REINFORCEMENT OF RESIDENTIAL MASONRY FOUNDATIONS TO MINIMIZE DAMAGE DUE TO MINING-INDUCED SUBSIDENCE[1]

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Abstract: The differential movement of the ground during longwall mine subsidence may damage residential structures by overstressing their structural members and by tilting the structures to unacceptable levels. Three subsidence mitigation techniques are commonly practiced in the United States: trenching to alleviate ground pressure buildup on the foundation masonry walls, bracing to reinforce masonry foundation walls, and jacking the superstructure off its foundation to minimize damage due to bending and racking. Despite these precautionary measures, the foundations usually need expensive repair or replacement owing to structural failure or tilt. Information on subsidence upgrade of existing residential structures is sparse, and few subsidence upgrade-methods are laboratory and field tested. As a result, the U.S. Bureau of Mines is investigating methods of protecting and reinforcing existing residential masonry basement foundations to minimize longwall mining subsidence damage.

This paper presents five full-scale in-plane bending tests that were conducted on a single, originally plain masonry wall. The purpose of the laboratory tests was to evaluate two postreinforcement designs that were proposed for existing residential foundations. The laboratory tests showed that the combination vertical and horizontal posttensioned tendons significantly increased the resistance of the masonry wall to in-plane bending.

Introduction

A majority of the literature available on the design of structures for protection against mine subsidence is for commercial, not residential, structures (Mautner 1948, Wasilkowski 1955, Healy and Head 1984). The designs are based upon the predicted final transverse profile of the subsided ground and do not take into account the dynamic ground movements that may be more severe and damaging (longitudinal and transverse profiles of a longwall panel). The National Coal Board's (NCB) subsidence prediction model is the most widely referenced in literature, but other subsidence models, such as Subside, SPASID, and SUBPRO, are available and are more appropriate for U.S. mining conditions (Ingram et al. 1989). The dated subsidence models usually only predict the final surface subsidence and strains for the transverse profile lines of longwall and/or retreat room-and-pillar sections, while recently developed prediction models provide three-dimensional dynamic ground movement data (Adamek et al. 1992). Simplifying assumptions are usually made concerning the soil-structure interaction, and the three-dimensional response of structures to mine subsidence is rarely reported.

Broad recommendations and guidelines have been made concerning the design and construction of residential structures in mine subsidence areas (Yokel et al. 1981, Basham et al. 1981). Suggestions for minimizing subsidence damage to existing structures are also made. Torsion of the foundation is rarely discussed, and methods of increasing the torsional rigidity of the foundation have not been pursued. No

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provisions are made in the proposed designs for leveling the superstructure and foundation following the subsidence event. None of the proposed designs have been tested in the field except for the response of a partly constructed reinforced masonry foundation (Powell et al. 1988). Laboratory and field testing of postreinforced crawl-space masonry foundations is reported in the literature (Marino 1992). The results are favorable for the crawl-space foundations tested.

Earthquake engineering research has produced a potential method of reducing subsidence damage to structures. The procedure involves the installation of an active spring system of support into the residential foundation (Hueffmann 1990). This system has only been used in Europe and no installations presently exist in the United States. The cost-effectiveness and practicality of retrofitting this system into a residential foundation are undetermined.

An investigation by the Bureau of subsidence insurance claims filed with the Commonwealth of Pennsylvania shows that 34 claims for longwall mine subsidence damage were processed during the years 1987-90. The cost to repair the residential structures as a result of structural damage and/or tilt exceeded $1.4 million. This figure would have been higher but the ceiling limit of $100,000 was imposed on two of the structures. In addition, a number of the residential structures were never releveled. Releveling the structures would have significantly increased the repair costs. It is estimated that the cost to repair or replace foundations damaged from longwall mine subsidence accounts for 70% to 85% of the total repair cost of residential structures (Motycki 1991).

The Bureau is conducting research to increase the limits of strength and deformation of residential basement foundations, since approximately 80% of the subsidence damage costs is attributed to their repair or replacement. Postreinforcement of existing residential foundations will increase their limits of strength and deformation and is a potential method of mitigating subsidence damage. Methods are also needed to reinforce foundations so that they may be releveled in a cost-effective manner rather than lifting the superstructure off its foundation and rebuilding a level foundation. The challenge is to develop postreinforcement methods that are cost-effective and feasible to retrofit onto an existing masonry foundation.

In this Bureau paper, five full-scale in-plane bending tests (hogging tests)[4] are described that were conducted on a single, originally unreinforced masonry wall at the Civil Engineering Structural Laboratory, University of Pittsburgh. The laboratory tests were conducted to evaluate two postreinforcement designs that were proposed for existing residential foundations. The types of steel reinforcement evaluated were vertical and horizontal posttensioned tendons (wire rope).

Experimental Test Program

Plans were initially developed for testing novel posttensioning designs for masonry walls under laboratory conditions. The motivation for posttensioned reinforcement is that it is an active, not a passive, reinforcement system. Active reinforcement systems are not dependent upon the deformations within a structure to mobilize resistance to internal loads as the structure reacts to applied loads and displacements.

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[4]A foundation subjected to a negative bending moment (designer's sign convention) is commonly referenced as a foundation that is hogging. A foundation that sags is subjected to a positive bending moment.

Vertical Tendons and Anchorage System

Prior to conducting in- and out-of-plane bending tests on a posttensioned masonry wall, an evaluation was made to determine (1) the feasibility of installing wire rope in the interior (cores of masonry units) of the masonry walls[5], (2) the optimum anchorage system for wire rope reinforcement, and (3) the availability of equipment that will meet the space limitations of the foundation-floor interface. Anchorage systems evaluated were different types of bonding agents and mechanical anchor-bonding agent assisted systems.

The selection of an optimum anchorage system was based upon a series of static pull tests and an evaluation of the ease and reliability of the installation. A concrete pad (1.83 m by 1.83 m by 0.15 m) with an average unconfined compressive strength of 33.66 MPa was poured and served as the host medium for the pull tests. The pad can accommodate 121 pull tests (holes were drilled on 15.2-cm centers.) The pad rests on a 5.1-cm semirigid foam layer (0.22 MPa crushing strength). The purpose of the foam is to provide a material that will offer minimal resistance to bottom-hole blowout. In general, when a hole is being drilled through concrete with a compressive strength less than 41.4 MPa, the bottom of the hole has a tendency to blow out from the percussion blows of the rotary hammer drill. Bottom-hole blowout will reduce the effective bonding length of the anchorage system and compromise its overall pullout capacity.

The first series of tests were conducted to observe the tendency for bottom-hole blowout during drilling and to evaluate three bonding agents (Ramset, Esco, and Rawl) for the wire rope to be used to posttension the masonry wall. Fifteen pullout tests were conducted to evaluate a ceramic-filled epoxy polymer, an acrylic epoxy, and a polyester resin as a bonding agent for the wire rope. Ten tests involved wire rope with straight ends, and five tests involved wire rope with bird cage ends, in which a portion of wire rope was unraveled to increase the wire rope-bonding agent area for increased bond strength. The tests showed that bottom-hole blowout occurred but was not a detrimental problem and that the bird cage ends prevented failure of the grout-steel interface, which was observed in a majority of the tests conducted on wire rope with straight ends. In all cases with the bird cage ends, the strength of the anchorage system exceeded the load-carrying capacity (107 kN) of the wire rope. Although the tests showed that the epoxies manufactured by Rawl and Ramset were basically identical, the decision was made to use the ceramic epoxy manufactured by Ramset.

The first method used to create the bird cage ends was labor intensive. Each individual strand needed to be cleaned since the wire rope is lubricated with grease during fabrication. A small residue of grease severely compromises the integrity of the wire rope-epoxy bond. Since cleaning the individual strands was so labor intensive, a new method of constructing bird cage ends for wire rope was developed. This method consists of making a bird cage end is first to crimp a compression ring onto the twisted ends of the wire rope to hold the individual strand ends together. Then place approximately 15.3 cm of wire rope in a lathe and twist the rope in an open position (permanent diameter of 3.8 cm) to form the bird cage. This method is much simpler and less labor intensive. The bird cage end is then thoroughly cleansed with a biodegradable organic solvent degreaser to remove all grease. In all cases, the strength of the anchorage system using this method to form the bird cage end exceeded the load-carrying capacity (107 kN) of the wire rope (fig. 1).

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[5]The use of steel cables was considered early into the project but the flexural rigidity of the cable eliminated their potential for use as vertical tendons. It was nearly impossible to force the cable into a tight radius so that it could be threaded down through the masonry unit cores.

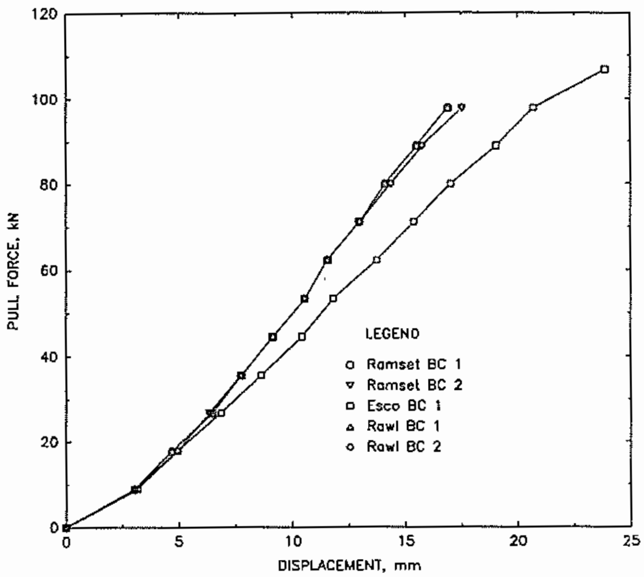

Figure 1. Pullout test results for wire rope with bird cage ends.

Masonry Wall

The overall dimensions of the hollow masonry foundation wall were 2.64 m high by 7.32 m long (fig. 2). The concrete footing was 50.8 cm wide, 20.3 cm high, and 7.32 m long and had an unconfined compressive strength of 33.66 MPa at 28 days. The wall was constructed of 194-mm two-core concrete masonry units with flanged ends. The masonry units were laid in running bond with type N mortar. Only the face shells and flanged ends were mortared. The masonry wall was built in the laboratory by an experienced mason using normal masonry construction practices and was air cured in the laboratory for 90 days before the structural testing program was initiated. The mass of the entire wall was 6,000 kg (estimated from hydraulic actuator pressures); the mass density of the concrete was 2,323 kg/m^3. The mass of the masonry assemblage per unit surface area was 238.3 kg/m^2.

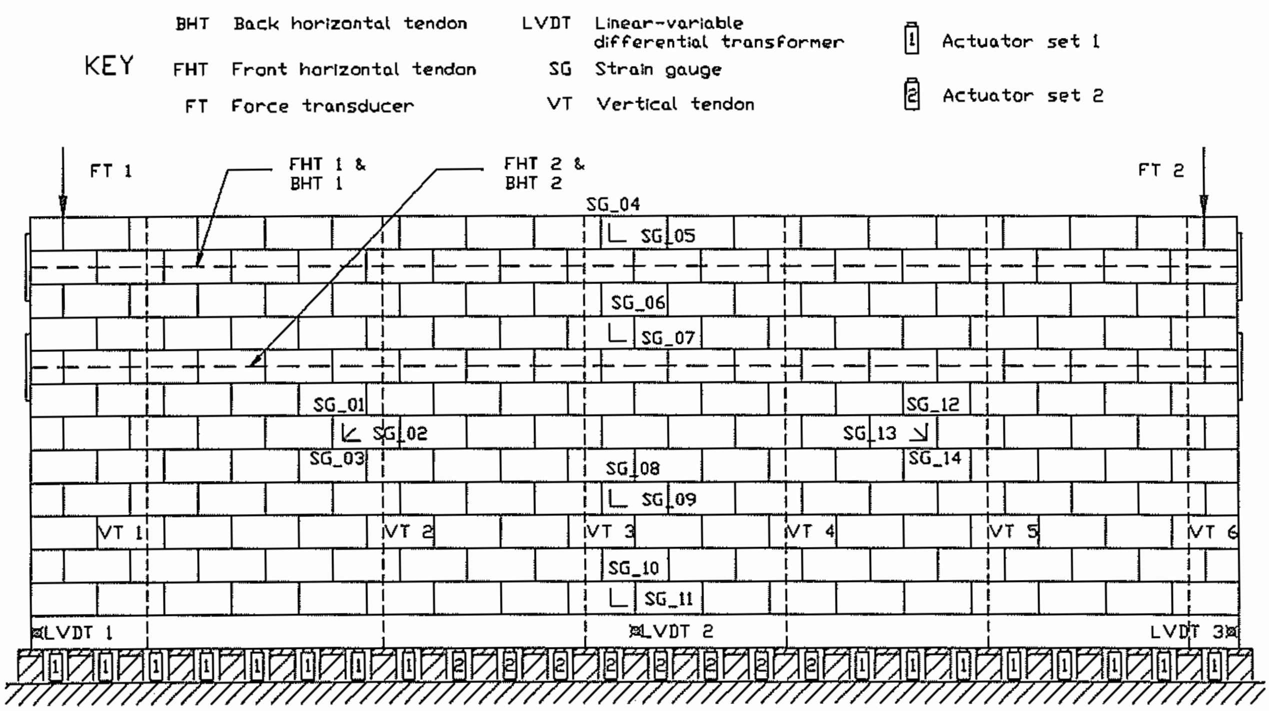

Figure 2. Masonry wall details.

The hydraulic loading system consisted of twenty-four 107-kN actuators and a hydraulic power unit with a complement of valves and hoses. Each actuator in a set shared a common hydraulic line so that uniform but different pressures could be maintained within each actuator set (fig. 2). As a result, all actuators applied an equal force within each set, but different forces could be maintained by each set. By varying the pressure within each actuator set, the magnitude of the maximum internal bending moment in the masonry wall was controlled.

The instrumentation used to monitor the hydraulic load and static frame systems and the deformations of the masonry wall were pressure transducers (PT), displacement transducers (LVDT), strain gauges (SG), and force transducers (FT). The force transducers were used to monitor the reaction forces in the two static frames that were used for two of the hogging tests, as will be discussed later in this paper[6]. The 120-ohm strain gauges were mounted on both sides of the wall and configured to form two opposite active arms of a 4-arm bridge. This doubles the output from strains due to in-plane bending and shear, and cancels out strains resulting from out-of-plane bending. A Campbell Scientific data logger and multiplexer were used as the data acquisition system.

Posttensioned Reinforcement

The masonry wall was posttensioned with vertical and horizontal tendons (fig. 2). Table 1 provides information for each corresponding test in which the reinforcement was used and the amount of tension in each tendon. (The hogging tests are designated by the acronym HG followed by the test sequence number.) The tension in the vertical tendons was measured by monitoring the pressure in the hydraulic actuator used to tension the tendons. The values obtained were not adjusted to compensate for the additional resistance due to friction in the actuator, nor for the loss in tension when the chuck jaws seat. A reusable chuck was used to hold the wire rope during and after the tensioning process. The tension in the horizontal tendons was determined using a proximity gauge and frequency counter as the horizontal tendons were torqued to the desired tension (Mangelsdorf 1983). The tension (H) in the wire rope was determined using the expression

$$H = 4\,\omega\,L^2 f^2, \tag{1}$$

where f = fundamental frequency of vibration, Hz,
L = wire rope length, m,
ω = wire rope mass per unit length, kg/m.

Time-dependent losses, due to creep, were not considered a factor because of the short time period between poststress and the in-plane bending tests.

The wire rope used for the vertical tendons was designated as 6 x 37 classification (Bridon American Corp.). The selection of this wire rope was based upon its flexibility and strength requirement. The diameter of the rope was 11.11 mm, and its nominal tensile strength was 91 kN. The specified mass per unit length of the wire rope was 0.48 kg/m.

The wire rope used for the horizontal tendons was 15.9-mm-diam Dyform®-18 HSLR rotation-resistant rope. The wire rope was selected because of its strength and resistance to rotation when tensioned. The nominal tensile strength and mass per unit length were 202 kN and 1.18 kg/m, respectively. The ends of the wire rope were swaged and fitted with standard sleeves (3.18-cm-diam, 14-cm-long, 10.8-cm thread length).

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[6]Reaction frames are not shown in figure 2, but their point of application is indicated by FT 1 and FT 2.

Bar stock (2.54-cm-wide) was welded to the ends of the swaged fittings so that the wire rope fitting could be held and torqued to the proper tension.

To install a vertical tendon, a portion of the face shell along the first course was removed. This allowed access to the top of the footing inside the masonry unit so that a 2.2-cm-diam, 20.3-cm-long hole could be drilled into the footing. The angle of the hole was approximately 14° from the vertical and the angle was governed by the geometry of the opening in the masonry unit. The hole was filled with a ceramic-filled epoxy, and the bird cage end of the wire rope was forced into the hole after it was snaked down through the cores of the masonry wall. A reusable strand chuck (11.11 mm) was used to grip and maintain the tension in the wire rope at the top masonry course as the rope was tensioned.

Hogging Test Program and Results

Five series of hogging tests were conducted on the masonry wall. The first hogging test (HG 1) performed was on the masonry wall with no reinforcement installed. HG 1 test conduct was as follows: The masonry wall was raised 2.5 cm off the wooden supports with all actuators operating from a common manifold (fig. 2). Actuator set 2 was locked in, and the pressure in actuator set 1 was slowly decreased at a rate of 1.4 kPa/s. (A micro-needle valve allowed such slow rates to be set). The masonry wall experienced a progressive staircase failure: tensile failure along its head joints and shear failure along its bedding joints early into the in-plane bending test (fig. 3). The moment-deflection curve is shown in figure 4. The moment resistance for the unreinforced masonry wall was calculated at its center, and the deflections of the masonry wall ends were measured with respect to the center of the masonry wall. The results of this test provided a baseline for evaluating the effectiveness of the proposed two reinforcement designs and also provided data for numerical modeling.

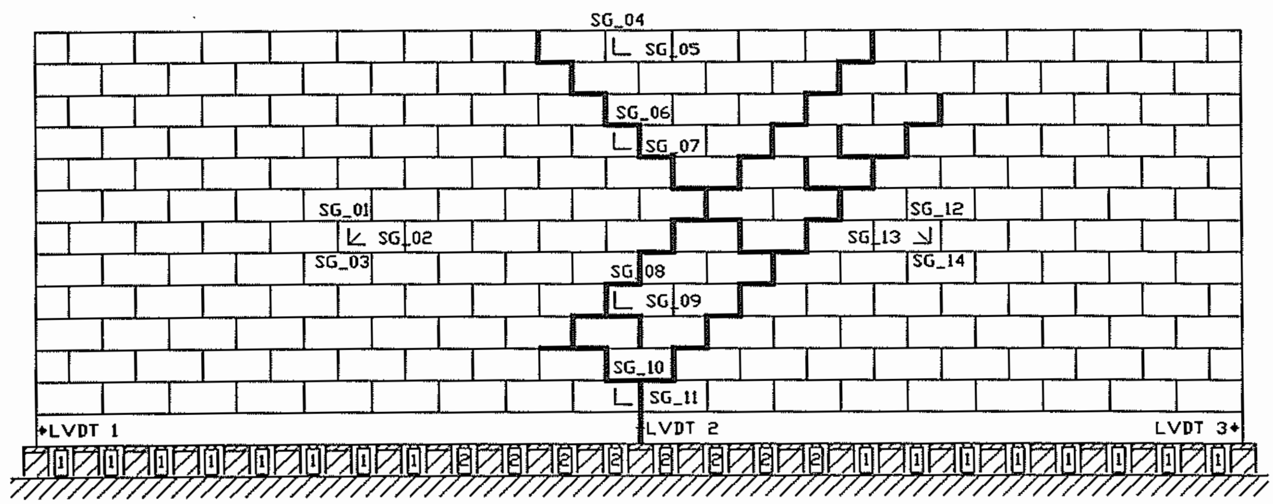

Figure 3. Failure of unreinforced masonry wall (HG 1).

Vertical tendons were then installed in the masonry wall[7] for the second test (HG 2, fig. 2) and posttensioned to the values provided in table 1. The purpose of the vertical tendons was to apply additional poststress to the wall in order to increase the frictional forces between the masonry units above those due to a superstructure load. It was believed that the additional frictional forces would increase the overall bending strength of the masonry wall. The posttension reinforcement also served as a means of repairing

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[7]The masonry wall was not repaired prior to this test or any of the additional hogging tests. This was felt to be the most conservative approach for evaluating the effectiveness of the posttension reinforcement.

the failed structure. The test conduct of HG 2 mirrored HG 1. The mode of failure for HG 2 was different than for HG 1 (fig. 5). Failure initiated in the top three courses, and the failure process stopped until the applied loads increased the bending tensile stresses to the level that the tensile strength of the masonry unit (masonry unit in fourth course from the top) was reached. Once this level was reached, the entire masonry wall failed. The vertical tendons actually caused failure to occur in another portion of the wall and prevented the HG 1 failure region from failing again. The vertically posttensioned wall was less stiff than the unreinforced masonry wall; this is attributed to slippage along the previously failed bed joints (figs. 3-4). The maximum moment capacity of the posttensioned wall was nearly the same as the moment capacity of the unreinforced wall, but the limit of deformation before failure doubled. It can be concluded that if horizontal tendons or steel straps had been present at the third course from the top of the wall to carry tensile stresses, the bending resistance and capacity of the masonry wall would have been significantly increased.

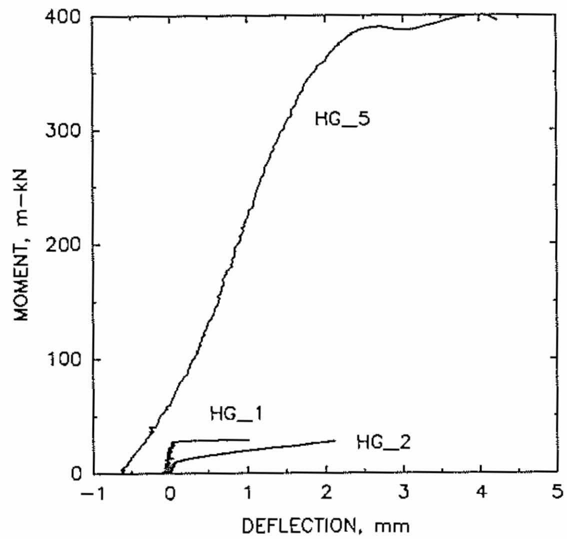

Figure 4. Moment-deflection curves for an unreinforced and posttensioned masonry wall.

For the third test (HG 3), horizontal tendons were installed (fig. 1) and tensioned to the values provided in table 1. The masonry wall was essentially converted into a truss system. The horizontal posttensioned region acts as the top cord and carries the tensile forces. The concrete footing and lower masonry courses are the lower cord and act as the compression member. The vertical members of the truss are the vertical tendons and carry tensile forces. The diagonal members are the masonry units and carry the compressional forces. The purpose of the horizontal tendons was to poststress the masonry wall and maintain it in compression across its entire cross section during the loading cycle. A hairline fracture was discovered after the horizontal tendons were tensioned at the anchorage location of VT 2. However, the hairline fracture did not affect

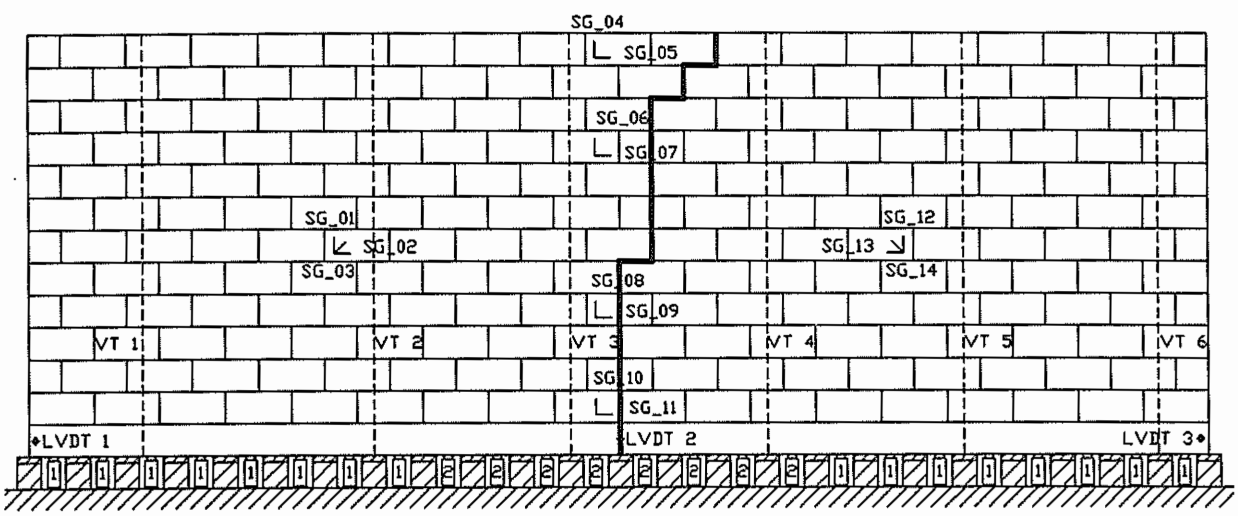

Figure 5. Failure of masonry wall reinforced with vertical tendons (HG 2).

the tensile force in VT 2. The test conduct of HG 3 was similar to that of HG 1. HG 3 was concluded when the masonry wall was able to cantilever (actuator set 1 pressure 0.0 MPa) and carry its own weight in bending. No failure in the wall was observed, including the previous failure regions HG 1 and HG 2.

Prior to conducting HG 4 the tension in each horizontal tendon was checked, and no time-dependent losses due to creep were observed. A static frame was installed at each end of the masonry wall. The reaction force locations are denoted by the locations of the force transducers (fig. 2). The hogging test proceeded as follows: The masonry wall was raised 2.5 cm, and then actuator set 2 was locked in. The pressure was slowly lowered in actuator set 1 to zero, and the wall was cantilevered. Actuator set 2 was then activated, and the pressure was slowly incremented. The test was terminated when the reaction force in each static reaction frame reached 116 kN. No failures or cracks in the masonry wall were observed. The reaction force of 116 kN represented the approximate load required for the bending moment tensile stresses to balance the poststress of the horizontal tendons.

HG 5 was conducted to determine the maximum load-carrying capacity of the posttensioned wall and to observe the failure mechanism. The test conduct of HG 5 mirrored that of HG 4, except that the test was not terminated until failure of the wall was observed. The wall failed in shear when the reaction load in the frames reached 120 kN. Despite the fact that the masonry wall failed, the wall still had a reserve of load-carrying capacity since the reaction frame forces dropped to only 49 kN. The maximum moment capacity of the vertically and horizontally posttensioned wall vastly exceeded the moment capacity of the unreinforced and vertically posttensioned masonry wall (fig. 4). The HG 5 curve is shifted because the horizontal posttensioning caused reverse flexure in the beam prior to the conduct of the test.

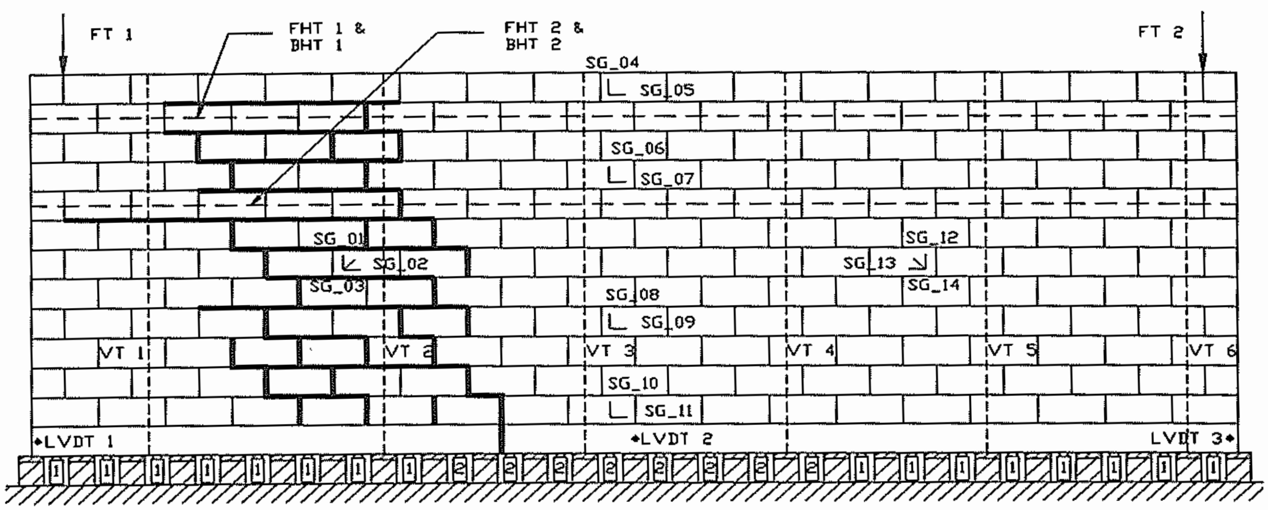

Figure 6. Failure of masonry wall reinforced with vertical and horizontal tendons (HG 5).

The sudden bond failure of vertical tendon VT 2 may have initiated the overall failure of the masonry wall since the applied vertical poststress in the area dropped to zero, and this is the area in which the wall failed in shear. It is believed that the wall would have achieved a greater load-carrying capacity had the anchorage of VT 2 not failed. The hairline fracture that was present in the footing (discussed earlier) must have propagated and subsequently compromised the integrity of the tendon anchorage.

The tension in all of the tendons was of interest, especially the vertical tendons, since their tension was determined from the setting actuator pressure. An access window was cut into the masonry, and the fundamental frequency of vibration of the vertical tendons was measured with a proximity gauge and frequency counter. The frequency and tensile values are provided in table 2. The results show that there is a discrepancy between the initial (table 1) and final readings for the vertical tendons (table 2). The major reasons for this discrepancy are (1) the error introduced by using actuator pressure to determine the posttension values and (2) the loss in tension when the chuck jaws seat. A third reason may be the failure of the masonry wall and crushing of the bed joints; however, this is doubtful since virtually no changes in tension occurred in the horizontal tendons. Another potential explanation for the discrepancy is slippage of the anchorage system. However, the wire rope pull tests showed that this explanation is not probable since, in all cases, the wire rope failed before the anchorage system failed, and anchorage slippage would have a dramatic effect on the slope of the wire rope load displacement curves (fig. 1).

Table 1. Tendon tension of HG 5.

Tendon	Frequency, Hz	Tension, kN
VT 1[1]	ND	53.4
VT 2[1]	ND	53.4
VT 3[1]	ND	53.4
VT 4[1]	ND	53.4
VT 5[1]	ND	53.4
VT 6[1]	ND	53.4
FHT 1[2]	29.9	53.4
BHT 1[2]	29.9	53.4
FHT 2[2]	26.1	42.7
BHT 2[2]	26.1	42.7

Table 2. Tendon tension--completion.

Tendon	Frequency Hz	Tension kN
VT 1	58.1	41.8
VT 2	0.0	0.0
VT 3	63.1	49.4
VT 4	62.4	48.0
VT 5	60.3	44.9
VT 6	52.6	34.3
FHT 1	28.6	54.7
BHT 1	29.2	54.7
FHT 2	25.3	42.7
BHT 2	25.2	42.7

Conclusions

This ongoing research project demonstrates the feasibility and positive benefits of posttensioning masonry walls. The combined use of vertical and horizontal tendons significantly increases the load-carrying capacity and bending strength of a masonry wall. The vertical tendons should also significantly increase the out-of-plane bending strength; this is currently being investigated in the laboratory. A method was developed to install vertical tendons in the interior of a masonry wall and to anchor the tendons into the footing. The installation of the tendons demonstrates the feasibility of posttensioning the vertical tendons within the foundation-floor interface and the availability of the equipment. Future work will concentrate on numerical modeling and evaluating postreinforcement designs for masonry walls when subjected to sagging (in-plane bending) and racking (out-of-plane bending).

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