THE EARTHQUAKE PERFORMANCE OF PARTIALLY REINFORCED MASONRY PIERS SUBJECTED TO IN-PLANE CYCLIC LOADING

George C. Manos, M. Yasin, J. Thawabteh, V. Kourtides

ABSTRACT

The earthquake performance of partially reinforced masonry piers when subjected to seismic loading is examined here in the framework of an extensive experimental investigation. The objective is to be able to construct earthquake resistant low-rise partially reinforced masonry buildings in areas of moderate seismicity of Greece. This paper includes an overview of the earthquake performance of partially reinforced piers employing a “Greek” type brick, which is developed and manufactured by Filippou Structural Clay Products and is now available from their industrial production facility. Almost all the brick units employed in the framework of the current research program were produced by this industrial process; similarly the rest of the materials employed in the construction of the examined masonry piers are also available in the current construction practice in Greece. This research has been conducted at the Department of Civil Engineering, Aristotle University and was under the financial support of the Greek Ministry of Energy and Industry, General Secretariat of Research and Technology.

Key words: Partially reinforced masonry, Earthquake performance, In-plane cyclic loading

Laboratory of Strength of Materials and Laboratory of Reinforced Concrete, Department of Civil Engineering Aristotle University, Thessaloniki 54006, Greece.
Fax: 003031 995769, email: gcmanos@civil.auth.gr.

INTRODUCTION

Extensive experimental programs have been performed, both in the USA as well as in various European Institutions, in order to examine the performance of structural elements of this type when subjected to various types of loading. Most of these tests employed masonry piers with various reinforcing arrangements, which were subjected to combined constant compression together with horizontal load reversals of varying amplitude (cyclic loading) in order to investigate the influence of certain parameters on the response. The most significant parameters, that are usually examined, are the type and strength of the materials (mortar and masonry units), the geometry of the masonry piers and their reinforcing arrangement (in quantity and structural details), and the level of axial compression (Tasios 1987). The behavior of masonry piers for seismic loads has been studied in the past by subjecting these piers simultaneously to combined horizontal and vertical loads (racking tests, Hidalgo et.al. 1978). The influence of various reinforcing arrangements on the pier's behavior was also studied under these loading conditions (Hidalgo et.al., 1978, Tomazevic et.al., 1993). In order to achieve a reasonable simulation of the actual earthquake loading conditions, the horizontal loads are usually applied in a cyclic manner, in order to represent the alternating nature of the seismic loads; the gravity action, simulated by the vertical concentrated load, is assumed to remain almost constant, an assumption that can be considered reasonably valid for low-rise buildings that represent the majority for this type of construction in seismically active regions. The basic structural components of this type of masonry building are connected together with diaphragms (with a varying degree of flexibility) so that each one of the basic masonry components is subjected to simultaneously in-plane and out-of-plane horizontal seismic actions on top of the gravity loads. Despite this, much experimental research has focused on the in-plane behavior, separately from the out-of-plane behavior. As was shown by Manos and co-workers (Manos 1983, Gulkan 1990), this can be a reasonable assumption under certain conditions. The dominant role in the earthquake performance of masonry buildings is played by the satisfactory in-plane performance of masonry piers that are distributed in such a way as to form the "shell" of a masonry building and to provide its earthquake resistance by in-plane actions in both horizontal directions. As a consequence, the importance of investigating the in-plane behavior of masonry piers is paramount. The current research focuses on the in-plane behavior of masonry piers when they are subjected to in-plane loads simulating the combined earthquake and gravitational actions. The final objective is to be able to employ such structural elements in order to construct earthquake resistant low-rise partially reinforced masonry buildings in areas of moderate seismicity of Greece.

TESTING ARRANGEMENT

Figure 1 depicts the testing layout whereby the masonry pier specimen is placed within a steel reaction frame with its foundation being anchored to that frame. This reaction frame, which is part of the Earthquake Simulator Facility of Aristotle University, also provides the support for the horizontal servo-hydraulic actuator, which has a capacity of 250KNt, a stroke of $\pm$ 50mm and a capability of displacement control with a good fidelity in its response in the frequency range from 0 to 50Hz. Moreover, the same reaction frame also provides support for the vertical hydraulic jack that has a capacity of 200KNt and a stroke of ± 200mm, with the force being applied statically. Because this vertical jack is not displacement-controlled, a system of accumulators was added, in order to avoid variations of the vertical force when the specimen develops excessive deformations that also include significant vertical displacements at the post-cracking stage. Thus, whereas the vertical load is kept almost constant at a predetermined level, the horizontal force applied at the top is varied in a cyclic manner; this results from controlling the imposed horizontal displacement at this point of the masonry pier in a predetermined way. The imposed cyclic displacement time history is depicted in figure 2, in terms of displacement amplitude versus number of cycles. The frequency of this cyclic loading is also one of the studied parameters and it can be specified at the beginning of the test for each pier. The prescribed horizontal displacement depicted in figure 2 has been already applied with two distinctly different frequencies; the first being a rather slow variation of the horizontal forces at 0.01Hz whereas the second is a rather fast variation of the horizontal forces at 1.00Hz. The former simulates the prototype earthquake forces only in the reversible (cyclic) nature of the loading, whereas the latter simulates both the cyclic nature as well as the frequency content of the seismic forces. It is believed that the employed frequency content is quite representative of the dominant frequency content that is expected to develop in the earthquake response of such masonry components as parts of a low-rise masonry building. A large number of masonry piers, with a height of 1330mm, were tested in the reaction frame of figure 1. Selected results are included in paragraph 5.1. A limited number of relatively large piers, with a height of 2475mm, were also tested, utilizing the strong reaction frame of the Laboratory of Reinforced Concrete Structures of Aristotle University, which includes a 250KNt vertical actuator and a 1000KNt horizontal actuator in an arrangement similar to that of figure 1. Selected results for these large piers are included in paragraph 5.2. and 5.3. The imposed horizontal displacement for the large piers is similar to that depicted in figure 2, but with twice the amplitude reaching a maximum displacement equal to 40mm at the top of the pier. The variation in time of the imposed displacement for this later testing arrangement was kept always at a low speed.

Masonry Specimens

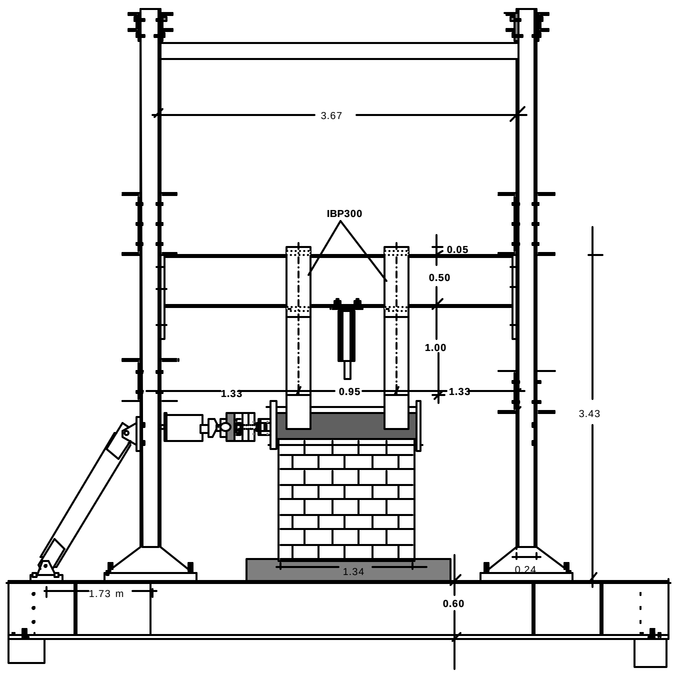


Figure 1. Masonry pier being subjected to the racking test in the strong steel reaction frame.

All masonry specimens were constructed with the special "Greek" brick with vertical holes as depicted in figure 3. This masonry unit was initially developed as a pilot brick unit in the framework of a Brite-Euram project by the industrialists Filippou Structural Clay Products in cooperation with the Technical University of Athens under the leadership of Professor T. Tasios. Moreover, during this Brite-Euram research effort, the mechanical properties of the masonry unit in itself were also examined, as well as those of masonry piers built with it similar to the ones of the current project and under similar loading conditions (Psylla et.al., 1996). The whole Brite-Euram research effort was related to the use of reinforced masonry for building in all seismic zones (Modena et.al. 1996). The geometry of this pilot brick unit underwent certain modifications in the current research effort with regard to its height as well as the dimensions of its vertical holes. Moreover, the reinforcing arrangements that are studied here during the current project are also different. What is investigated now is the applicability of this type of construction for low-rise housing (1 or 2-story buildings) in moderate hazard seismic zones of Greece (zones 1 and 2 of the 1992 Greek Seismic Code).
The employed variation of significant parameters is presented in brief below:

1. The piers were loaded in the vertical directions with forces that represented ***different levels of compression stresses normal to the horizontal bed joints***.

2. The tests that have been performed so far, utilizing the testing arrangements described in paragraph 1, included partially reinforced piers of the following ***proportions***:
- Length 1330mm , height 1330mm and thickness 154mm.
- Length 660mm , height 1330mm and thickness 154mm.

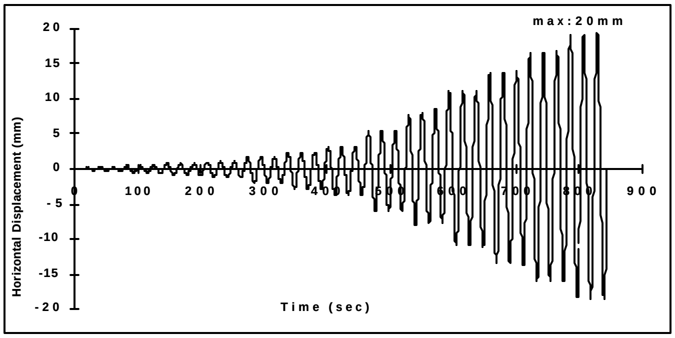


Figure 2. Applied horizontal cyclic displacements.

Apart from the above geometry, which must be considered as ½ scaled specimens and represent the bulk of the testing sequence, a limited number of specimens near to prototype scale were also tested. These specimens are of the following geometry:

- Length 2700mm , height 2475mm and thickness 320mm.
- Length 2700mm, height 2475mm and thickness 154mm.

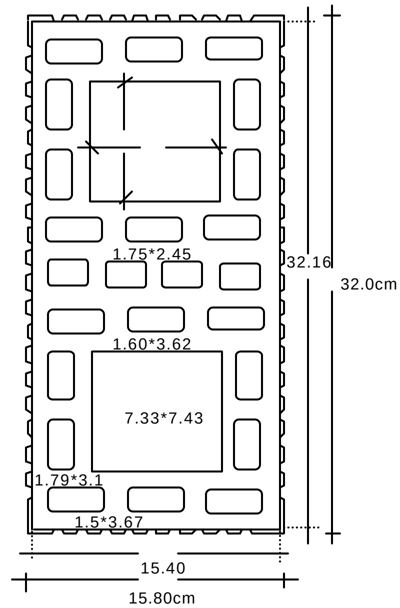


Figure 3. “Greek” brick unit
D Cross-section

2. These later specimens, which were 2475mm high, were tested as described in paragraph 1. The pseudo-dynamic variation of the horizontal forces at 0.01Hz was only applied in this case of the masonry piers of 2475mm height whereas the piers of 1330mm height were tested both with the ***pseudo-dynamic as well as with the fully dynamic*** variation of the horizontal forces. All these test specimens were constructed with the “Greek” brick masonry units with vertical holes; these were initially developed in the framework of a Brite - Euram project by Filippou Structural Clay Products industry. The units, shown in figure 3, were modified in terms of their basic dimensions and composition of their ceramic material in the framework of the current project; moreover they are currently produced by Filippou industry under full production which implied certain modifications in the furnace and dryer conditions.

3. ***The compressive strength of the mortar*** employed in the construction of these piers was aimed at having the following strengths:
- Category O, target compressive strength 2.5Mpa
- Category N, target compressive strength 5.0Mpa.

4. Finally, various ***longitudinal and transverse reinforcing arrangements*** were examined together with a variation of the level of uniform compression.
Forty partially reinforced masonry piers were constructed and tested in order to meet the variation of the parameters listed above. The majority of the masonry piers were 1330mm high. These were

tested at the special steel reaction frame that is part of the earthquake simulator facility of the Laboratory of Strength of Materials, of Aristotle University. A limited number of masonry piers were 2475mm high, almost the height of a single floor masonry pier in prototype construction conditions; these were tested at the steel reaction frame of the Laboratory of Reinforced Concrete, of Aristotle University.

Standard Tests

A number of standard tests were carried out prior to the complex racking tests. The following tables 1 and 2 include the final summary results from these tests. These are:

- Simple compression tests of the masonry units.
- Simple compression tests of cubes taken from mortar during the construction of the racking specimens.

- Simple compression tests of cubes taken from the grout during the construction of the racking specimens.

- Simple tension tests of the reinforcement used in the construction of the racking specimens.

- Simple pull-out tests of the reinforcement used in the construction of the racking specimens from cubes of grout.
- Simple compression tests of masonry piers, without reinforcement. These piers had 660mm length, 1330mm height and 154mm thickness; they were constructed at the same time and with the same materials as those used in the construction of a group of racking specimens.

Table 1 Compressive Strength of Brick Units and of Mortar and Grout

Brick Unit	Strength (Gross) fb (Mpa)	Strength (Net) fb (Mpa)	% of Voids
Â	2.59	5.20	50%
C	8.93	17.38	49%
D	7.14	15.48	54%
Wall Name		Mortar Strength fm (Mpa)	Grout Strength fck (Mpa)
Wall-5(Í)		3.48	7.39
Wall-6(Í)		4.31	7.24
Wall-7(Í)		3.50	7.67
Wall-8(Í)		3.48	7.56
Wall-9(Í)		3.44	7.73
Wall-11(Í)		3.46	7.50
Wall-12(Í)		4.56	7.54
Wall-13(Í)		4.53	7.73
Wall-14(Í)		5.08	7.81
Wall-15(Í)		4.64	7.62
Wall-19(Í)		6.21	8.47
Wall-20(Í)		5.99	8.84
Wall-21(Í)		3.70	8.43
Wall-22(Í)		3.68	8.92
Wall-23(Í)		3.89	8.81
Wall-27(Í)		4.28	9.78
Wall-34(N)		4.38	11.47
Wall-29(Ï)		2.00	8.81
Wall-30(0)		2.16	9.71
Wall-31(Ï)		1.88	10.69
Wall-32(Ï)		2.32	8.69
Wall-33(Ï)		2.08	8.91

Table 2 Compression and Shear (from diagonal tension) Strengths

Specimen Name	Brick Category	Dimensions of Pier (Ç/Â) (cm)	Mortar Strength **fm** (Mpa)	Type of Test	Measured Strength (Mpa)	Predicted ***** Strength (Mpa)
Pier 1 (Í)	B	131 / 65.5 /15.5	4.03	Compression	fk = 3.85	fk = 2.07
Pier 1 (Ï)	Â	104.5 / 65.5 /15.5	2.09	Compression	fk = 2.51	fk = 1.75
Pier 2 (Í)	C	130 / 66.3 /15.5	4.03	Compression	fk = 4.56	fk = 4.53
Pier 3 (Í)	C	130 / 66.1 /15.5	4.03	Compression	fk = 4.53	fk = 4.53
Pier 4 (Í)	Â	133 / 65.6 /15.5	4.03	Compression	fk = 3.79	fk = 2.07
Pier 5 (Í)	C	129.5 / 65.5 /15.5	4.03	Compression	fk = 5.10	fk = 4.53
Pier 2 (Ï)	C	130 / 66.3 /15.5	2.09	Compression	fk = 3.48	fk = 3.85
Pier 3 (Ï)	C	129.5 / 66.3 /15.5	2.09	Compression	fk = 3.37	fk = 3.85
Pier 1D(N)	Â	106 /100 / 15.5	4.03	Diagonal Tension	fvk=0.169	fvko=0.15
Pier 1D(Ï)	Â	106 / 99.5 / 15.5	2.09	Diagonal Tension	fvk=0.165	fvko=0.1
Pier 2D(Ï)	D	66 / 66 / 15.5	2.09	Diagonal Tension	fvk=0.204	fvko=0.1
Pier 2D(Í)	D	97.5 / 100 / 15.5	4.03	Diagonal Tension	fvk=0.240	fvko=0.15
Pier 3D(Ï)	D	97.5 / 100 / 15.5	2.09	Diagonal Tension	fvk=0.199	fvko=0.1

* Euro-Code 6 (K=0.50, fk = Ê $fb^{0.65} fm^{0.25}$, fvko= limit shear strength for zero normal stress)

- Simple diagonal tension tests of masonry specimens without reinforcement. These specimens had length 99.5mm, height 106mm and thickness 106mm; they were constructed at the same time and with the same materials as those used in the construction of a group of racking specimens.

The variation of the applied load together with certain important deformation levels were recorded during some of these standard masonry tests through the appropriate instrumentation.

Cyclic Tests

The measured response has been treated in such a way as to deduce the most significant state of stress. In general the following dominant response modes are expected to develop in these masonry specimens:

- A predominantly rocking mode at the foundation level that is due to the development of a limit flexural state of stress at the foundation level under the combined action of the horizontal and vertical loads.
- A predominantly flexural mode for the rest of the masonry pier, apart from the foundation level due to the flexural state of stress for the rest of the masonry pier under the combined action of the horizontal and vertical loads.
- A predominantly sliding mode at the foundation level that is due to the development of a limit sliding state of stress at the foundation level under the combined action of the horizontal and vertical loads.

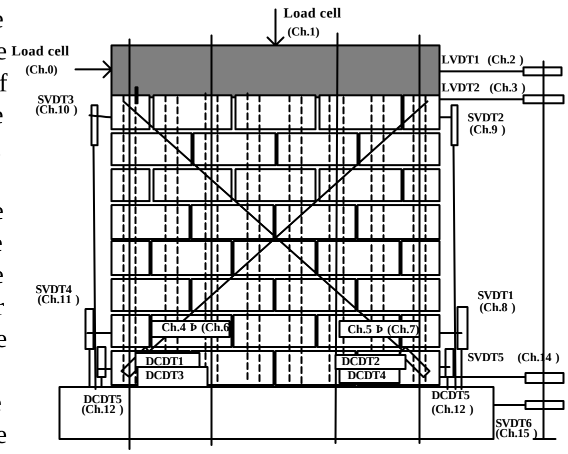


Figure 7. Used Instrumentation scheme

- A predominantly shearing mode for the rest of the masonry pier, apart from the foundation level, that is due to the development of a shearing state of stress for the rest of the masonry pier under the combined action of the horizontal and vertical loads.

The variations of the basic parameters studied in the present test sequence were expected to exert an influence on the development of the above dominant modes of response. The instrumentation scheme was aimed at being able to identify the contribution of each one of the above response modes to the total response of the masonry piers during the racking tests. The basic parameters, whose influence was studied in the framework of the current project, are listed below:

- The geometry in terms of height over length ratio; one represents rather slender piers with height over length ratio approximately equal to 2 whereas the second addresses less slender piers with height over length ratio approximately equal to 1. This must be viewed together with the two different types of thickness, as described in paragraph 2.

- The type of mortar; two distinct types, O and N are examined.
- The level of axial compression that is applied to these piers together with the horizontal loads.

This level of compression is set as a percentage of the compressive strength of the masonry specimens subjected to the standard simple compression test presented in paragraph 3. As already mentioned, it was intended here to investigate the performance of this type of masonry piers as part of low-rise housing; it is expected that the level of axial compression of vertical structural elements composing such low-rise housing is rather low. Consequently, two levels of axial compression were adopted; the first is 4% of the masonry strength and the second is 8% of the masonry strength.

Moreover, a limited number of specimens were tested with no axial compression.

- The influence of the amount of horizontal (transverse) reinforcement was studied in terms of ratio of the area of this type of reinforcement of the corresponding gross cross sectional area. This ratio was varied from a relatively low value, approximately equal to 0.05%, to a somewhat larger value, approximately equal to 0.150%. The amount of the vertical (longitudinal) reinforcement, in terms of ratio of the area of this type of reinforcement of the corresponding gross cross sectional area, remained constant for all the specimens, approximately equal to 0.125%.

- In addition to the above parameters the influence of the frequency of the cyclic horizontal loading, as mentioned in paragraph 1, is also one of the studied parameters in the reduced scale specimens.

DISCUSSION OF TEST RESULTS

<u>Masonry Piers with 1330mm height and a nominal thickness of 155mm (Wall-17N)</u>

Some of the measured response during this test sequence, together with the observed damage patterns, are presented and discussed in what follows. Figure 8 depicts the details of a specimen with dimensions: 1330mm length by 1330mm height and thickness 155mm. The mortar used was of type N and the axial compression level was 4% of the strength of this type of masonry in compression. The vertical and horizontal reinforcing arrangement is also shown in figure 8 whereas figure 9 portrays the damage sustained by this specimen at the end of its test.

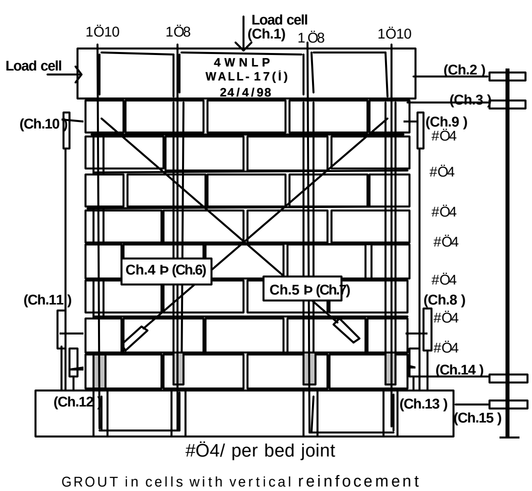


Figure 8. Reinforcing Details of Wall-17N

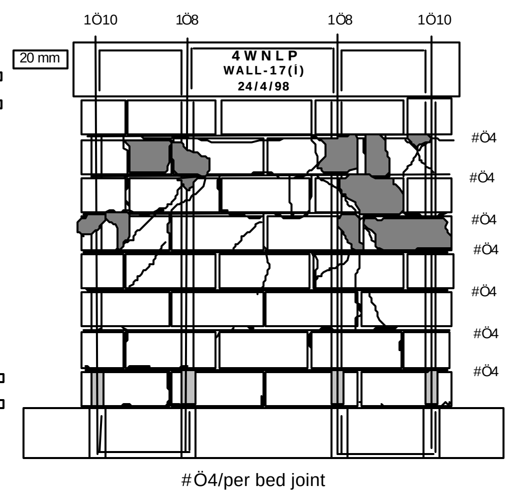


Figure 9. Observed Damage of Wall-17N

The following observations can be made on the basis of the observed behavior for this partially reinforced pier (Wall-17N).

- The tested pier exhibits an increasing horizontal load capacity up to the 9th group of cyclic loading, which corresponds to a maximum displacement of 7.5mm.
-

- For this level of deformation, the degradation observed for the subsequer cycles, which are of the same displac level as the first cycle of this group, is limited.
- An increasing horizontal load capacity can also be seen even for the 10th group of cyclic loading, which corresponds to a maximum displacement of 10mm. However, for this level of deformation the load degradation observed for the subsequent two cycles becomes noticeable.

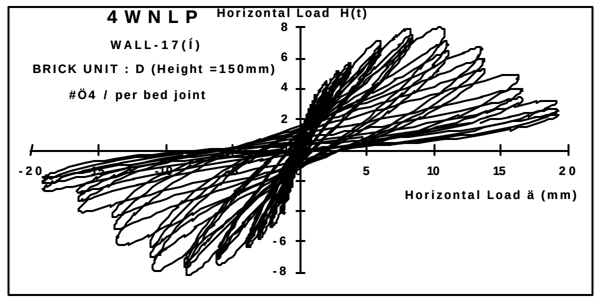


Figure 10. Load-Deformation Cyclic Behavior of Wall-17N

- For the remaining three groups of cyclic loading, from 12.5mm to 20mm, the pier cannot sustain the maximum load that was measured before (just above 8t), and its capacity deteriorates in each subsequent cycle.

The above observations are in agreement with the observed damage. This is initially flexural but, as the imposed level of deformation is increased, the shear type of damage prevails, accompanied with the disintegration of the central and upper part of the pier during the last group of cyclic loading.

Masonry piers of 2475mm height and a nominal thickness of 155mm (Wall 27N)

This pier was tested at the strong reaction frame of the laboratory of Reinforced Concrete Structures of the Department of Civil Engineering of Aristotle University of Thessaloniki, utilising a horizontal actuator with maximum capacity of 1000KNt and a vertical actuator of 250KNt capacity. Figure 11 depicts the details of one of these large specimens with dimensions: 2700mm length by 2475mm height and a thickness 155mm. The mortar used for this specimen (Wall 27N) was of type N and the axial compression level was 4% of the strength of this type of masonry in compression, as it was for specimen 17N. The vertical and horizontal reinforcing arrangement is also shown in figure 11, which, as can be seen, is similar to the one employed in specimen 17N. Figure 12 portrays the damage sustained by this specimen at the end of this test. Finally, figure 13 depicts the horizontal load and horizontal displacement variation at the top of this pier. The following observations can be made on the basis of the observed behavior for this partially reinforced pier (Wall 27N).

- The tested pier exhibits an increasing horizontal load capacity up to the 8th group of cyclic loading, which corresponds to a maximum displacement of approximately 15mm.

- For this level of deformation, the load degradation observed for the subsequent two cycles, which are of the same displacement level as the first cycle of this group, is noticeable.

-For the subsequent two groups of cyclic loading, which corresponds to maximum displacements from 10mm to 25mm, there is a moderate reduction in the horizontal load capacity. However, for this level of deformation, the load degradation observed for the subsequent two cycles does not accelerate.

- For the remaining three groups of cyclic loading, from 30mm to 40mm, the horizontal load sustained by this pier rapidly decreases to a small part of the maximum horizontal load capacity (equal to approximately 15t at the 8^{th} group of cyclic loading). This capacity deteriorates in each subsequent cycle.

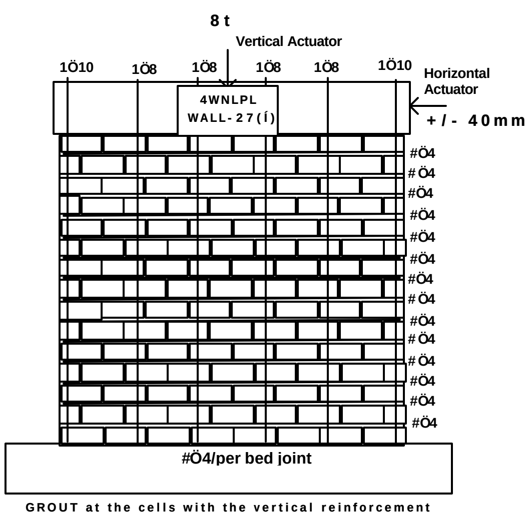


Figure 11. Reinforcing Details of Wall-27N

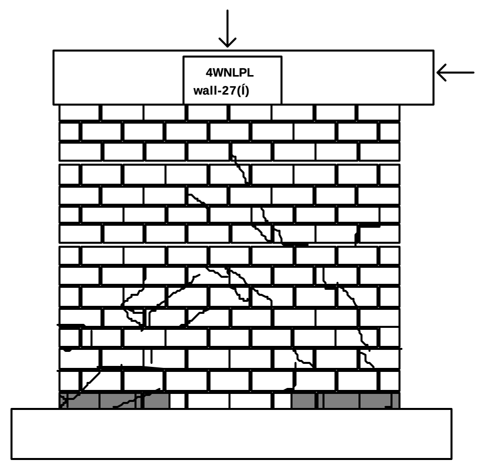


Fig. 12. Detail of Damage at the bottom part of masonry Wall-27(N).

- It is interesting to note that the horizontal cyclic load capacity of this wall (Wall-27N) is almost twice as much as the wall 17N, which was presented in paragraph 5.1.

As already pointed out, the length and height of wall 17N are approximately ½ of the corresponding dimensions of wall 27N. The above observations for wall 27N are in agreement with the observed damage. This is initially flexural, as was also observed for wall 17N, but as the imposed level of deformation is increased the shear type of damage prevails. However, this time no disintegration of any part of the pier takes place.

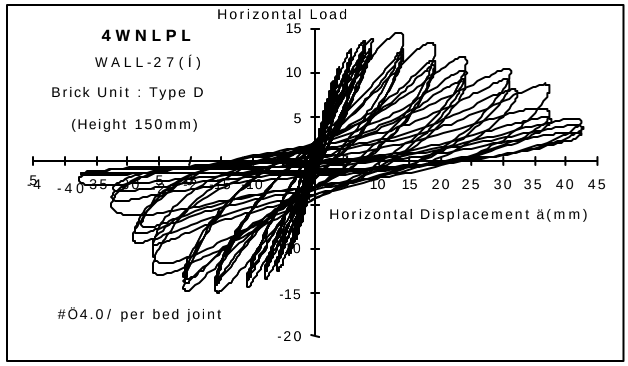


Figure 13. Load-Deformation Cyclic Behavior of Wall-27N.

Masonry piers of 2475mm height and a nominal thickness of 320mm (Wall-34N)

This pier, constructed with double brick thickness, was also tested with the same loading arrangement described before for Wall 27N. The reinforcing and grouting arrangements are depicted in figure 14. The mortar used was also of type N and the axial compression level was 4% of the strength of this type of masonry in compression, as it was for specimens 17N and 27N. Figure 14 portrays the end-damage sustained by this specimen and figure 15 depicts the horizontal load and horizontal displacement variation at the top of this pier. The following observations can be made on the basis of the observed behavior for Wall-34N.

- The tested pier exhibits an increasing horizontal load capacity up to the 8th group of cyclic loading, which corresponds to a maximum displacement of approximately 15mm.

- For this level of deformation, the load degradation observed for the subsequent two cycles,

which are of the same displacement level as the first cycle of this group, is noticeable.

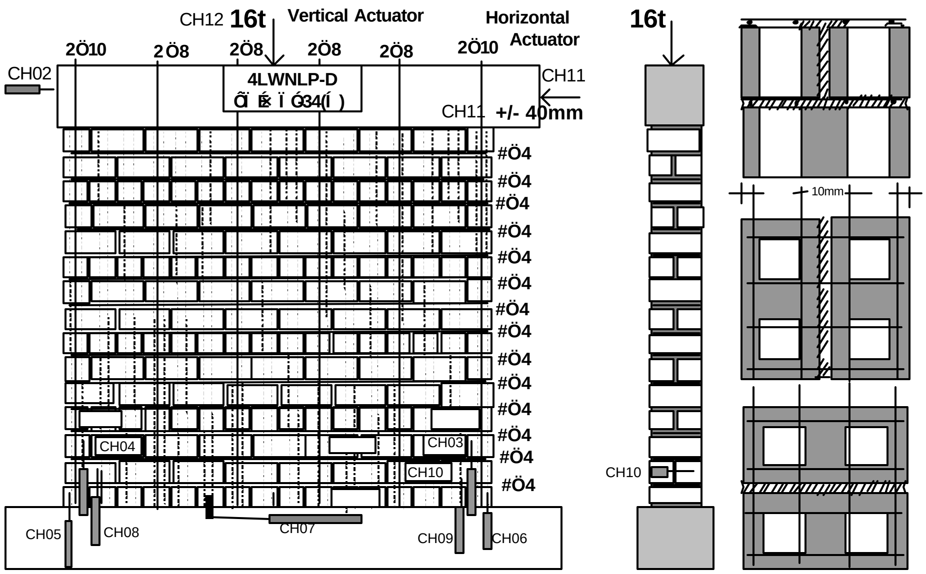
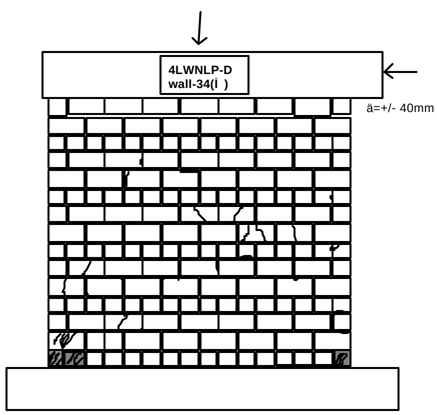


Figure 14. Reinforcing Details and Instrumentation of Wall-34N



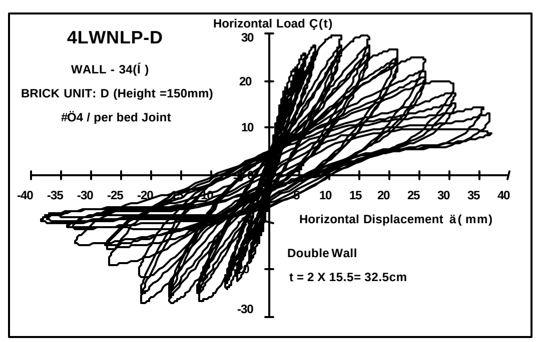


Fig 15. Detail of Damage of Wall-34(N). Fig. 16. Load-Deformation cyclic behavior of masonry all-34(N).

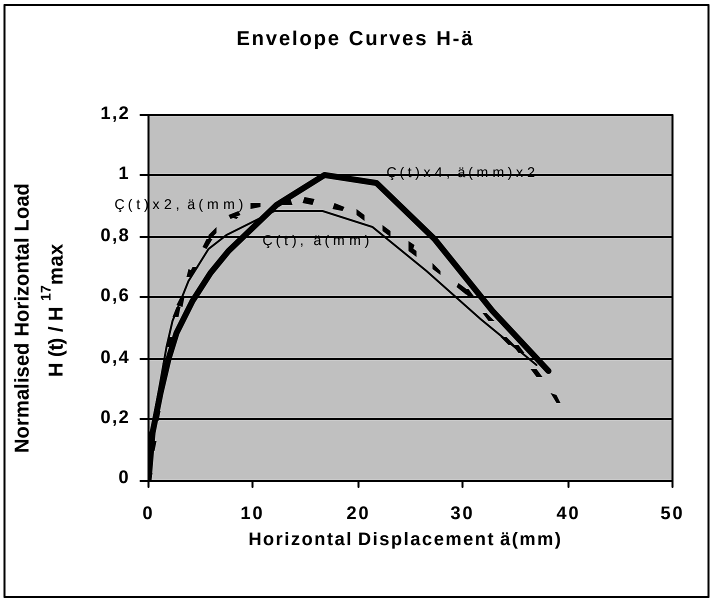
- For the subsequent two groups of cycling loading , which corresponds to maximum displacements from 10mm to 25mm, there is a moderate reduction in the horizontal load capacity. However, for this level of deformation, the load degradation observed for the subsequent two cycles does not accelerate.

- For the remaining three groups of cyclic loading, from 30mm to 40mm, the horizontal load sustained by this pier rapidly decreases to a small part of the maximum horizontal load capacity (of the order of approximately 30t at the 8th group of cyclic loading). This observed maximum load capacity deteriorates in each subsequent cycle.

- The above observations for wall 34N are in agreement with the observed damage. This is initially flexural, as was also observed for walls 27N and 17N, but, as the imposed level of deformation is increased, the shear type of damage prevails. However, this time no disintegration of any part of the pier takes place.

- As can be seen, almost all the observations made above for the cyclic behavior of Wall-34 are very similar to the corresponding behavior of Wall-27 and Wall-17.

- It is interesting to note that the horizontal cyclic load capacity of this wall (Wall-34N) is almost twice as much as that of Wall-27N, which in its turn, as already observed, is twice as much as that of the Wall-17N. Wall-17N, according to its dimensions, can be considered as a ½ scaled model of Wall 34N. And the scaling factor for the horizontal load is equal to 4 whereas the length, width and thickness scaling factor is equal to 2. The comparison of the observed behavior of these three tested walls (Wall-17N, Wall-27N and Wall-34N) in terms of envelope curves is depicted in figure 17. The observed cyclic behavior of all these walls is made comparable to the reference level of Wall-34N, using the appropriate scaling factors. As can be seen from this comparison, despite certain differences, the cyclic behavior of all three walls is very similar in the most significant aspects. The comparative larger strength of Wall-17N than the corresponding strength of Walls -27N and Wall-34N must be attributed to certain differences in the vertical reinforcing arrangement resulting in this increased (flexural) strength. This important finding establishes a degree of confidence in extrapolating the observed behavior of all the tested walls of reduced height and thickness (1330mm height and 155mm thickness) to prototype conditions (2475mm height and 320mm thickness).



- - - - 4WÍLPL Wall-27(Í)

#Ö4/bed joint (14 bed joints), Brick Unit Height =15cm, Mortar Í, **ñ**$_v$ =0.0086%Axial Load 8t, Grout in 6 cells

—— 4WNLPLD Wall-34(N)

#Ö4/bed joint (14 bed joints) , Brick Unit Height =15cm, Mortar N, **ñ**$_v$ =0.0082% Axial Load 16t, Grout in 12 cells

—— 4WNLP Wall-17(N)

#Ö4/bed joint (7 bed joints), Brick Unit Height =15cm, Mortar N, **ñ**$_v$ =0.0125%, Axial Load 4t, Grout in 4 cells

Fig. 17. Comparison of Observed Cyclic response for Walls

CONCLUDING REMARKS

- The outline of an extensive experimental sequence has been presented, whose aim is to examine the seismic performance of partially reinforced brick masonry piers constructed by a newly developed hollow “Greek” brick unit with vertical holes. This investigation focuses on low-rise housing to be constructed in low to moderate seismic zones of Greece.

- The basic parameters whose influence on the piers’ performance was studied are also outlined together with the instrumentation scheme that was employed in identifying the dominant modes of response.

- Finally, typical results related to two tested masonry piers, the first with 1330mm height and the second with 2475mm height, are presented and discussed. It can be seen that both these piers had reached their ultimate state by the used loading system. Moreover, the employed instrumentation recorded this performance in a way that adequately describes the observed behavior. The performance of both piers is well controlled up to drift values of 0.6%. For deformation levels larger than this value the tested piers cannot sustain the applied horizontal load and at the same time they develop damage that is rather of the shear type, with partial disintegration of the wall integrity for deformation levels reaching drift values of the order 1.6% .

ACKNOWLEDGEMENTS

The partial support of the General Secreteriat of Research and Technology of the Ministry of Development of Greece is acknowledged. The encouragement of Professor George Penelis is also gratefully acknowledged with thanks.

REFERENCES

- P. A. Hidalgo, R.L. Mayes, H. D. McNiven and R.W. Clough. (1978), “Cyclic Loading Tests of Masonry Single Piers” Volumes1,2,3, EERCE Reports No. 78/27, 78/28, 79/12, University of California at Berkeley.
- Gulkan P., Clough R.W., Manos G.C. and Mayes R.L. (1990) “Seismic Testing of Single-story Masonry Houses : Parts 1 and 2”, Journal of Str. Eng. ASCE, Vol. 116, No 1, January 1990, pp. 235-274.
- G.C. Manos, R.W. Clough, R.L.Mayes “Shaking Table Study of Dingle-Story Masonry Houses”, Report No. 83/11, EERC, University of California, Berkeley, 1983, U.S.A.
- C. Modena, A. Bernardini et.al. (1996), “Reinforced Masonry for Buildings in Seismic Zone”, Report of University of Padova in the framework of Brite Euram Project No 4001.
- N. Psilla, E. Vintzileou, T. P. Tasios (1996), “Seismic Behavior of Reinforced Masonry”, Proceedings of Greek Conference on Reinforced Concrete Structures, Vol. II, pp. 284-294, Cyprus, (in Greek).
- T. P. Tasios (1987), “The Mechanics of Masonry”, Athens, (in Greek)
- M. Tomazevic, M. Lutman, L. Petkovic (1993), “In-Plane Behavior of Reinforced Masonry Walls Subjected to Cyclic Lateral Loads”, Institute for Testing and Research in Materials and Structures, Ljubljana, Slovenia.