Experimental analysis of a discountinuous wall-to-floor connection

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ABSTRACT: In this paper the authors present the results of a set of tests which were conducted both in the UA testing laboratory and in a construction site in order to experimentally assess the structural performance of a wall-to-floor connection obtained by a discontinuous anchoring of the masonry wall to the floor by means of reinforcing bars inserted into the wall on one side and embedded into the floor slab on the other. At this stage of the research, the ability of this connection to restrain the masonry panel from forces acting in the direction orthogonal to its own plane was investigated.

1 INTRODUCTION

Replacement of wooden floors with reinforced concrete slabs represents one of the most common techniques for the rehabilitation of existing residential masonry buildings. In seismic zones, this technique allows one to provide floors with sufficient membrane stiffness and to obtain an efficient linkage of all the masonry panels[1]. This technique, however, calls for the solution of a number of technical, economical and social issues[2], therefore it may represent a socially and economically valuable strategy to avoid replacement of still sound wooden decks and to provide them with all the features necessary to make them seismically effective.

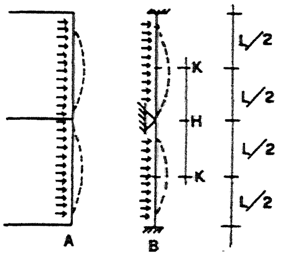

Figure 1. Schematic diagram of the wall-to-floor connection.

2 OBJECTIVES OF THE RESEARCH

A most commonly used technique for enhancing both the flexural and the membrane stiffnesses of a wooden floor is to cast a concrete layer of adequate thickness, reinforced by a mesh of steel bars[3], atop it. By anchoring this floor slab to the masonry panels, one can improve the global seismic response of the building and, in particular, the lateral response of the outside main walls to transverse inertial loading.

An efficient link between slab and panels may be obtained by first positioning the bars into properly sized boreholes drilled in the brickwork and then grouting the boreholes. The portion of bars sticking out of the wall will be embedded into the concrete slab. The length of the boreholes through the wall thickness depends on: 1) whether there exists an exterior facade that ought to be preserved and 2) whether the wall in question is a party wall.

If these are the cases, borehole depth should be limited to only a portion of the whole wall thickness.

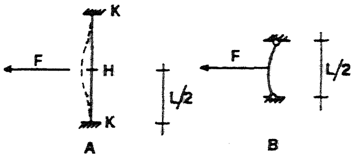

Figure 2. Simplified diagram of the wall-to-floor connection.

Nailing the wall panels to the floor slab makes it possible to accomplish the twofold purpose of: 1) having the panels firmly confined between the floors and the transverse shear walls, which significantly diminishes the out-of-plane effects, and 2) improving the story shear distribution among the walls. An enhanced capability of distributing story shears may be obtained by either exploiting the dowel effects of the reinforcing bars or skewing the steel mesh (by 45 degrees, for instance) in such a way as to exploit the bars' tensile strength.

The objective of this stage of the research was to

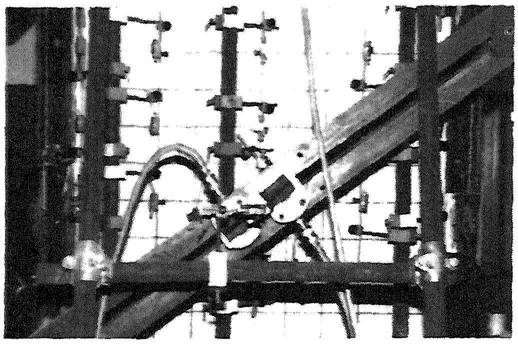

Figure 3. Laboratory test setup.

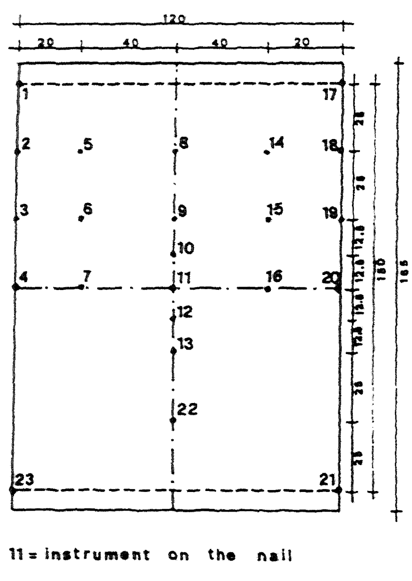

Figure 4. Model front view.

experimentally investigate on the ability of this wall-to-floor connection to support an outside masonry panel acted upon by lateral loading in the situation, more typical, where the borehole depth is kept shorter than the wall thickness. Due to the function exerted by this peculiar type of structural connection, each reinforcing bar will be thereafter called a "nail" and the connection itself will be similarly called a "discontinuous nailing".

3 LABORATORY TESTS

3.1 SELECTION OF THE STRUCTURAL MODEL

Scheme in figure 1 depicts a typical floor-to-outside wall subassemblage in a multistory building from which the structural model was derived.

Figure 5. Failure of connection.

The structural model simulated the behavior of the upper stories where: 1) the floor-to-wall connection is usually less efficient due to lesser stabilizing effects of the vertical loads and 2) the panels' out-of-plane flexural stiffness is usually less than it is at the lower levels due to an often significant reduction in thickness. Also, it was assumed that 1) the roof-garret structural system is sufficiently rigid to make it licit to consider a fixed support condition atop and 2) the membrane deformation of the shear walls, which brings about an out-of-plane rotation of the panel, is negligible so that the magnitude and distribution of both vertical and lateral loads, acting upon the panel, remain unchanged. Furthermore, since it is of importance to study the behavior of a vertical strip of wall located in a position centered with respect to the transverse shear walls because the restraining effects of these walls is less significant, it seemed not illegal to consider the strip as if it were isolated from the remainder of the panel.

From the foregoing assumptions, the mechanical scheme shown in figure 1 was derived, where the hinged support represents the nailing between the floor and the masonry panel. The practical difficulties of building a full-scale model, obtaining real fixed-end conditions and applying a distributed lateral load, called for a reduction of the model's dimensions and for a modification of both the loading and the restraining conditions. The final model layout is shown in figures 2a and 2b, where F indicates the pull-out load that is to be applied to the nail in order to have the same bending at cross-section H as that generated by a uniformly distributed lateral load applied onto the panel. Considering the deformed shape of the panel in figure 1, it was assumed legal to locate two hinges at the null-moment points, thus reducing the dimensions of the physical model (see figure 2b)[4].

The materials used for the model brickwork were as follows:

- solid clay bricks taken out of an old masonry building under rehabilitation. On the average, bricks were 0.285 m long, 0.139 m wide and 0.06 m deep;
- mortar composed of a mixture of calcium hydroxide and river sand with a ratio in volume of 9 parts of sand to 3 parts of calcium hydroxide; this composition was chosen in such a way as to reproduce the type of mortar

employed in most of the buildings of the Italian historic centers. The mortar joint thickness varied from 0.5 to 1.0 cm.

The raw panel, whose thickness resulted equal to 0.29 m, was given a 0.7 cm thick cement plaster with the twofold purpose of 1) making the surface flat and smooth to have the transducers safely and efficiently clamped onto it and 2) having the crack pattern clearly shown up during the pull-out test.

Upon completion of the model, a high bond type Fe38k 16 mm dia steel bar was positioned into a 30 mm dia borehole drilled at the panel's mid-span cross-section and then the borehole was grouted by means of an anti-shrinking cement mortar[5]. The borehole depth was 0.218 m but the bar was inserted by only 0.18 m.

4 TESTING SETUP

The panel was first subjected to a uniformly distributed vertical load in order to reproduce that actually transmitted by the roof and the garret in the real case. Load was applied by a hydraulic jack through a rigid steel girder. Upon completion of vertical loading, the nail was pulled out by means of a system of two coupled hydraulic jacks anchored to a reaction frame. The two jacks exerted a thrust on a set of three steel wedges fastened around the bar (see figure 3). The measuring instrumentation consisted of 23 mechanical displacement transducers with a sensitivity of 1/100 mm, positioned as shown in figure 4.

5 TEST RESULTS

As mentioned before, the panel was first subjected to vertical loading then the nail was gradually pulled out by a lateral force which was increased by steps of 2.10 kN. Readings of the displacements were taken at each loading step. The earlier cracking was of flexural origin as shown by the horizontal direction of fissures and occurred at the panel's mid-span cross-section under a force of about 10.5 kN. It was immediately followed by numerous cracks developing from the nail in a radial fashion. As the horizontal force was increased , the radial crack pattern spread out until a collapse mechanism formed under a pulling force of 14.7 kN.

Failure of the connection was originated by the slipping, along the mortar joints, of two bricks cemented to the nail with respect to the surrounding brickwork when the force exceeded the brick-to-mortar bonding strength (see figure 5).

The nail stub embedded into the panel's aftermost half thickness detached from the brickwork after crushing the grouting mortar and the bricks as well.

Analysis of the test results in terms of crack development clearly demonstrates that the abatement of the connection's strength was initiated by flexural cracking. Analysis of deformation confirmed this behavior: in fact, it can be seen from figure 6 that the lateral displacements along the panel's vertical axis of symmetry correspond to the elastic bending behavior of a simply supported beam up to 8.4 kN. Upon increasing of the horizontal force, the panel's initially elastic deformed shape gradually turned into a markedly bilinear shape in consequence of a rigid-body rotation of the upper half with respect to the lower about the horizontal axis at the cracked mid-span cross-section. It can be also noted that the deformed shape around the nail exibited a sharp increase in slope for post-cracking loads exceeding the bonding strength between bricks and mortar. This is a result of the progressive reduction

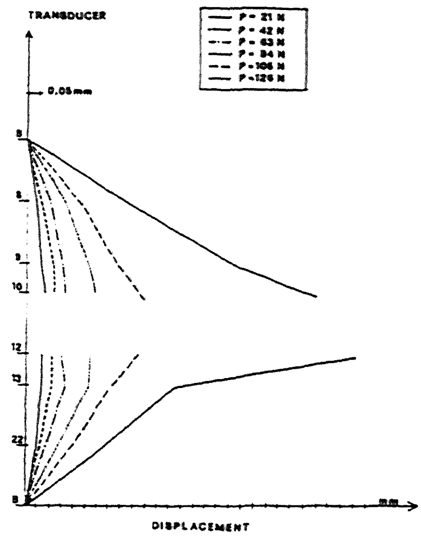

Figure 6. Model deformed shape.

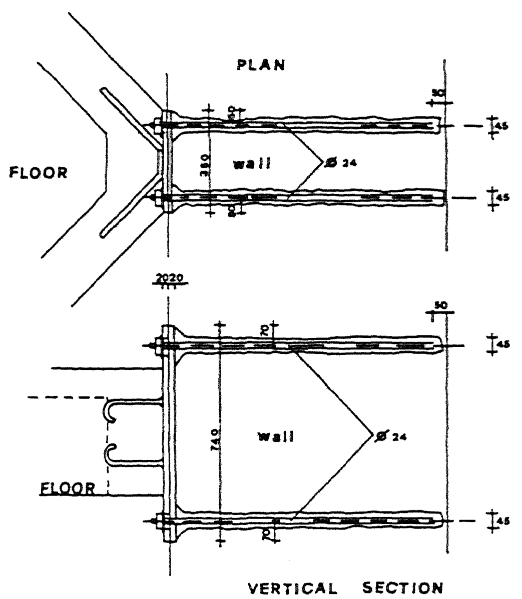

Figure 7. Schematic diagram of in-situ nailing.

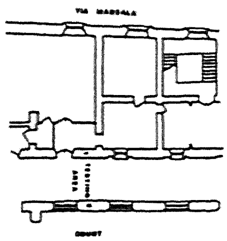

Figure 8. In-situ testing area.

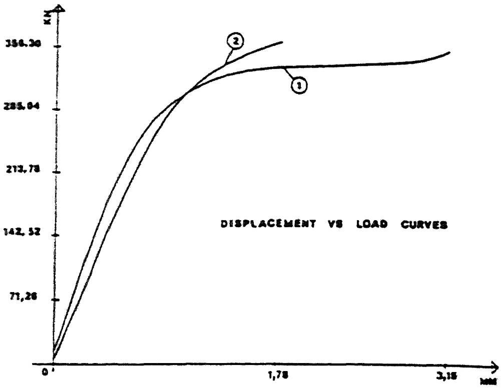

Figure 9. Load versus displacement curves.

of axial compression occurring at the cracked zone of the midspan cross-section. Such a reduction is responsible for the loss of bonding and brings about the slipping mechanism. Conversely, increase in compression at the uncracked portion of the mid-span cross-section enhances bonding between bricks and mortar. The connection's failure is then initiated by the pull-out mechanism when tension exceeds the brickwork tensile strength.

In order to double check this type of collapse mechanism, a new test was carried out upon another panel similar to the first one as to the materials' characteristics. However, bending span was limited to 0.6 m in order to reduce the panel's flexural behavior. The nail, of the same size as that in the preceding test, was inserted into and pulled out of the borehole in the same manner and in the presence of the same uniformly distributed vertical load.

In this test, cracking of a zone surrounding the nail commenced under a horizontal force equal to 15.1 kN, while ultimate failure of the connection occurred under a force of 18.2 kN. Besides the significant increase in force magnitude, the crack pattern was different from the preceding test in that it just exibited radial fissures developing from the nail. However, the collapse mechanism was the same, that is a block of brickwork, cemented to the nail, slipped out of the panel.

6 CONCLUDING REMARKS ON THE LAB TESTS

Based on the test results some conclusions may be drawn:

i) ultimate disconnection of the nailing is caused by the sliding of a block of brickwork around the nail out of the rest of the panel. This mechanism occurs when the horizontal force exceeds the brick-to-mortar bonding strength and is facilitated by early flexural cracking;

ii) an increase in bending stiffness and strength leads to the same type of collapse mechanism but the crack pattern becomes more complicated with fissures developing from the nail in a radial fashion. The pull-out strength increases with the bending stiffness;

iii) in neither of the tests did the nails slip out of the boreholes.

7 IN-SITU TESTING

7.1 SELECTION OF TESTING ELEMENTS

In the spirit of the tests conducted in the laboratory, a similar experiment was repeated on an existing building, housing the Ancona Hall of Justice, where rehabilitation works had been undertaken. Since the consolidation design required replacing of the interior masonry walls with a new moment resisting steel frame, it was thought that anchoring the original outside walls to the steel frame would prevent them from crushing under lateral seismic loads. A type of discontinuous anchorage was thus conceived which consisted of four nails cemented into as many boreholes as shown in figure 7. The testing area was selected in such a way as to carry out, at the same time, two pull-out tests upon two opposite wall strips differently constrained at their boundaries (see figure 8). A strip of wall no. 2 in figure 8 was chosen which was separated from the remainder by two adjacent windows. Due to its free lateral boundaries the behavior of this strip was expected to be of flexural type. Conversely, panel no. 1 in figure 8 was constrained by a transverse shear wall. In this case the pull-out test was expected to be less influenced by the panel's flexural behavior. The 0.7 m thick no. 1 wall was made of hewn limestone interleaved by layers of solid clay bricks, whereas the 0.6 m thick no. 2 panel consisted of a hollow wall of solid clay bricks with a rubble-filled inner core.

8 TESTING SETUP

In order to apply the horizontal force to both of the testing walls at the same time, a mechanical self-reacting device was fabricated. This device was composed of two countermoving platens which encased a hydraulic jack. Each platen was bolted to four reinforcing bars sticking out of the opposite wall. Pull-out forces yielded by the jack were measured by means of a pressure gauge mounted on the pump. Displacements of the two walls

Figure 10. Crack pattern of panel no.1.

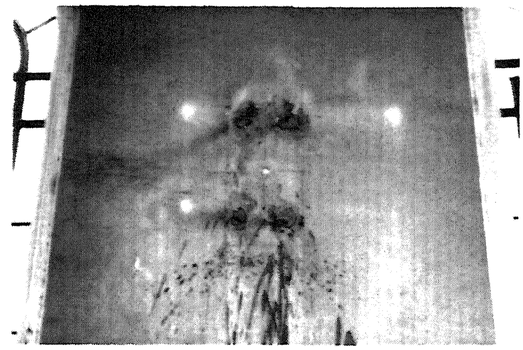

Figure 11. Crack pattern of panel no.2.

were measured at the center of each set of four nails by means of inductive transducers.

9 IN-SITU TEST RESULTS

The displacement vs load curves in figure 9 show that the structural behavior is nearly linear elastic up to a load of about 156 kN for panel no.1 and of about 213 kN for panel no.2, thereafter departing from linearity in a more significant extent for panel no.1.

In particular, this panel exibited a non-linear behavior in the range of loads between 284 kN and 327 kN, due to incipient cracking. From this load on, the panel's behavior became increasingly inelastic until the test was stopped because of the danger that a sudden lack of stability could occurr. The complex crack pattern consisted of fissures, both horizontally and vertically oriented, which were located in correspondence of the mortar joints between the stone blocks (see figure 10).

Panel no.2 exibited different behavior: the earlier flexural cracking occurred in the range of loads between 320 kN and 341 kN, the crack pattern remained almost unchanged upon subsequent loading and , under a horizontal force of 355 kN, the wall still showed little cracking (see figure 11).

In neither panel did the nails slip out of the boreholes.

10 CONCLUDING REMARKS

1) It is apparent from the tests that failure of the *discontinuous connection is never* caused by slipping of the nails out of the boreholes;
2) Collapse may occurr either when the pull-out force exceeds the bonding strength or when it overcomes the tensile strength of the brickwork, depending on the panel's restraint conditions at the boundaries.
3) the pull-out force that induces the earlier flexural cracking may be considered as the upper strength limit of the connection.
4) This limit allows one to evaluate the connection's efficiency by comparing the equivalent lateral uniformly distributed cracking load to the lateral uniformly distributed inertial load provided by the seismic codes.

[1] According to the results of a research on the repairing techniques used in Friuli after the 1976 earthquake, it was found that, on a sample of 90 buildings, about 19% of the total cost of rehabilitation was to be related to works executed on the floors especially for enhancing *the floor-to-wall connection's effectiveness.*
[2] Indeed, replacing of the floors always requires evacuation of occupants.
[3] This technique has been studied by some authors who have outlined the technical aspects and proposed a computing procedure for accurately sizing all the structural details.
[4] Such a model represents a typical building with an interstory height of 3 m.
[5] An anti-shrinking cement mortar was used in order to ensure efficient bonding between the nail and the brickwork