



Connections of Roof-Diaphragm to Perimeter Walls in Historic Masonry Buildings

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Abstract. Traditional seismic retrofit of historical masonry buildings is mainly aimed at inhibiting local mechanisms and at engaging the whole building into a box-like structural behavior through either perimeter ties, or floor and roof diaphragms. When the adoption of “minimally impairing” perimeter ties is unviable, floor and roof diaphragms are usually adopted. The effectiveness of such interventions requires careful design and detailing of their connections to the perimeter walls, which enable shear flow transfer from the out-of-plane loaded masonry walls to the diaphragms and from the diaphragms to the seismic resistant walls. Focus is made on the assessment of the structural performance of the connections of roof diaphragms to the perimeter walls. Stud connectors are investigated and a possible technical solution improving the connection capacity is presented. In the case of historic masonries, theoretical and numerical prediction of the connection capacity may be unviable because of the substantial heterogeneity of the masonry at local level. Accordingly, experimental in-situ tests may be the only viable solution to assess the connection capacity, thereby providing evidence of the feasibility of the intervention and providing useful reference design values to be used by design professionals. A specific experimental test is presented for the in-field measurement of the connection capacity, and experimental results are critically illustrated. A purpose designed testing frame is proposed, which allows mimicking the actual load and restraint conditions of the connections in a real application.

Keywords: Historic masonry buildings · Seismic retrofit · Roof diaphragm
Stud connection · Stud capacity assessment · In-field testing

1 Introduction

The priority mitigation measures aimed at reducing the seismic vulnerability of historical buildings frequently consist in the preliminary improvement of the quality of the masonry, if necessary, and in the adoption of either perimeter ties or floor and roof diaphragms. Such interventions are aimed at inhibiting local mechanisms, particularly

the onset of partial or global overturning of the perimeter walls [e.g. 1–3], and at enabling a global box-like behaviour. This paper focuses on roof diaphragms (or shells), which are usually adopted when less invasive intervention, such as perimeter ties are ineffective (see Fig. 1a). Roof diaphragms were first proposed as a retrofit measure in historical buildings following the Salò earthquake (Italy, 2008), and later adopted to enhance the seismic performance of several churches damaged in the Emilia earthquake (Italy, 2012) [4–7]. The roof diaphragm collects the inertia forces of the roof and the out-of-plane loaded masonry walls and transfers them to the seismic resistant walls. The roof shell is composed of perimeter chords that resist the shell bending action and of web panels that resist shear actions. The shell is usually made of wooden panels, a double wooden plank overlay, or light metal trusses, whereas the perimeter chords are made of steel plates [5–7]. The solution also applies to the strengthening of churches featuring transverse arches and vaults. In such cases the roof diaphragms, besides preventing out-of-plane mechanisms, provide a constraint on the extrados of the transverse-arches partly constraining the rocking of the piers and the differential rocking of the nave bays, thereby protecting the nave vaults [8, 9].

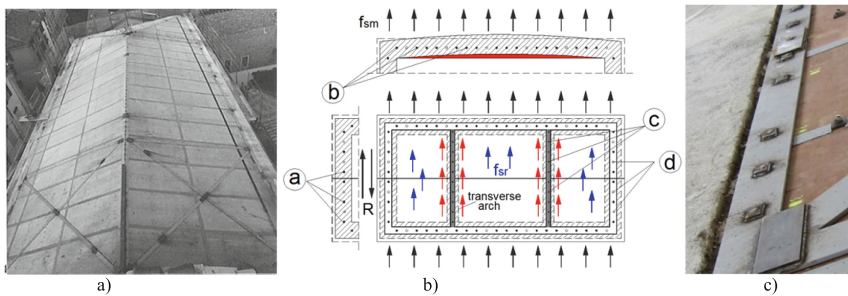


Fig. 1. (a) Aerial view of a roof shell structure; (b) types of connections required to secure the roof diaphragms to the perimeter walls

Careful structural detailing, both in the design and construction phases, is fundamental for the structural effectiveness of the roof diaphragm. Particular attention must be paid to the connections between the roof shell and load-bearing walls or transverse arches (see Fig. 1b). Connections are usually made by means of steel dowels or studs [5–7, 11]. Depending on their location along the wall top end, connections may serve multiple purposes: they transfer the seismic load collected by the diaphragm to the seismic-resistant shear walls (see Fig. 1c, type a); they retain the walls subjected to out-of-plane inertia loads and transfer their seismic action to the roof diaphragm (see Fig. 1c, type b); they secure the upper portion of the masonry walls of possible transverse-arches to the roof diaphragm (see Fig. 1c, type c) [10]. Additional grouted deep anchors are placed along the top of the masonry walls to prevent roof uplift; such connections also establish an effective bond between the restored upper courses of the masonry walls and the underlying portions (see d in Fig. 1c) [12, 13].

Specific detailing of the stud connection and the adoption of an improved lime-mortar overlay on the top of the masonry walls are proposed to improve the connection

strength. Without such improvements, the connection capacity would be substantially jeopardized by the reduced shear resistance of the masonry wall due to the absence of significant vertical confining action at the roof level [10]. Emphasis is made on the conceptual design of an in-field test. In-situ measurement of the connection capacity provides a substantial evidence of the actual feasibility of the retrofit solution, and provides the practitioner engineers with design values to be used in the proportioning of the intervention.

2 Stud Connection Securing Roof Diaphragms to the Masonry Walls

Connections between roof diaphragms and masonry walls are usually made by means of steel dowels or studs (see Fig. 2). Such connections meet the strict compatibility requirements of historic monument preservation for being minimally invasive and mostly reversible. Dowels are typically obtained from smooth galvanized steel rods and they are dry-driven into tight-fitting holes, which are core-drilled into the top of the masonry wall. Alternatively, they are placed into larger holes injected with either epoxy resins or lime-based mortars. It is relevant to notice that, unless specific improvements or pre-consolidation actions are adopted, the **capacity of stud connections** can be small and quite unreliable as a consequence of the reduced shear resistance of the masonry wall in the absence of significant vertical confining action at the roof level. Moreover, the upper layers of the masonry often exhibit quite relevant decay.

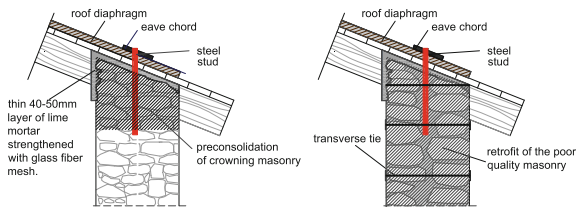


Fig. 2. (a) Masonry pre-consolidation obtained from rearranging the top courses of the masonry wall and by introducing an improved lime mortar overlay; (b) possible preliminary retrofit in the case of poor quality of the masonry [10].

2.1 Technical Solution Improving the Stud Connection Capacity

In order to improve the capacity of the connection, pre-consolidation of the masonry wall top courses was proposed [5–7]. Pre-consolidation can be obtained by rearranging the top courses of the masonry wall (by repositioning or replacing any loose stones and bricks, and by using good quality mortars), and by adding a thin structural lime mortar overlay (see Fig. 2a).

The adoption of the thin slab significantly reduces the shear stresses on the masonry wall. The load transferred by the dowel to the strengthened lime mortar layer spreads across the depth of the overlay and is transferred to the top courses of the masonry wall

as a significantly lower shear stress distribution distributed across the area (a), as shown in Fig. 3a. The actions along the stud shank are plotted in Fig. 3b. The connection attains its maximum capacity if failure occurs with a 2-hinge mechanism in the stud shank, therefore if no early failure of the masonry occurs. Such condition requires that the plastic hinge develops in the thickness of the mortar slab (i.e. the effective length l_{ef} in Fig. 3b, is contained within the mortar overlay thickness). In the absence of the slab, concentrated point shear loads would be applied directly to the masonry, where shear resistance is small because of the absence of significant vertical confining action. This would cause the masonry wall early failure prior to the yielding of the stud. For the overlay, premixed high-performance lime mortars with $15 \div 18$ MPa compressive strength can be adopted. The slab must also be strengthened with a few layers of glass fibre mesh, or with welded mesh made of galvanized steel wires, to improve the bearing strength [14, 15].

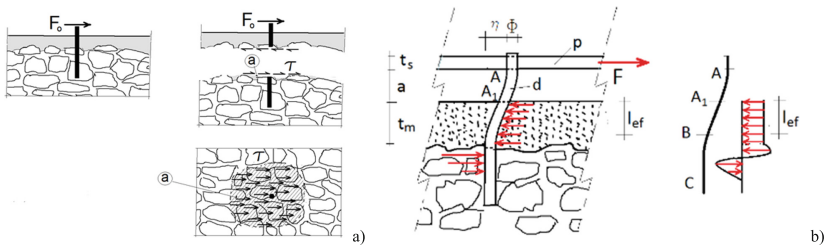


Fig. 3. (a) Improved lime mortar overlay, spreading the point load transferred by the stud into a shear stress distribution over the overlay-masonry interface; (b) dowel in ultimate condition, showing the bearing strength stress distribution along the stud shank [10].

It is worth noting that when the quality of the masonry is poor [16], or when the masonry walls are arranged as three or two leaves lacking an effective interlocking of the withies, the building may crumble in the event of an earthquake, without the onset of the typical local failure mechanisms involving macro-elements. In this case, apart from the proposed pre-consolidation measures, preliminary retrofit of the masonry must be carried out. An example of such a retrofit is shown in Fig. 2b, where the “monolithic” behavior of the masonry is enforced through the adoption of artificial bonders, such as transverse ties and structural plaster, and with the filling of the cavities with lime mortar injections. Discussion of such retrofit, however, lies beyond the scope of this paper.

2.2 Experimental Assessment of the Effectiveness of the Technical Improvement of the Connection

In order to assess the improvement of the connection capacity obtained by introducing the thin lime mortar slab, cyclic shear experimental tests were carried out. Three $3.4 \times 0.5 \times 1$ m specimens (A, B, C) were assembled in the laboratory, mimicking the top courses of a historic brick masonry wall. Specimen A was equipped with a 40 mm

high-performance lime mortar slab ($f_m = 15$ MPa after 120 days curing, and $E = 15000$ MPa), strengthened with a glass fibre mesh to improve its bearing strength ($f_h = 44$ MPa). Specimen B lacks any strengthening slab and for specimen C a slab of regular lime mortar was used. In each specimen 8 studs ($d = 20$ mm; S355) were dry-driven into core-drilled holes with 400 mm embedment length and 500 mm spacing. The system allows testing of two studs at a time. Cyclic loads of increasing magnitude were applied using a hydraulic jack until one of the studs (“the weaker stud”) failed. Details of the testing apparatus are given in [10].

Figure 4 shows the typical load-displacement curve obtained from the tests on the stud connections in Specimen A. All connections tested on Specimen A exhibited a ductile failure, with the development of two plastic hinges in the stud shank; average yielding displacement and load were equal to $s_{y,av} = 1.67$ mm and $V_{y,av} = 29.5$ kN, respectively. The average capacity is $V_{max,av} = 34.7$ kN. No dispersion of the results was observed, proving the reliability of the proposed solution. When no lime mortar overlay was adopted (see Specimen B in Fig. 4) no remarkable shear capacity is observed. The curve shows a very low connection capacity, with quite disperse results. Average connection capacity is approximately equal to 4.5 kN, occurring for relative displacement of about 3 mm. Collapse is always brittle, with crushing of the upper layer of bricks. In Specimen C, introducing an overlay made of lime mortar with regular characteristics ($f_m = 2$ MPa after 120 days curing), strengthened with glass fibre mesh (bearing strength, $f_h = 6.5$ MPa), a substantially lower capacity of the connection was measured and results were quite disperse. Failure of the connection was anticipated by the brittle failure of the masonry wall, either failing in crushing or in sliding shear. Connections on Specimen A remarkably outperformed the stud connections of both Specimen B and C, thereby proving the effectiveness of the proposed solution.

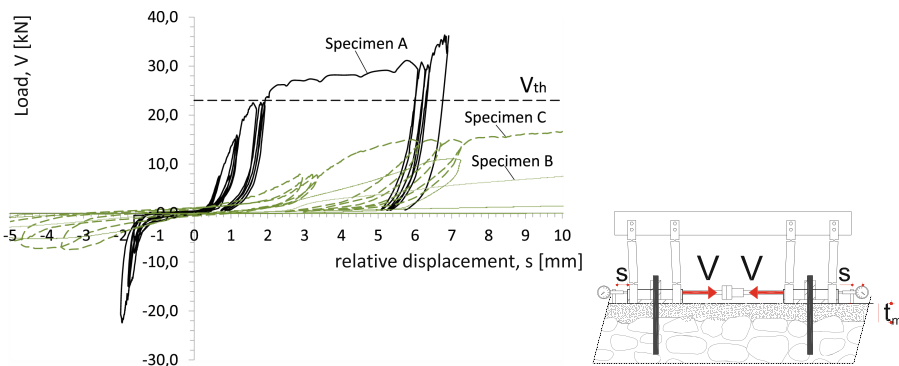


Fig. 4. Typical load-displacement curves obtained from laboratory cyclic tests for the weaker stud connection. Adapted from [10].

2.3 Assessment of the Connection Capacity

The capacity of the stud depends on the local properties of the masonry, as well as on some local structural details and features of the brickwork, on the position of the studs, etc. The complexity and heterogeneity of the masonry [17] make it unfeasible to address those analytical models available in the literature, which only apply to studs embedded within homogeneous continuum materials [e.g.10, 18,], nor global three-dimensional numerical modelling. For all these reasons, in-field experimental assessment of the capacity of the connection is needed.

In the case the pre-consolidation of the upper courses of the masonry is carried out (Sect. 2.2), reference to the analytical models can be done, provided that the improved lime mortar slab introduces a layer of closely homogeneous material that may plastically deform when subjected to the dowel action. The shear strength is attained with the 2-hinge mechanism developing in the stud shank if the effective length (see Fig. 3b) [10, 18], typically $1.5 \div 2$ times the dowel diameter, entirely develops in the improved lime mortar layer (this occurs when the slab thickness is higher than $4 \div 5$ cm). In this situation the capacity of the dowel depends on the steel grade, particularly on the yield stress of the steel and on the local characteristics of the lime mortar layer. As an example, the theoretical prediction of the capacity (V_{th}) for Specimen A (Sect. 2.2), as obtained with the analytical model by [10] is plotted in Fig. 4. The theoretical value matches well with the experimental average value of the yielding load. On the other hand, if the thickness of the mortar layer is lower than the effective length of the dowel, the analytical formulations are unreliable, and the load-bearing capacity of the stud connection can only be directly assessed through in-field tests.

It is worth noting that regardless the applicability and reliability of the analytical formulation, in-field tests are always necessary to: (1) assess the feasibility of the application of the roof diaphragm as a retrofit measure; (2) assess the correct execution of the connections and the effectiveness of the pre-consolidation of the top of the masonry wall, particularly concerning the accuracy of the construction steps and the actual attainment of high performances of the thin lime mortar slab; (3) provide a reference design load of the connection. A specific in-situ test is proposed in the following section.

3 In-Situ Assessment of the Connection Capacity

A special portable aluminium testing frame was developed for on-site assessment of the connection capacity (see Fig. 5). The testing frame replicates the actual load and boundary condition of the studs in real applications, and enables monotonic loading tests of pairs of studs. The studs (see 1 in Fig. 5) are connected to the steel plates (2), placed above the roof shell plywood panels and representing the perimeter chords of actual roof shell. A torque wrench (see 3 in Fig. 5) applies a shear action F , which is transferred to the plates by means of the vertical elements (see 4 in Fig. 5) connected by the transverse plate (see 4' in Fig. 5). Spherical washers see (see 3' in Fig. 5) enable rotation of the vertical elements (see 4 in Fig. 5). The transverse plate (see 4' in Fig. 5) has a low rotational stiffness and behaves like a truss connecting the vertical elements.

Rotation of the steel plates (see 2 in Fig. 5), caused by small eccentricity e between the shear action F and the sliding plane is prevented by the pressure applied by the plate onto the sliding surface (see Fig. 5b). Such pressure enables an additional friction contribution ΔF , which may result in the overestimate of the stud shear capacity and must therefore be deducted from the experimental results.

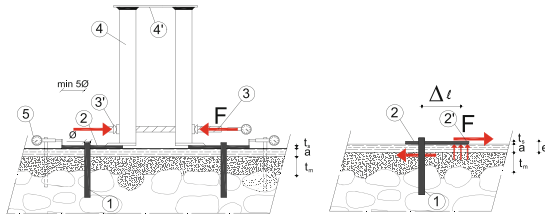


Fig. 5. (a) Schematic view of the testing frame; (b) small eccentricity of the F resulting in the overestimate stud capacity [10].

ΔF does not occur in the actual roof shell structure, provided that the connection rotation is negligible because of the continuity of the perimeter chord. In the on-site tests, the friction load ΔF can be evaluated by enforcing balance to rotation (Eq. 1):

$$\Delta F = F \frac{e}{\Delta l} \mu \tag{1}$$

where Δl is the distance between the stud and the edge of plate (2), μ is the wood-to-steel friction coefficient, ($\mu = 0.2 \div 0.3$); the eccentricity is $e = t_s + a$, where t_s is the steel plate thickness and a is the wood panel thickness. In the test set up, mechanical displacement transducers (see 5 in Fig. 5) measure the relative displacements between the plates (see 2 in Fig. 5) and the masonry wall.

Figure 6 shows the results obtained from the shear tests carried out on the connections securing the roof shell structure to the masonry walls in a few churches located in northern Italy. In all cases, the top of the masonry walls, made of rubble stone masonry of good quality [16], was pre-consolidated by introducing a high-performance premixed lime mortar overlay of 50 mm (having the same characteristics as in Specimen A). 20 mm S355 studs were driven into 400 mm depth holes. The load–displacement curves measured in the three construction sites are comparable in terms of connection stiffness and capacity; this provides an evidence that, after pre-consolidation, the connector capacity no longer depends on the masonry characteristic (which are different in the three cases), but on the strength of the overlay and the yielding stress of the stud. In all cases failure occurred with the onset of the envisioned 2 plastic-hinge mechanism of the connector.

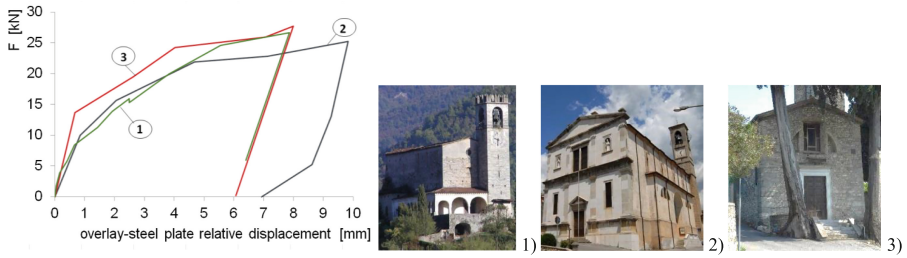


Fig. 6. In-situ tests: (1) Madonna della Rocca, Sabbio Chiese; (2) S. Maria Assunta, Bione; (3) SS. Trinità, Roè Volciano, Italy.

4 Concluding Remarks

This paper focuses on the design and structural performance assessment of the dowel connections securing roof diaphragms to the perimeter walls. Specific structural detailing for the pre-consolidation of the top portion of the masonry walls was proposed in order to improve the connection capacity, which would be otherwise jeopardized by the reduced shear resistance of the masonry wall in the absence of relevant vertical confining actions. A thin lime mortar overlay strengthened by glass fibre mesh, was shown to effectively serve the purpose. After pre-consolidation the connection capacity no longer depended on the heterogeneous properties of the masonry, but on the properties of the overlay and the yield strength of the studs. For poor quality masonry, apart from the proposed pre-consolidation, further substantial preliminary retrofit is necessary to enforce a monolithic behaviour of the masonry walls. Field tests, together with a special testing frame, were introduced to assess the connection capacity in order to assess the correct execution of the connections, particularly concerning the actual attainment of high performances of the thin lime mortar overlay and the viability of the envisioned strengthening solution. Experimental tests showed that after pre-consolidation, failure of the connection occurred with the development of two plastic hinges in the bolt shank, thereby providing evidence of the effective improvement in the connection capacity granted by the mortar overlay.

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