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DISPLACEMENT CAPACITY OF CONFINED MASONRY STRUCTURES REINFORCED WITH HORIZONTAL REINFORCEMENT: SHAKING TABLE TESTS

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Abstract

Two half-scale three-story specimens with a single axis of a typical earthquake resistant structural system for housing with different amounts of horizontal reinforcement were constructed. Scaled masonry units were obtained by cutting traditional solid clay bricks. Specimens were tested in a shaking table and were subjected to strong ground motions representative of Mexico City's seismic environment. Instrumentation was designed to measure detailed responses of story accelerations, displacements and strains in tie-columns, tie-beams and horizontal reinforcement steel bars. Displacement capacity of walls was compared to estimated capacity obtained from preliminary pseudo-static tests of half-scale masonry walls. Based on test results, estimates of appropriate drift limits are recommended; drift and ductility demands calculated from roof displacements and base shear envelopes were used to estimate the appropriate seismic shear force reduction factor. Measured shear strength was compared with that calculated from code provisions.

Keywords: Shear strength; confined masonry; horizontal reinforcement; shaking table tests

1. Introduction

Previous shaking table tests of confined masonry structures with no horizontal reinforcement have demonstrated that displacement capacity of confined masonry walls can be as large as two times that measured in pseudo-static tests [1, 2]. Data obtained in those tests was very useful for the determination of code drift limits for this construction system. However, no equivalent information was available for this type of structures with horizontal reinforcement included in the mortar joints.

Shaking table testing of two half-scale three-story confined masonry specimens is described in this paper. Specimens were made of solid clay bricks. First specimen tested had no, horizontal reinforcement; second specimen was reinforced with a conventional amount of reinforcing steel wires placed within the mortar joints..

The objectives of the study were: 1) to corroborate the failure mechanism, cracking patterns, load and deformation at maximum strength and behavior in advanced stages of damage; 2) To check the contribution of horizontal reinforcement to strength and to inelastic deformation capacity in order to calibrate the current design criteria; 3) To collect information for proposing allowable lateral drift for seismic design purposes, and its correlation with the lateral ductility of systems, and the seismic reduction factor for elastic seismic loads; and 4) Obtain information on the overstrength of the systems studied.

2. Design of the specimens

2.1 Geometry of specimens

The final geometry was selected to attain failure of specimens at high demands of inelastic deformation. Geometry constraints took into account the size of the shaking table (4 m side), maximum height of 4 m and maximum table payload of 196 kN (20 t). Design of the specimen considered the response of a half-scale three-story complex confined masonry structure tested by Arias and Alcocer [1, 2]. It was selected to study specimens of three stories with the same scale (1:2) and same construction system (confined masonry walls), with model floor height, H, of 1.25 m corresponding to 2.5 m in the prototype, which is typical of housing in Mexico.

The specimens have a wall system contained in a single plane at the center of the model in the test direction (Fig. 1). At each story, the structural system consisted of two 1.2-m square walls. Walls were separated by 440-mm wide door openings. In the direction perpendicular to the test, masonry walls were constructed at the ends to give out-of-plane stability to the specimens. Thus, square walls between axes 1 and 2 worked as T-shaped walls. A short wall between axes 5 and 6, connected to the perpendicular one at axis 6, was left for give it out-of-plane stability. Geometry and reinforcing details were the same in the three stories of each specimen.Materials

2.2 Materials

Models were built using solid handmade clay bricks. Walls were confined with tie-beams and tie-columns. Steel reinforcement characteristics and reinforcement ratio were representative of that used in typical construction. Mortar and concrete were scaled down. A preliminary wall with similar geometry as that of square walls was constructed and tested under quasi-static cyclic loading. Preliminary material properties are shown in Table 1-

2.3 Steel reinforcement

Steel reinforcement details are shown in Fig. 2. Tie-columns were reinforced with four 6-mm diameter bars in the longitudinal direction. Longitudinal reinforcement was cold-drawn to obtain a 412 MPa (4200 kg/cm²) yield stress. Walls were reinforced with 10.5-gauge wire (3.25 mm) hoops spaced at 90 mm; this spacing corresponds to 1.5 times the wall thickness. Hoop spacing was reduced to 45 mm in the three hoops at the upper and lower ends of tie-columns and at the edges of tie-beams around openings. Specimen M3ND-0 did not have horizontal reinforcement. Second specimen, M3ND-1, was reinforced with 3.96-mm (5/32-in.) horizontal reinforcement wires placed every five courses in all walls.

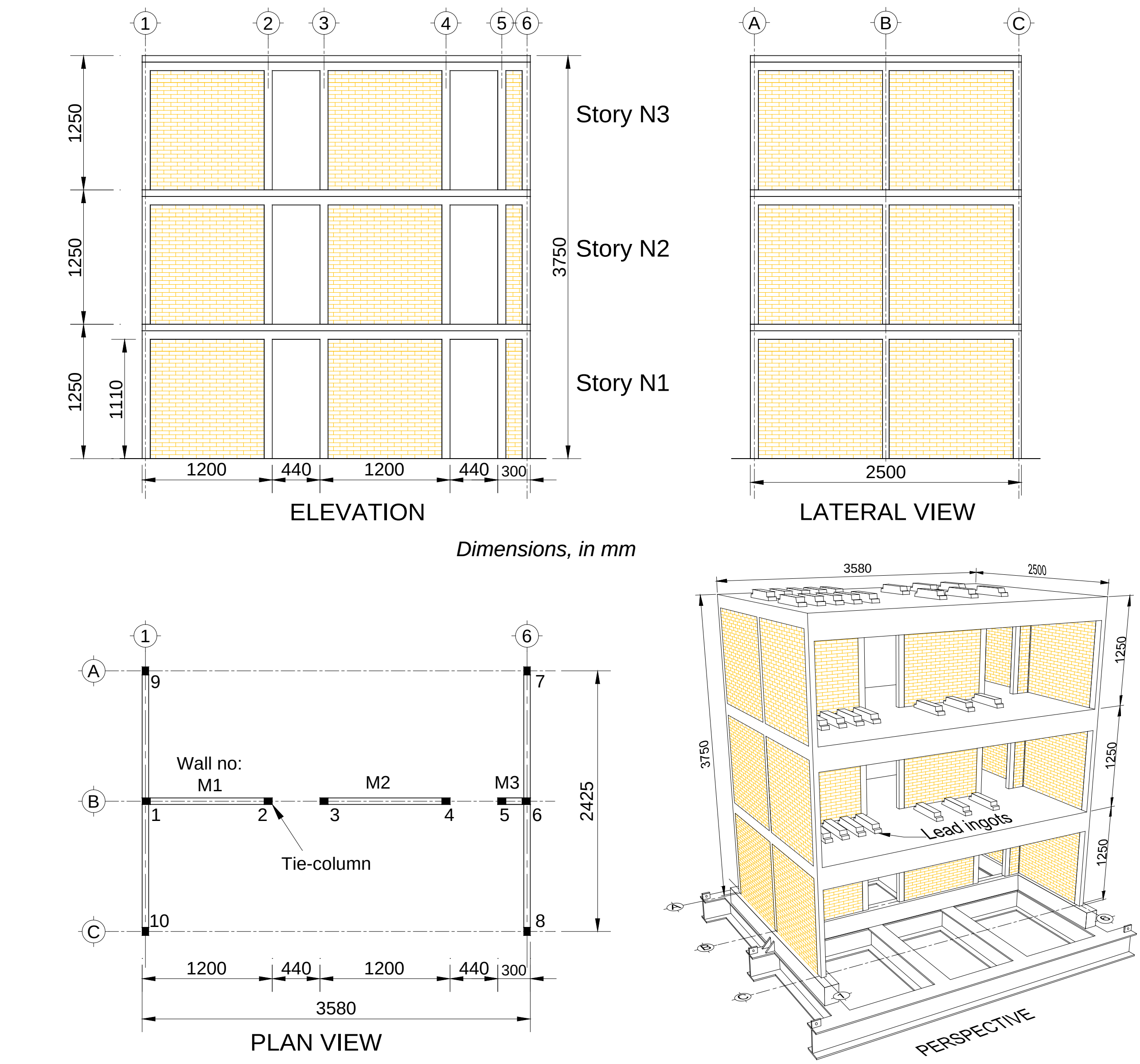


Fig. 1 – Geometry of the two specimens

Table 1 – Preliminary (nominal) properties of materials and symbols

Material or element, property	Symbol	Data
Tie-column concrete, compression strength	f_c’	20 MPa (200 kg/cm²)
Tie-beam and slab concrete, compression strength	f_c’	20 MPa (200 kg/cm²)
Longitudinal bars in tie-column and tie-beam, yielding stress	f_y	412 MPa (4200 kg/cm²)
Transversal wires in tie-column and tie-beam, yielding stress	f_y	206 MPa (2100 kg/cm²)
Horizontal reinforcement wires, yielding stress	f_{yh}	600 MPa (6000 kg/cm²)
Solid clay brick, compression strength	f_p’	9 MPa (90 kg/cm²)
Mortar type I (1:¼:3), compression strength of 50 mm-cubes	f_j’	12.5 MPa (125 kg/cm²)
Masonry: design compression strength of prisms	f_m’	6 MPa (61 kg/cm²)
design shear strength (diagonal compression of wallets)	v_m’	0.95 MPa (9.6 kg/cm²)
average of diagonal compression strength tests in wallets	v_m	1.4 MPa (14.5 kg/cm²)

2.4 Instrumentation

Models were extensively instrumented to understand local and global behavior. Ten accelerometers, three in each floor (at floor center and at opposite corners of the slab) and one at the base of the structure, were installed. Transducers used had a capacity to measure 2g and 4g (g = acceleration of gravity). In addition the shaking table has two accelerometers on the platform.

To register the behavior of reinforcing steel strain gauges of 2 and 3 mm in length were used. Sensors were placed in the tie-columns to allow deducing the distribution of bending moments on the walls. In the case of some tie-beams instruments were placed in reinforcing steel in door openings to identify any bending behavior of the coupling system between walls. Finally, strain gages were placed in the first hoop of three tie-columns at the base, above the foundation. In Fig. 2 the internal instrumentation is shown in tie-columns and tier-beams of the specimens, and in the horizontal reinforcement only in the case of specimen M3ND-1 where was placed along the main wall diagonals in the first two levels, being that diagonals the possible location of future cracks.

For external measurement transducers were placed to measure the absolute horizontal displacement at the level of the three slabs, to reconstruct displacement and story drift histories. Additionally, 25 mm capacity transducers for vertical measurements and 50 mm transducers for the diagonals were placed. In Fig. 3 the arrangement of external instruments is shown..

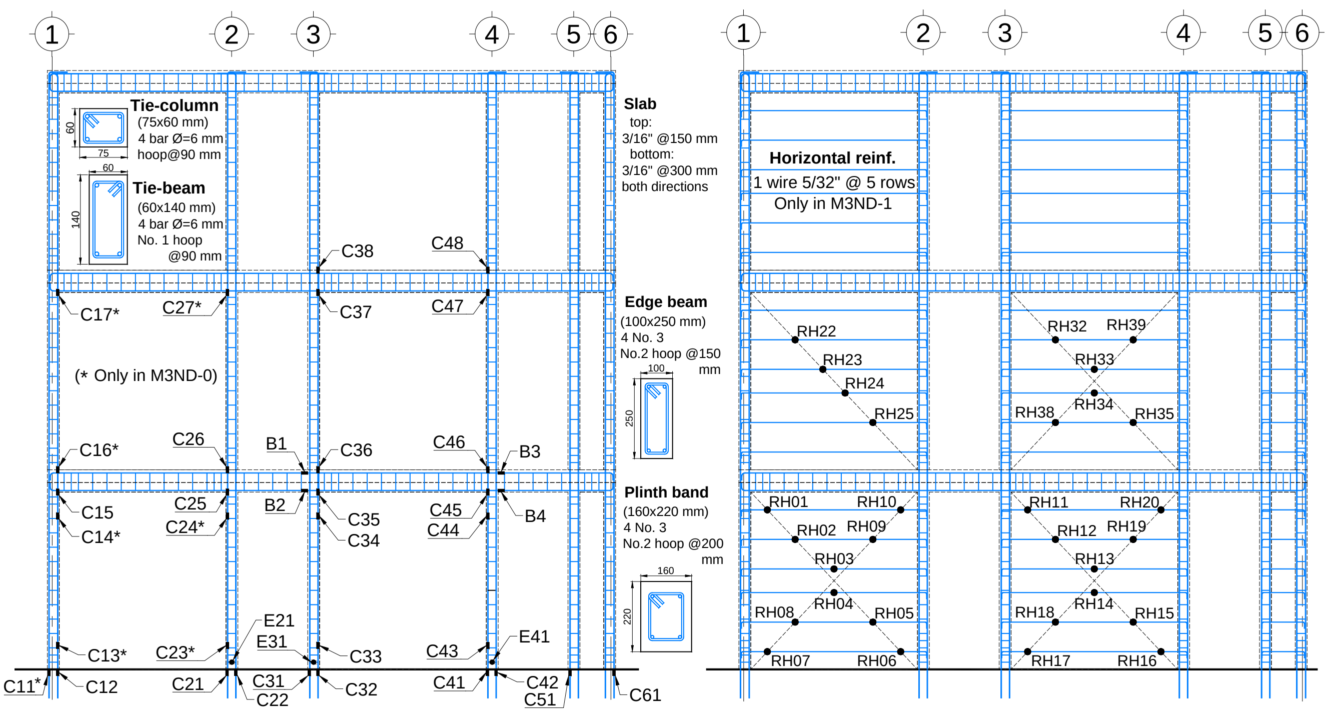


Fig. 2 – Steel reinforcement and strain gauge location; (horizontal reinforcement only in specimen M3ND-1)

2.5 Theoretical strength and stiffness

The design the experiment requires a theoretical prediction of the expected behavior of the structure, a careful consideration about the ability of the shaking table to generate the required base accelerations and displacements and the appropriate instruments for the expected displacements.

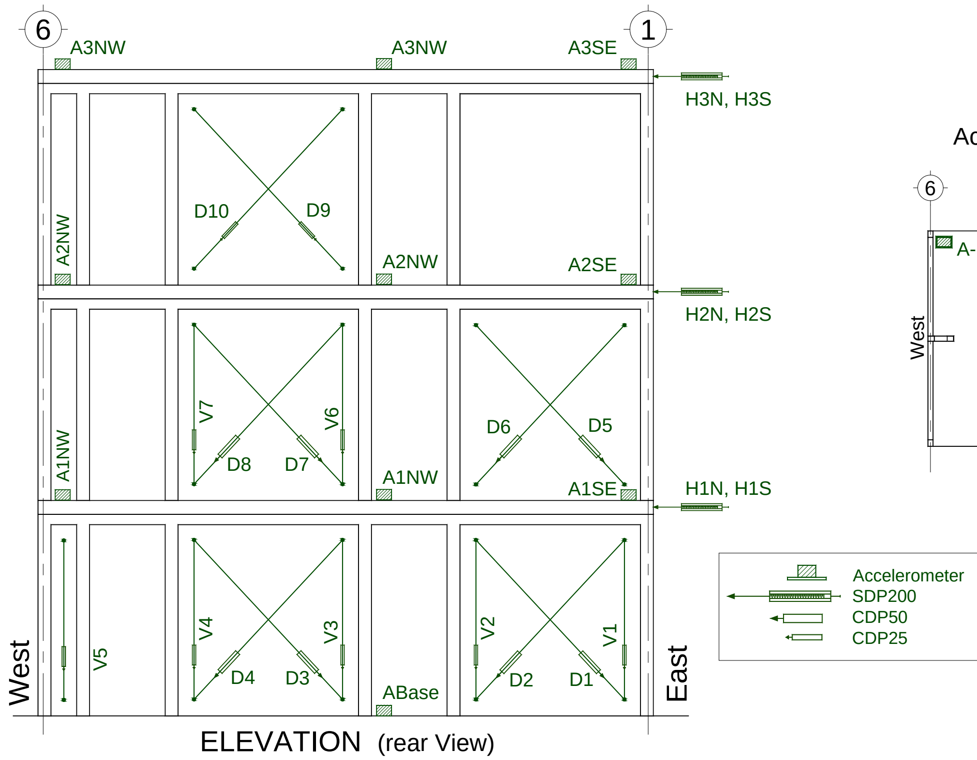


Fig. 3 – External instrumentation and accelerometers

2.5.1 Shear strength

2.5.1.1 Contribution of the masonry shear strength

To predict the strength of the specimens calculation was made using the proposed new the Mexico City Building Masonry Code (NTCM) of [4], which will enhance the current standard, based on a study of Cruz and Pérez-Gavilán [5, 6]. The resistance of the masonry of each wall, V_a , is:

$$V_a = (0.5v_m' A_T + 0.3P) \cdot f - M_{au}/H_k \leq 1.5v_m' A_T \cdot f \quad (1)$$

v_m' shear strength of masonry, obtained of wallet (small wall) in diagonal compression tests [7];
A_T transversal area of wall (thickness by length);
P vertical load on the wall. The vertical stress on the walls was $\sigma = 0.3$ MPa;
f factor taken into account the aspect ratio; $f = 1.0$ for square walls ($H/L = 1$);
M_{au} bending moment in the plan of the wall; because it has double curvature we may take $M_{au} = 0$.

In this calculation, only the contribution of the two square walls of 120 cm was considered, despising the short wall. In this study the average diagonal compression, $v_m = 1.4$ MPa, was used instead of design shear strength, v_m', for the prediction of shear strength: $V_a = 56.9$ kN.

2.5.1.2 Contribution of horizontal reinforcement to shear strength

With the new NTCM the strength for the horizontal reinforcement is:

$$V_{sR} = \psi q_e A_T \quad (2)$$

$$\psi = \frac{V_a}{(q_e A_T)}(k_0\, k_1 - 1) + \eta_s \quad (3)$$

$q_e = p_h f_{yh} \leq 0.1 f'_m$ is the effective contribution of horizontal steel, where $p_h = A_{sh}/(s_h \cdot t)$; s_h is horizontal reinforcement spacing, A_{sh} steel area and t the wall thickness;

k_0, k_1 constants; for square walls: $k_0 = 1.3$, and $k_1 = 1 - 0.45 q_e$.

η_s factor, where $\eta_s = 0.55$ if $f'_m = 6$ MPa (60 kg/cm²).

One wire 4 mm (5/32 in.) in diameter, area $A_s = 12.4$ mm², every five courses: $q = p_h f_{yh} = 0.0011 \times 600 = 0.66$ MPa, meets minimum and maximum ($q_{min} = 0.3$ MPa, $q_{max} = 0.15 f'_m = 0.9$ MPa). Thus: $V_{sR} = 21.6$ kN.

For specimen M3ND-0 the shear strength on the base in the test direction was the sum of the two squares walls: $V_{R,\mathrm{M3ND0}} = 2V_a = 114$ kN. The total weight of the specimen was $W_T = 102.6$ kN, the shear to weight ratio represented 1.1 times the gravity acceleration ($a_{\mathrm{M3ND0}} = 1.1$ g).

In the case of the specimen M3ND-1, the expected final resistance for each square wall was the sum of the contribution from the masonry plus that of horizontal reinforcement: $V_a + V_{sR} = 78.5$ kN, so the specimen, with two equal walls, had a nominal resistance: $V_{R,M3ND-1} = 157$ kN, $a_{M3ND-0} = 1.53$ g.

2.5.2 Prediction of stiffness

For the calculation of deformations, vibration periods, estimated displacements and internal forces distribution, a numerical model using SAP2000 software was developed; it was modeling with the wide-column method (modeling each wall as a column in the middle of the wall and with rigid beams inside the wall width). Nominal mechanical properties were taken as:

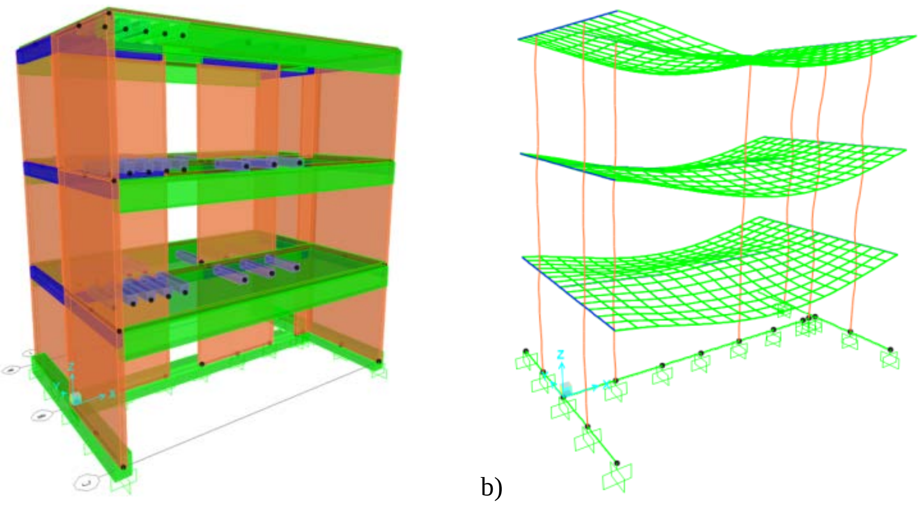
- $E_c = 8000\sqrt{f'_c} = 8000 \times \sqrt{200} = 113{,}137$ kg/cm² (11,100 MPa), modulus of elasticity for concrete;
- $E_m = 600 f'_m = 600 \times 60 = 36{,}000$ kg/cm² (3,531 MPa), modulus of elasticity for masonry [3];
- $G_m = 0.4 E_m = 14{,}400$ kg/cm² (1,413 MPa), shear modulus for masonry [3, 4].

In Fig. 4 the computer model, showing the geometry of the specimen and its elements, is presented. The model was analyzed fixing the walls in the base and placing lead ingots as additional loads distributed on the slabs. These ingots represent the dead load additional to the self-weight of the slabs plus instant live load for housing.

To determine what would be the loads to be applied to the specimen a dimensional scaling analysis was made, where the linear dimension scale factor is $S_L = 2$ (for scale 1:2). In the case of loads the scale factor of weights equals the scale factor for volumes S_{Volume}, admitting that material would be used with equal density in the prototype and in the model ($S_{Density} = 1$), i.e. a factor $S_{Weigth} = S_{Volume} = 8$. For loads distributed on the slabs the scale factor corresponds to be the area scale: $S_\omega = S_{Area} = 4$.

The slabs were modeled with a mesh of "shell" finite elements and with edge beams modeled as "frame" elements. The walls were modeled as wide-columns by frame elements and the area and moment of inertia was modified considering the increase in the geometric properties of the section with concrete tie-columns using the transformed-section method.

As a result of the analysis dynamic properties of the specimen, including a fundamental period $T_1 = 0.12$ s was determined in the direction of the test. In a similar analysis for the prototype a period of 0.24 s was defined (the time scale factor is $S_{Time} = 2$).



a) b)

Fig. 4 – Numerical model using wide column method: a) Geometry, b) Deformed shape for gravity load case

2.6 Selection of the ground motion

Checking the shear strength expected to lead to cracking and the strength (maximum shear) to each of the specimens, it was determined that the required ground motion must generate a minimum spectral acceleration of 0.7g in prototypes. Such acceleration should be representative of the worst case of seismic demand in the Mexico City soft soil zone for this type of structures. Using the Mexico City Building Code (MCBC), design spectra for soil period of $T_s = 1.5$ s, was selected to submit the prototype with period of 0.3 s.

In Fig. 5, the required spectrum is shown in blue line, for the prototype structure with a plateau between 0.2 and 0.5 s with a seismic coefficient of 0.7 g, along with the design spectrum (without overstrength) required by the MCBC. Note that for a period of 0.3 s the seismic demands match. Using the scale factors for accelerations, $S_{Acc} = 0.5$, the spectrum to use in the dynamic test had a maximum of 1.4 g, with characteristic periods of from 0.1 to 0.25 s, so that the generated synthetic accelerogram is shown in Fig. 6 along with its calculated response spectrum [8]. This record was called "test stage 100%.

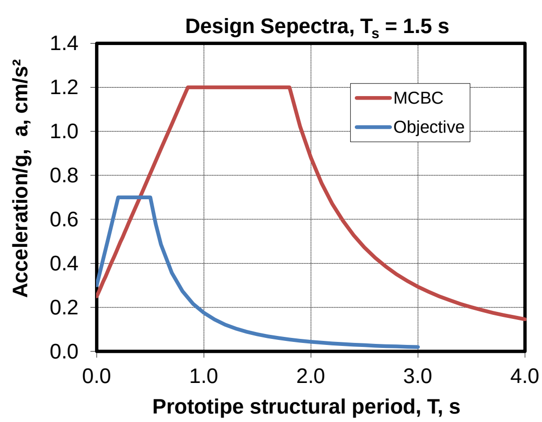


Fig. 5 – Target response spectrum and design spectrum for Mexico City Building Code

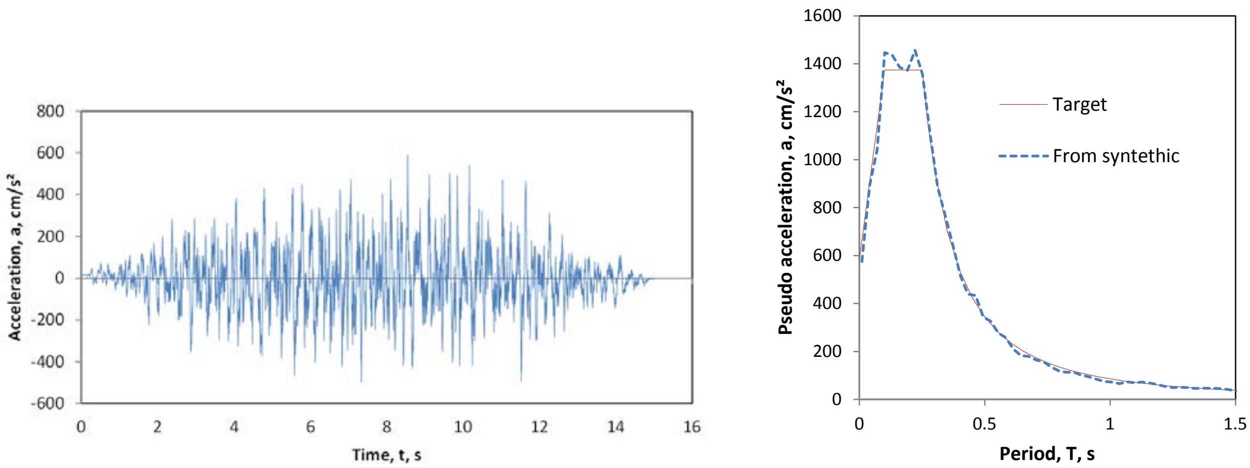


Fig. 6 – Synthetic accelerogram and response spectrum for 5% damping ratio

3. Specimen construction

To manufacture the masonry units the selected option was cutting pieces from bricks of normal size. The walls were built bonding bricks with mortar, one part (unit volume) of cement by ¼ of lime and three parts of sand (1:¼:3). Toothing was practiced in the ends of the walls cutting the corners of the units. Construction of the tie-columns in each story was conducted in two stages, casting at half height (no concrete additive was used). Each slab formwork was prepared and the slab reinforcement placed preparing the concrete cast. For concrete, 9 mm gravel and clean sifted sand was used. In the specimen M3ND-1, horizontal wires were located at every five courses, having 90° hooks anchored to the tie-columns. Fig. 7 shows one of the specimens and their placement on the platform of the shaking table by the laboratory crane.

Fig. 7 – Final view of specimen and its movement to the shaking table platform

4. Dynamic test

4.1 Description of behavior and failure process

4.1.1 Specimen M3DN-0

The first specimen was tested under accelerograms whose maximum acceleration was scaled with respect to synthetic ground motion generated: 0.25, 1.0, 1.5 and 2.0 times the synthetic ground motion. The first test was done to 0.25 times the synthetic accelerogram (stage 25%). The following tests were 100, 150 and 200%. The first cracking at the base of the square walls of story 1 (ground floor) occurred under stage 150%. The cracking was slightly inclined, as shown in Fig. 12.

During the application of the ground shaking to 200% (factor of 2.0) the specimen came to its strength (maximum seismic forces) with damage characterized by severe inclined cracking pattern in the walls of Level 2 (intermediate floor). In Fig. 8 cracking in the specimen is shown, and the following figure shows details of the registered damage.

Fig. 8 – Final crack pattern in the Story 2 of specimen M3ND-0, test stage 200%

Fig. 9 – Final cracking detail and damage of tie-column at Story 2, specimen M3ND-0

4.1.2 Specimen M3DN-1

For the test of the specimen with horizontal reinforcement the test stages of 25%, 100%, 150%, 200%, 250% and 300% were applied. The first damage was obtained when the test stage 100% was applied and the occurrence of a horizontal crack was observed in the second and third course in the central wall M2, Level 1 (ground floor).

When test stage 150% were applied it appeared a new horizontal crack in the central wall M2, Level 1, and the first inclined crack in the East (M1) wall was formed. When test stage 200% was applied the central wall M2 was cracked (Level 1) and the inclined cracking in the M1 wall was developed. At this stage the crack due to the initial horizontal pattern penetrates the base of the tie-column no. 3 in East side of the wall (at left in photo).

It was continued with the following test stages, increasing to ground motion of 250 and 300%. In these stages damage generalizes at the base of the tie-columns of the central wall M2 in Level 1 with a evident sliding between the first and third row, exposing the core of the tie-columns no. 4 and the hook of the first horizontal wire. A pattern of inclined cracks in the wall M1 is completed, but without damage to their tie-columns.

Finally, the test stage to 350% was applied and it was reached the strength of the specimen with the opening of the main inclined cracks and breaking several horizontal wires of both square walls in level 1 (ground floor). In Fig. 10 it is shown the final cracking pattern.

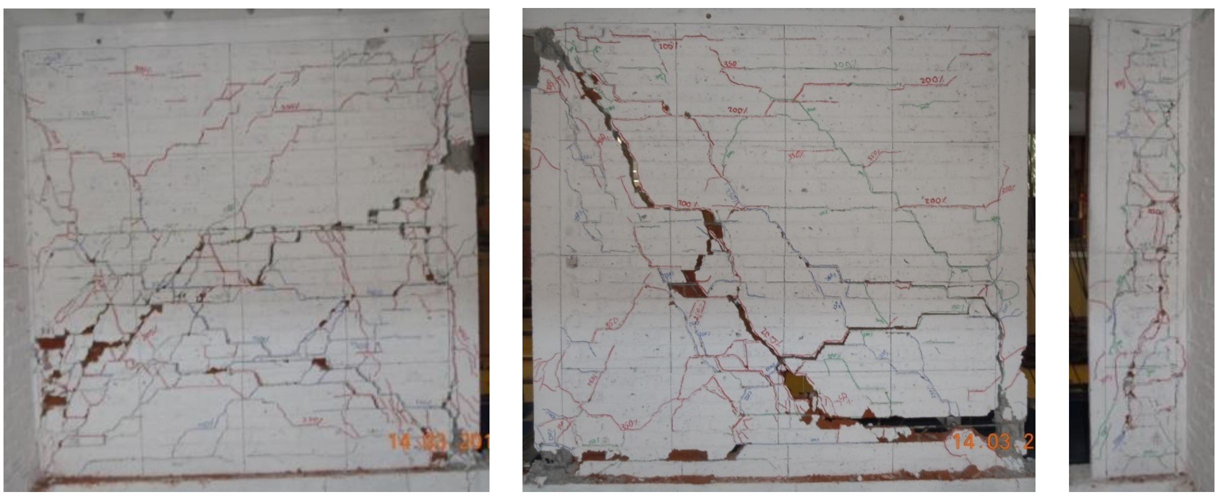


Fig. 10 – Final cracking pattern of Story 1, specimen M3ND-1, test stage 350%

Fig. 11 shows details of the final damage condition: dislocation of tie-columns no. 3 at its base, and tie-column no. 4 in its top; also breaking of one of the horizontal wires is shown.

Fig. 11 – Final cracking pattern, detail of tie-column ends and the fracture of horizontal wire

4.2 Comparative results between specimens

In the Table 2 it is shown the global results of each dynamic test, reported for the story that had the maximum damage and where presented the failure mechanism. In the table V_{max} is the maximum absolute value of story shear calculated as the sum of the horizontal seismic forces above the story; D_{max} is the maximum absolute value of the story drift ratio calculated as the difference between horizontal displacement, δ, in two adjacent slab divided by the story height ($H = 1250$ mm), $D_i = (\delta_{i+1} - \delta_i)/H_i$.

For having a comparative information about the experimental acceleration registered in the test it has been included in Table 2 the values of the absolute maximum acceleration measured at the base, a_{base}, and at the top of the specimen $a_{max} = a_3$.

In the case of seismic forces they were calculated as the absolute acceleration times the story mass, $F_i = m_i a_i$. In this analysis the acceleration history data was that registered by the central accelerometer and it was verified that the three accelerometers in each story registered very similar information. The mass was first estimated by a geometric calculation using the experimental weight of materials, and checked obtaining the total weight of each specimen by load cells when the specimen was moved to the platform: $W_1 = W_2 = 35.6$ kN, $W_3 = 31.4$ kN.

Table 2 – Global results of tests

Test stage	a_{base} cm/s²	a_{max} cm/s²	Story no.	V_{Elast}, kN	V_{max}, kN	D_{max}, mm/mm	Description
M3ND-0							
100%	515	1,259	2 1	70 95	71 88	0.0021 0.0043	No damage
150%	703	1,598	2 1	81 96	90 108	0.0027 0.0018	Inclined cracks near the base
200%	976	1,894	2 1	121 144	96 116	0.0091 0.0024	Failure at story 2 with severe inclined cracking pattern
M3ND-1							
25%	84	279	1	21	17	0.00014	Initial elastic test, no damage
100%	423	1,107	1	88	76	0.0009	Horizontal crack at the base
150%	481	1,251	1	97	89	0.0011	First inclined crack
200%	904	1,977	1	153	126	0.0037	Inclined cracks and penetration in tie-column
250%	1,241	2,228	1	204	147	0.0064	Horizontal crack at base, damage of concrete
300%	1,369	2,431	1	242	159	0.0103	Horizontal sliding along the base crack
350%	1,707	2,333	1	322	154	0.0194	Failure of specimen with opening of inclined cracking and horizontal wires fracture

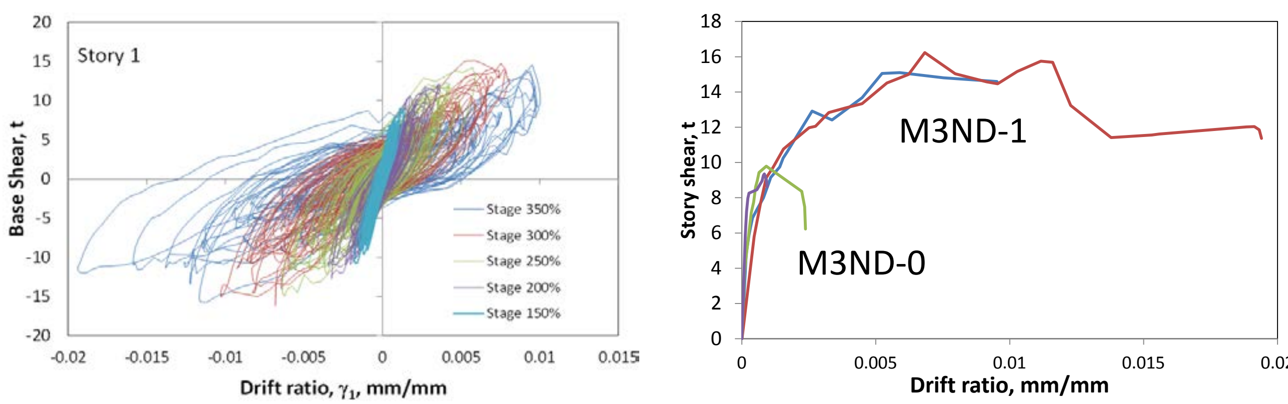


Fig. 12 – Hysteresis curves of M3ND-1, all test stages; and response envelops of two specimens

5. Concluding Remarks

As a preliminary result of the study, it can be highlighting the following::

- The behavior of the specimen was consistent with the failure mode expected by cracking by diagonal tension in the square panels, and showing no problems for bending effects, slides or other possible failures.
- The calculated strength of the specimen without horizontal reinforcement, M3ND-0, of 114 kN was closely predicted compared with the test strength of 116 kN. Additionally, failure was expected at story 1 (ground floor) but it was presented at story 2 (intermediate floor). The explanation is that the masonry panels offered overstrength, but apparently, level 1 still exceeded the strength over that of level 2.
- In the case of horizontal reinforcement, M3ND-1, the predicted shear strength was 157 kN and the test reached 159 kN. The shear failure was presented by the yielding and fracture of horizontal wires.
- Comparing the equivalent elastic base shear with the experimental inelastic base shear, for the same experimental ground motion, the ratio V_{Elast}/V_{max} was 1.24 and 2.1 for specimens M3ND-0 and M3ND-1, respectively.
- The inelastic drift ratio to 80% of the shear strength, that is, after the strength decay 20%, had a drift ratio of 0.008 mm/mm for specimen M3ND-0, and 0.017 mm/mm for M3DN-1, both in the story that had shear failure.

6. Acknowledgments

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