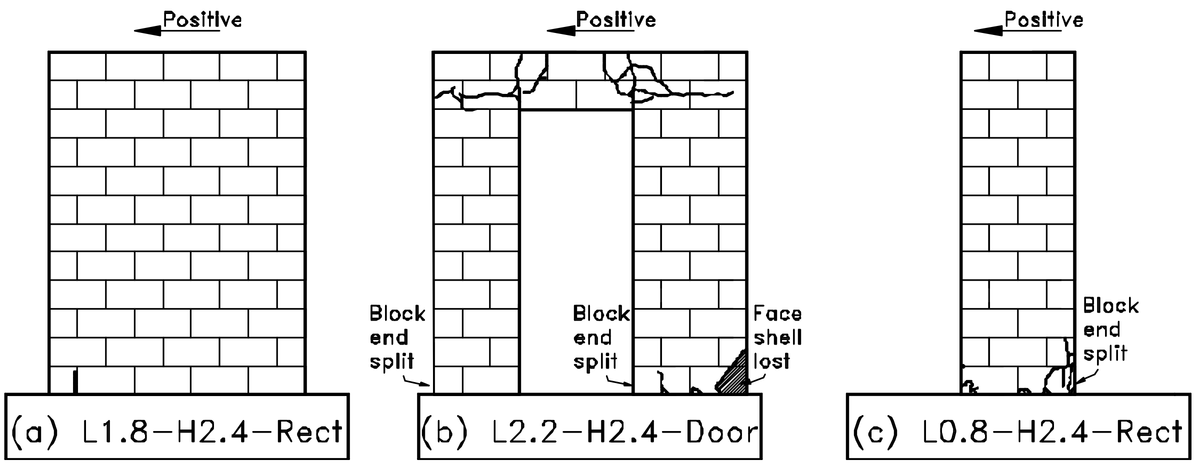


Figure 5. Crack Patterns at end of testing