

Wall-to-horizontal diaphragm connections in historical buildings: a state-of-the-art review^{*}

Fabio Solarino^{a,b,*}, Daniel V. Oliveira^a, Linda Giresini^b

^a*ISISE, Institute of Science and Innovation for Bio-Sustainability (IB-S), Department of Civil Engineering, University of Minho, Guimarães, Portugal*

^b*Department of Energy, Systems, Territory and Constructions Engineering (DESTEC), University of Pisa, Pisa, Italy*

Abstract

Wall-to-horizontal diaphragm connections play a crucial role in the global stability of historical buildings under seismic actions. When these links are ineffective or absent, engineered measures should be considered to enhance the earthquake-resistant box-type behavior. Besides the great variety on the construction systems and materials, common damages were observed in recent seismic events showing the high vulnerability of local mechanisms promoted by the lack of structural integrity. Although the acknowledged importance of connections, this topic has been practically neglected over time among the research community and practitioners and only few of them focused on the influence of diaphragm-to-wall connections on the dynamic behavior of the building as a whole.

This paper presents a literature review of the traditional wall-to-floor or wall-to-roof connections in unreinforced masonry buildings and summarizes

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^{*}Corresponding author

Email address: solarino.fabio@gmail.com (Fabio Solarino)

typical and innovative strengthening solutions, taking into account the indications provided by the few design codes addressing this topic. Experimental laboratory researches are investigated, including shaking table tests on global and local scale, and cyclic or monotonic tests to characterize anchoring systems. An overview of the typical vulnerability assessment approaches and modelling techniques is given, considering present standards that account for connections.

Keywords: unreinforced masonry buildings, wall-to-diaphragm connections, out-of-plane collapse, strengthening techniques, design codes, state-of-the art

1. Introduction

Overturning of the perimeter out-of-plane (OOP) walls is considered the first-mode of failure and the last desirable in historical buildings [1, 2], cause of dramatic consequences, as shown in past seismic events in New Zealand
5 (1931 M7.8 Hawke’s Bay earthquake), California (1933 M6.4 Long Beach earthquake), San Francisco (1989 M6.9 Loma Prieta earthquake), Australia (1989 M5.6 Newcastle earthquake), or in Nepal (1988 M6.9 East Nepal earthquake) [3], among others. Old masonry buildings are most susceptible to seismic motions not only because they were built following the “rules of art”, but
10 also because of lacking anti-seismic criteria and poor quality of the materials [4].

Besides the in-plane (IP) stiffness of the floor, the connection quality between floors and vertical elements strongly influences the seismic response of existing URM buildings. Without proper connections, the walls behave like

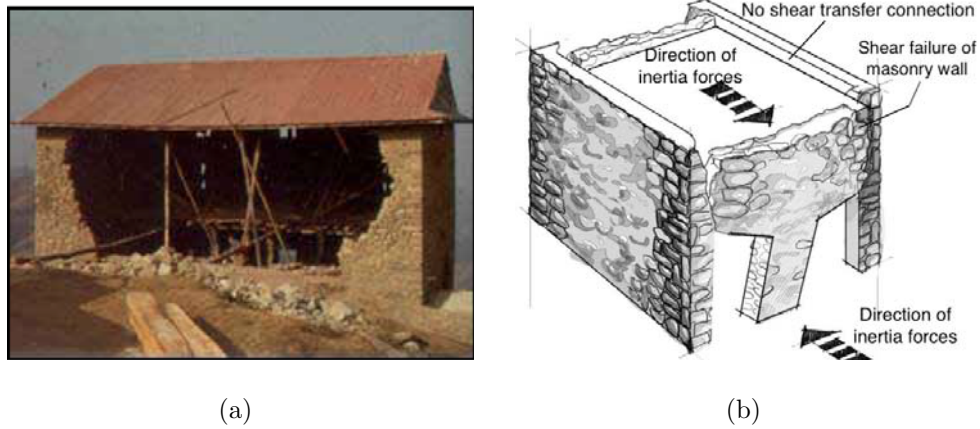


Figure 1: OOP failures of masonry walls due to the lack of wall-to-roof connections [6]: (a) collapse of a long wall in the 1988 East Nepal earthquake; (b) Inertia forces causing local failure mechanism

15 a tall unrestrained cantilever vibrating laterally, highly vulnerable to flexural OOP action [5, 6] (Figure 1(a)). Moreover, diaphragms play a key role in the transmission of the seismic actions through the shear connections to the lateral resisting walls (Figure 1(b)). Vertical seismic actions can also make the weak supports ineffective, causing the partial or total collapse of the structure. Effective solutions, together with the adoption of general seismic
20 criteria, lower the vulnerability of the URM buildings ensuring the “box-behavior” and activating global equilibrated mechanisms [7] (Figure 2(a)). On the other hand, inappropriate use of retrofit techniques can increase the seismic risk in existing constructions, as observed in failures during the 1997
25 Umbria-Marche earthquake (Figure 2(b)), where the increased seismic forces due to the new heavy roof caused the separation from the supporting walls. Researchers mainly aim at understanding the global seismic behavior of the building neglecting the influence of connections, and strengthening solutions

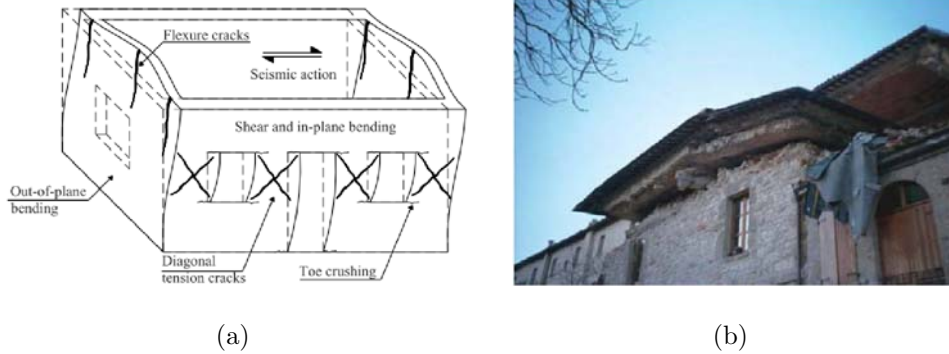


Figure 2: URM buildings seismically damaged: (a) global damage pattern embraced by Tomažević [7]; (b) stiff diaphragm disconnected from walls [1]

are suggested for the solely primary structural elements [8, 9]. The importance to account for OOP loading of the walls and to guarantee proper connections is highlighted in design codes and guidelines, yet it is not clear how to account for these effects in analytical models. Eurocode 8, part 3 [10] gives general technical criteria specifically for masonry structures, for which inadequate connections between floors and walls or between roofs and walls should be improved and OOP horizontal thrusts against walls should be eliminated, but no assessment equations can be found to verify the quality of the current connections. The 2008 Italian code [11, 12] allows to perform kinematic analysis to assess local mechanisms of masonry structures constituted by macro-elements with a displacement-based approach. No suggestions can be found, however, on the value of stiffness of the connections between the OOP wall and the possible diaphragm. Circolare n. 26 [13], aligned with NTC08, highlights the importance of reducing lacks in connections proposing the use of tie rods, external bounding and ring beams. It is worthy to notice that this code only mentions some connection details for existing constructions

45 (e.g. use of steel elements to anchor the timber beam to the masonry wall
at floor level) without giving clear guidelines on how to account for them
in the numerical models. Few details and recommendations can be found
in the new NTC2018 [14] suggesting mechanical or injected anchor elements
to increase the pull-out resistance. No analytical formulation is, however,
50 suggested for the design and assessment of those connections.

This paper presents a state-of-the-art of traditional and innovative wall-
to-horizontal diaphragm connections solutions used in historical buildings
to enhance the OOP strength of the URM walls. Section 2 reports the
typical traditional connections, while Section 3 describes main strengthening
55 techniques to reinforce wall-to-diaphragm connections in existing buildings.
Experimental and numerical analyses are critically discussed in Sections 4
and 5, respectively, by considering the different modelling techniques and
the role of current standards.

2. Typical traditional connections

60 Investigating the actual type and characteristics of the connections be-
tween vertical and horizontal structures is a complex but fundamental task,
influencing the accuracy of assessment models to evaluate the building vul-
nerability and to design the optimum strengthening solution. Moreover, de-
tails are not on sight and no drawings are available for ancient buildings.
65 Post-earthquake surveys from the inside are often risky.

The great variability of connections mainly relies on the typology of struc-
tural elements (single, multi-leaf or cavity walls, columns, floors, roofs, arches
and vaults), and on their materials (bricks, stone, timber, steel and reinforced

concrete), but similarities can be found among construction systems in differ-
ent countries worldwide. Either one- or two-way timber floors are typically
70 used in masonry buildings, while timber truss is the most common type of
roofing system. Traditional examples of connection details are summarized
and discussed below.

A brief description about construction details typically found in older
75 URM buildings in the US was given by Lin and LaFave [15], citing the Ar-
chitectural Graphic Standards [16] and the I. T. B. Company [17], indicating
some methods by which wooden floor joists were connected to brick ma-
sonry walls. Suggestions about how to perform proper connections were also
summarized by Cestari and Lucchio [18] or Russel et al. [3], who analyzed
80 different construction typologies in Italy and other countries all over the
world.

Certain rudimentary arrangements of stones or bricks could serve as cor-
bel support for the joist as shown in Figure 3, providing a flat and stable
support for the timber beams [19, 20, 21, 22]. The simplest and oldest prac-
85 tice was to fix the joist directly in a slot inside the wall for a depth equal
to the same thickness of the wall or to the half, for thin or thick walls re-
spectively [23, 24]. Other authors indicated a fixing depth of $0.25\text{ m} - 0.30\text{ m}$
according to the masonry walls thickness; for considerable timber sections
and for walls with a thickness of more than 0.50 m , Emi [25] suggested
90 $0.32\text{ m} - 0.33\text{ m}$ depth. The beam was simply supported from the walls, com-
monly constructed around the beam itself, either tightly filling with masonry
the recessed support or, by using weak grout to fill an oversized rectangular
cavity housing the support for the beams [26]. Historical treatises proposed

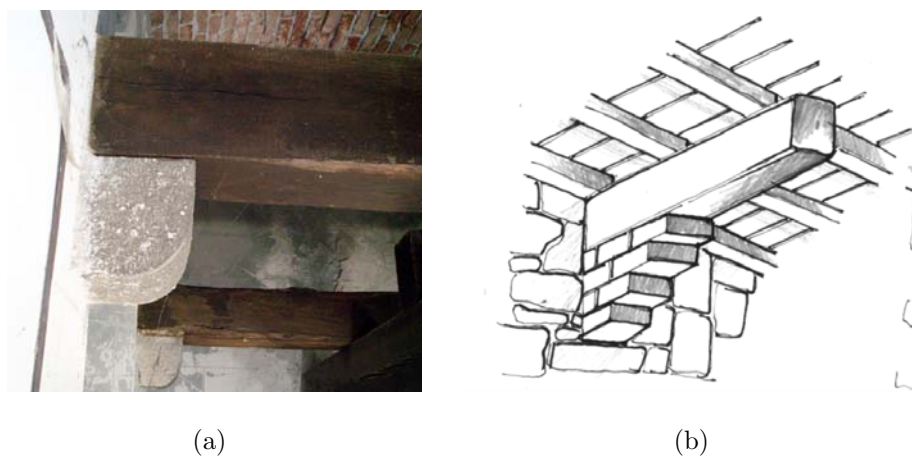


Figure 3: Timber beams on corbels: (a) Connection detail inside Castello d'Albertis, Genova, Italy [22]; (b) Example of bricks arranged as corbels to support timber beams [19]

solutions to avoid the decay of headpieces of the timber beams infixed in
95 the masonry, such as, leaving a gap between the timber and the masonry
allowing airing [20, 25], sometimes including a hole on the outer wall covered
with a grid [27]. Formenti [28], Donghi [29] and Chevalley [30] suggested to
form a box around the headpiece through air bricks or insert hardwood or
stone brackets under the headpieces of the floor beams. Further solutions
100 comprised timber wall-plate ($0.12 - 0.15\text{ m}$ thick) fixed in $3/4$ or in the
entire thickness of the wall linked to the joists through mortise and tenon
joint type [31]. Another way to better distribute the load on the wall was the
adoption of wrought timber beams as shown in Figure 4(a). The peripheral
beam could also be supported on stone corbels to avoid reduction of wall
105 thickness of upper floor (Figure 4(c)). Timber struts were often adopted to
reduce the span (Figure 4(d)). Common New Zealand URM constructions

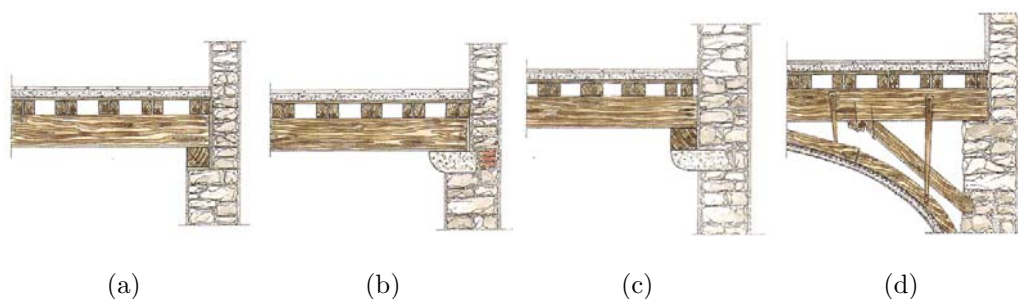


Figure 4: Examples of typical floor-to-wall connections [22]

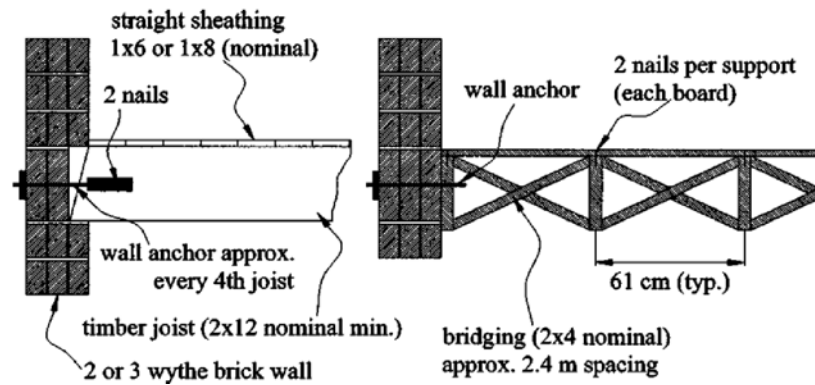
built up until the 1931 Napier earthquake were typically cavity walls, with no rubble fill, supposed to be linked with steel ties. Details of such buildings were described by Russel et al. [3] highlighting that the floor/roof diaphragm were just carried from the inner wythe of the cavity wall without any proper connection to the wall. Moreover, the gables in the upper end part of the walls were highly vulnerable because of the complete absence of effective connection with timber members.

All the above-mentioned solutions are mainly friction-based providing poor or absent linkage between the structural elements, causing weak load transmission. Sometimes, the reduced thickness at upper story led the timber floor system to simply bearing on the top of the lower wall with no embedment at all. Later solutions, studied by Lin and LaFave [15] and Peralta et al. [32], provided the use of iron straps to anchor timber joists or main beams to the masonry perpendicularly or in parallel (Figure 5(a)). Typical fish-tail iron anchors, where the strap was nailed to the timber joist and infixed into a pocket created in the masonry, in the other end, were shown by C3ias [33] and used to link the new timber headpiece in place of the degraded one (Figure 6(a)). Post-1755 Lisbon earthquake Pombalino buildings presented a much

125 complex detailing if compared to other constructions of the same period: the
timber floor joist would be connected to a top and bottom timber wall-plates
embedded in the masonry wall, through carefully done cross-lap joints and
the use of nails [34]. Metal straps were nailed to the timber joists and going
through or into the wall. Effective floor to wall connection could be obtained
130 making use of a 1 m long steel bar anchored to the masonry fixed diagonally
to the timber joists in contact with it. The bar was usually embedded in the
timber joists covered by the timber plank (Figure 5(b)).

It was common to cover large span using timber roof because of the
availability of the material and easy manufacturing. A good rule to improve
135 the mechanical characteristic of its connection to the masonry was to use
steel strap infixed inside the wall and connected to the bottom horizontal
member (Figure 5(c)). The steel strap could also be fixed to a bended bar
which was used also to improve the connection between the chain and the
rafter. A timber truss seated on a concrete padstone was shown in NZSEE,
140 part C8 [35].

The use of horizontal metal tie-bars is common in heritage buildings to
control the horizontal thrust exerted by vaults and arches or by wooden truss
roof, but it is also one of the simplest and widespread solutions on new and
retrofitted constructions ensuring the box-type behavior connecting the walls
145 at floor levels [36, 37, 38]. Metal tie-rods in the pre-industrial age were forged
steel bars in pseudo-circular or quadrangular sections anchored to the bearing
structure and inserted with minimum tensile action. Usually, anchor plates
or simply bars, forced into the eye-end of the tie rod with the help of metal
wedges, restrained the bar, acting on the outer surface of the wall.



(a)



(b)



(c)

Figure 5: Metal anchorages between timber beams and masonry wall: (a) connections between the parallel or perpendicular joist and the wall [32]; (b) diagonal bar embedded into the transversal joists [34] ; (c) roof-to-wall connection [33]

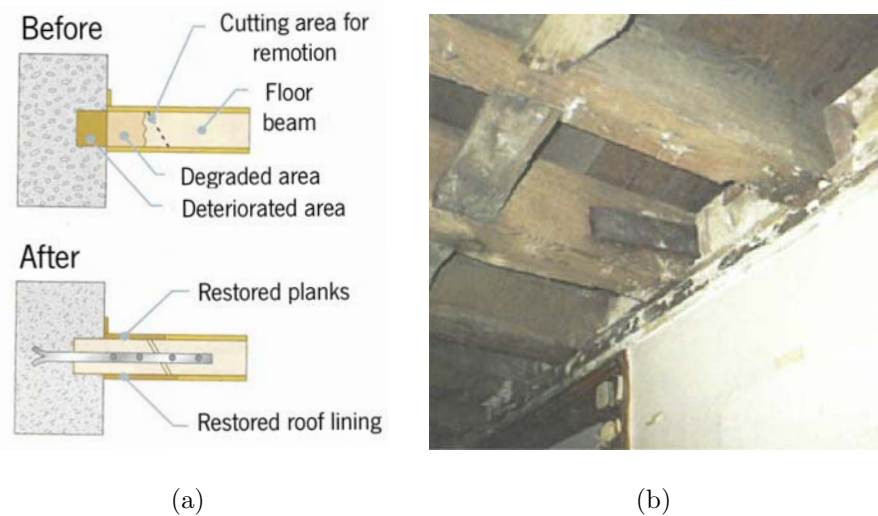


Figure 6: Timber to masonry anchors [33]: (a) Degrade timber joist headpiece replacement; (b) iron strip from the inside

150 A typical Mediterranean early 20th century floor, known as *jack arch system* (Figure 7), comprised iron beams supporting shallow brick masonry arches [6]. Here the metal beams were simply infixed in masonry slots and, sometimes, steel bars were present to reduce horizontal forces.

The arrival of reinforced concrete (RC) almost totally substituted ma-
155 sonry as main structural material for constructions, the latter being used

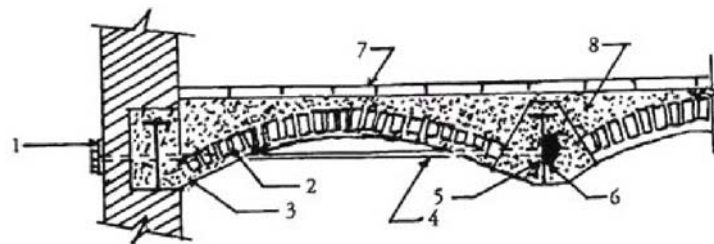


Figure 7: Jack arch roof: 1. M.S. plate 10 mm thick; 2. Brick arch; 3. Plaster; 4. M.S. Tie rod; 5. R.S.J.; 6. Cement concrete; 7. Flooring; 8. Lime concrete

merely for infill walls or non-structural components, such as partitions or aesthetic façades. The presence of RC bond-beams (tie beams) in newly masonry buildings at floor levels allowed the requested connection between the slab (usually made of RC) and the walls. This allowed the transfer of horizontal forces from the floor to the cross walls and improved the IP rigidity of horizontal floor diaphragms [7].

3. Seismic strengthening techniques

Among the general technical criteria for a successful strengthening of masonry buildings in seismic areas, there is proper geometrical layout of structural walls, their sufficient load bearing capacity, regularity and symmetry, adequate foundation capable to transfer loads to the soil. In addition, it is well recognized that the walls should be adequately tied and connected to each other, and the floors should be well anchored to the walls preventing OOP collapses [7]. The latter criterion provides the structural continuity between different components and improves the capability of the building to dissipate energy through the introduction of newer ductile elements, which can improve the seismic response. In addition, not only the joist slipping is avoided, but anchoring the diaphragm to the wall prevents hammering, typical in historical constructions where the vertical and horizontal elements natural frequencies are uncoupled. Moreover, the selected solutions should be simple and economic, experimentally characterized and should fulfill the basic requirements of restoration and conservation of cultural heritage (in the case of historical monuments) [39, 40, 41].

The use of metal tie-bars is a very simple and widespread intervention

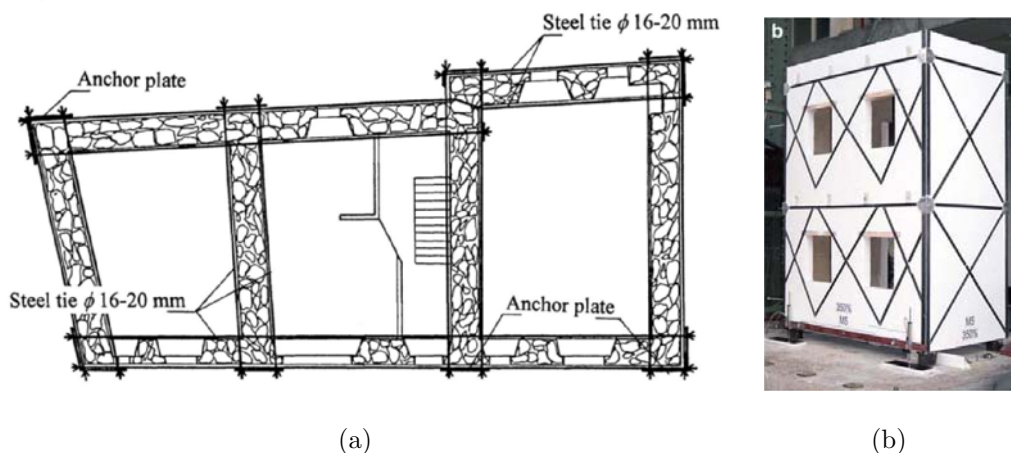


Figure 8: Tying URM walls for a seismic reinforcement: (a) Steel tie rods retrofitting [7]; (b) CFRP laminates retrofitting [44]

180 technique for the seismic upgrade of the URM building improving its structural integrity [36], useful both for semi-rigid and flexible floors such as vaults [42]. Typically, 16 – 22 mm diameter reinforcing steel slightly prestressed bars are placed at the floor levels on each side of the walls, anchored at the ends to steel anchor bars or plates (Figure 8(a)). However, as highlighted
185 by Tomažević [43], tying the walls is sometimes not sufficient for providing adequate structural integrity, and existing wooden floor structures should be strengthened, anchored and connected with masonry walls. Furthermore, the prestressed force must be conceived, designed and executed carefully: despite the masonry shear force increases with the compression level, the pre-tension
190 induces alterations in the equilibrium of the structure and modifies the tension state of the masonry [34]. Recent suggestions for tying the masonry walls include the possibility to replace the steel ties with reinforced polymer laminates (Figure 8(b)), significantly improving the resistance and the lateral

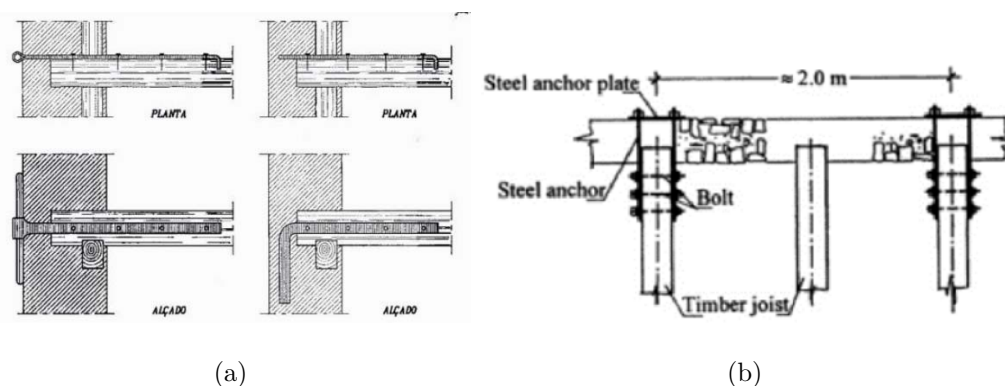


Figure 9: Wall-to-floor strengthening solutions: (a) wrought iron straps [33]; (b) double anchor system [7]

capacity, confining the building with vertically and horizontally placed strips
195 [43, 45, 46, 47]. For the latter solutions, precautions must be undertaken to
face the great difference in strength and deformability between traditional
masonry and innovative polymer, adopting compatible technologies for the
bond and the anchoring system.

One of the first solutions used to improve the connection of the joists to
200 the wall makes use of iron straps nailed to the timber joist and fixed into
or through the wall, anchoring on the exterior face with simple iron bar,
squared, star shaped, or circular plates [33], as shown in Figure 9(a). Fig-
ure 9(b) illustrates a strengthening solution proposed by Tomažević [7] with
double steel anchors aligned with the axis of the floor joist avoiding eccen-
205 tricity. Blaikie and Spurr [48] described some retrofitting practices adopted
after the 1931 Hawke’s Bay earthquake, in New Zealand. Many of them in-
clude the installation of wall-floor and wall-roof connections, mainly through
bolt anchors, used in conjunction with a steel bearing plate located on the
exterior of the building and a bolted connection on the timber diaphragm

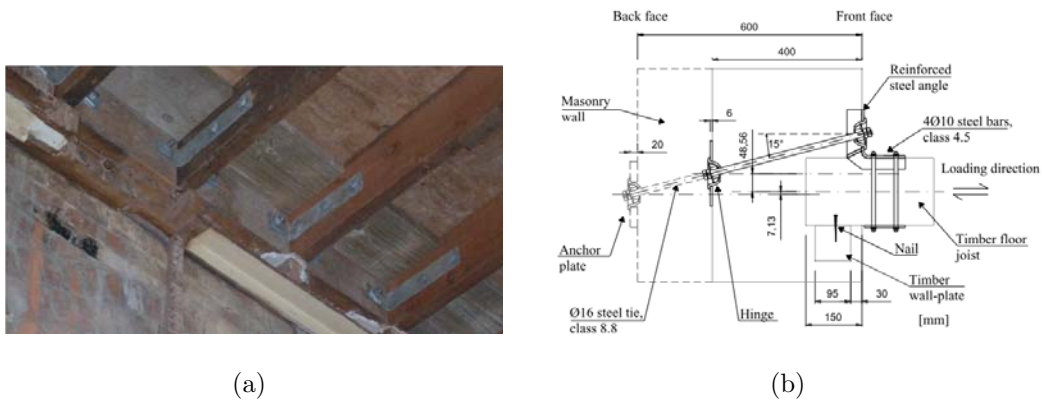


Figure 10: Wall-to-diaphragm bolt anchorages: (a) steel rods bolted to steel angles [48]; (b) innovative solution studied by Moreira [49]

joist, usually interconnecting a steel angle plate (Figure 10(a)). The bolted connections increase the shear resistance and improve the pull-out behavior of the solution. Newer interventions, studied by Moreira [49], contemplate hinges at both ends of the anchor allowing for a diagonal fixing of the tie directly from the top of the floor (Figure 10(b)), reducing the eccentricity and acquiring a more cost-effective solution. C3ias [33] proposed different strengthening solutions including injection anchors, where the grout injection controls the bond behavior between the steel tie and the surrounded masonry. Such a solution (Figure 11) requires access only from one side of the wall, facilitating the possible interventions on the faade or party walls. The connection to the timber joist is usually ensured by a bolted steel angle.

With respect to traditional steel anchors, dissipative passive systems may be integrated to improve the energy dissipation capability of the connections and may be based on the plasticity of the steel or on friction [50, 51].

Horizontal diaphragm action on floors or roofs is usually very poor in

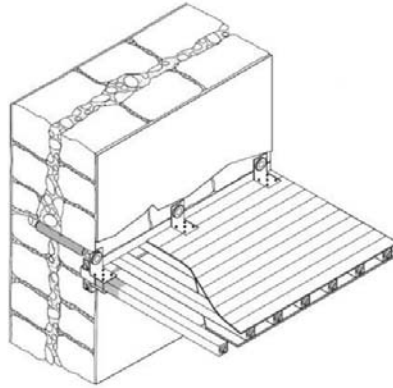


Figure 11: Strengthening solution proposed for Pombalino buildings [33]

225 existing URM buildings because of the high flexibility of the diaphragm in
comparison with the lateral resistant masonry walls. To increase their IP
stiffness, timber floors or roofs are often replaced by RC or precast slabs
with perimeter bond-beams similarly to the modern masonry constructions
where the RC ring beams provide the connection between the RC slab and
230 the wall so that the structural system performs as a monolithic unit during an
earthquake. If the bond-beams are not completely embedded in the masonry,
the slabs should be adequately anchored to the walls as shown by Tomažević
[7], see Figure 12. However, using RC slabs and ring beams should be con-
sidered as the last option as the increasing mass could lead to unfavourable
235 high seismic inertia forces, which can cause OOP modes and produce tor-
sional effects. Effective solutions can be achieved by overlapping a second
timber deck anchored to the existing one and providing proper connections
between the joists and the walls, with or without steel tie rods [52, 53].

The ring beam also increases the IP stiffness of the floor behaving as di-
240 aphragm chords [54] (Figure 13(a)) whose force can be computed considering

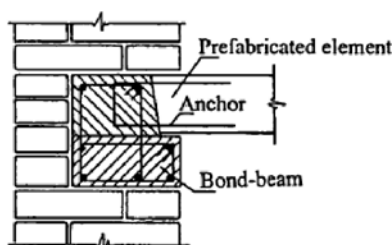


Figure 12: Connection between new RC bond beam and existing floor [7]

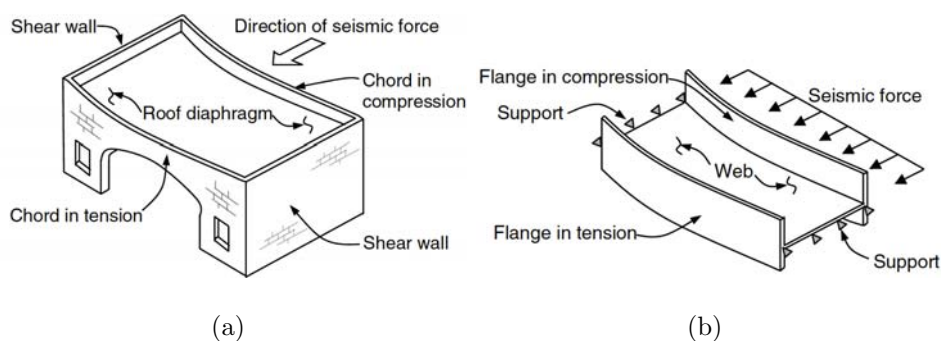


Figure 13: Chords and diaphragm corresponding to flanges and web [55]

the system (chords + diaphragm) as an I-beam under bending and shear in the direction of the earthquake (Figure 13(b)). Solutions shown in Figures 9, 10 and 11 are not usually able to perform as a diaphragm chord element, as they were designed mainly for the prevention of the shear and axial failure
 245 caused by IP and OOP forces, respectively. A chord strengthening solution is proposed by Hsiao and Tezcan [55] on the basis of FEMA [56]. The retrofitting strategy considers the strengthening of the shear and axial connections between wood diaphragms and URM bearing walls, following the concepts of maintaining the original figure of the historic building
 250 14).

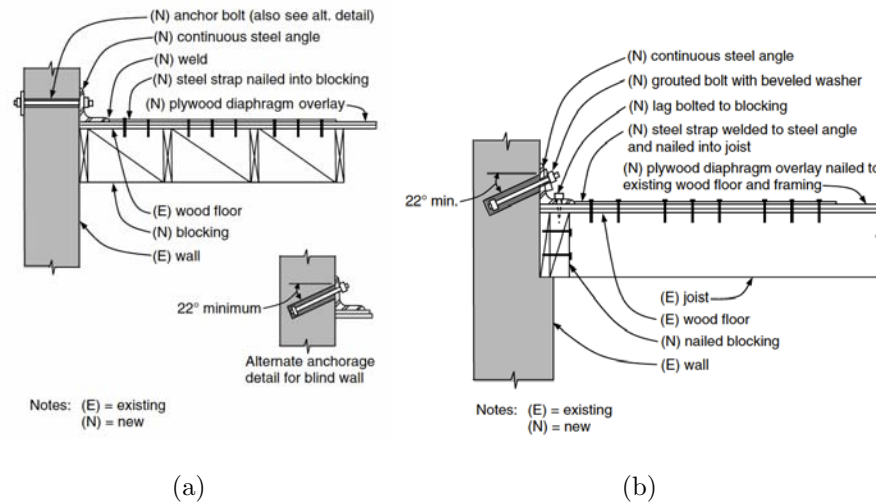


Figure 14: Chord strengthening connections [55]: (a) OOP bolted connection parallel to wall; (b) injected connection with joists perpendicular to the wall

A seismic strengthening solution applied on an old and damaged church timber roof structure is shown in Figure 15(a). The main aim was to increase the diaphragm behavior without destroying the original inner aspect of the church. The solution included the replacement of old timber with new timber beams and the insertion of a steel truss. Particular attention was given to the connection of beams to the perimeter walls (Figure 15(b)). A lattice girder lying along the bearing walls functions as lower tie beam.

Figure 16 shows a solution called “Perimetro Forte” (strong perimeter), where steel anchors are injected to the perimeter walls and bolted to the floor in special steel clamps [58]. Reinforcing bars are seated onto the clamps longitudinally to the walls, and a reinforced cementitious slab of minimum 6 cm is laid over and anchored to the existing floor through steel bolted connectors. Such solution can be easily installed over a steel or RC floor achieving a reasonable weight-to-stiffness ratio.

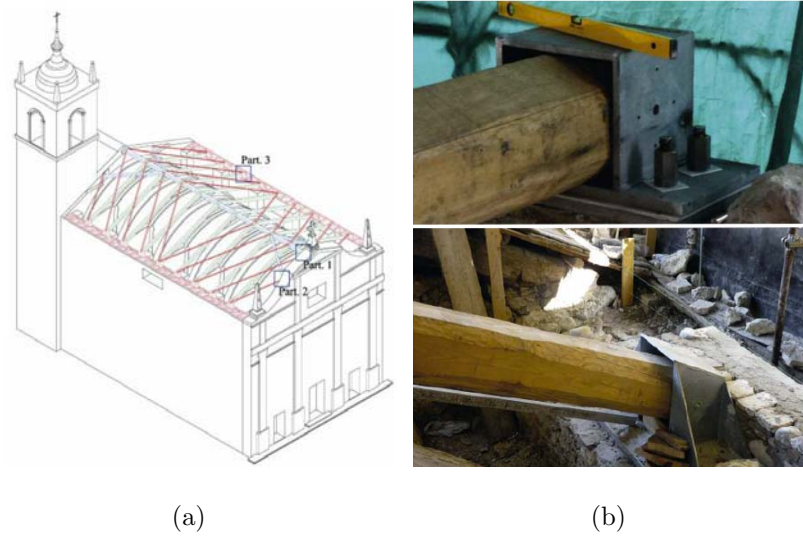


Figure 15: Seismic strengthening of a timber roof of a monumental building [57]: (a) Insertion of the truss system; (b) details of timber beam-to wall links



Figure 16: Strengthening solution recently developed by Leca Laterlite S.P.A. [58]

265 4. Experimental studies

Various experimental researches have been dedicated to the study of the global behavior of URM buildings, e.g. [59], or to characterize the solely timber floor/roof or IP/OOP walls, not paying much attention to the connection between vertical and horizontal components. Not only it is essential
270 to understand the shear and pull-out behavior of the connection, but it is also important to study the influence of the links on the overall global system. Moreover, even if retrofitting solutions comprise newly material, this is not true for the existing ones, which should be experimentally characterized.

4.1. Shake table tests

275 An experimental investigation on the dynamic behavior of reduced-scale URM buildings, including OOP walls with flexible diaphragms, was conducted by Costley and Abrams [60], where a steel bars framed system was developed to reflect a timber diaphragm flexible enough to have a natural frequency well separated ($\leq 1/3$) from that of the equivalent masonry structure with a rigid diaphragm. The steel beams were pinned at their ends to
280 the shear walls (IP walls) making use of washers and long steel box sections (Figure 17(a)), while the floor system was tied to the transverse walls through bolted threaded rods and anchor plates set in the inner and outer faces of the wall (Figure 17(b)). Test results showed that substantial strength and
285 deformation capacity still existed after the walls cracked during the experiments, indicating residual ductility within the structure. It was suggested the story drift to define different performance levels for URM buildings in performance-based design approaches.

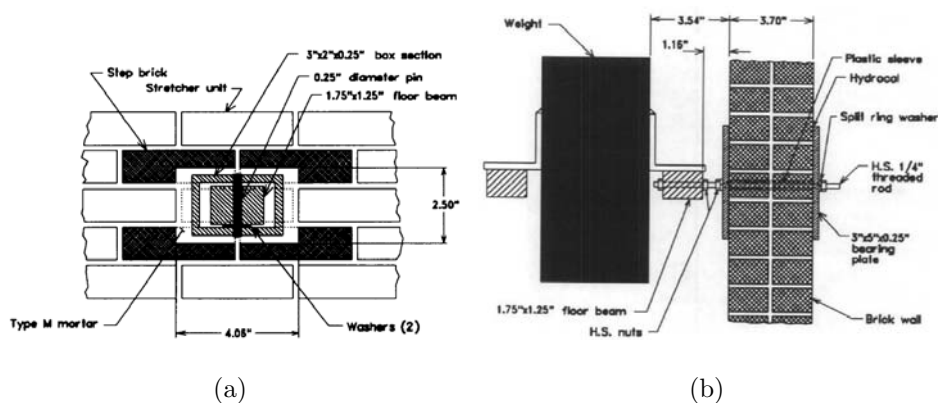


Figure 17: Shake table tests on reduced-scale URM building [60]: (a) front view of floor beam-to IP wall connection; (b) section of floor beam-to OOP wall connection

Bothara et al. [61] and Magenes et al. [59] performed shake table tests on half-scale and full-scale brick and stone masonry buildings, respectively, with conventional timber floor and roof, representative of existing URM dwellings. The incremental dynamic motions in both experimental campaigns indicated OOP response of portions of the walls perpendicular to the shaking direction, especially in top story zones, in agreement with observations on similar buildings stroke by real seismic events.

A shake table experimental program carried out at EUCENTRE aimed at studying the effectiveness of improved wall-to-floor and wall-to-roof connections through comparison with the unreinforced homologous prototype [62, 53]. Strengthening interventions included, among the others, the activation of steel tie-rods; the use of L-shaped steel beams at floor levels bolted to external steel plates; the adoption of steel through bars connected to the perimeter masonry walls and embedded into a lightweight RC slab cast above the existing floor; reinforced masonry ring beams employed at

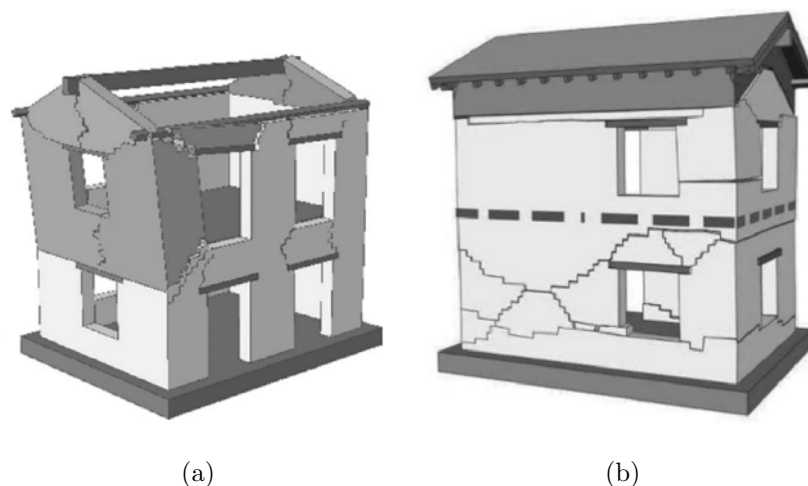


Figure 18: Shake table tests on full-scale URM buildings performed at EUCENTRE [62]:
(a) illustration of OOP mechanism on unreinforced building under nominal PGA of 0.4 g;
(b) illustration of shear failure modes on reinforced prototype under nominal PGA of 0.7 g

roof level. As shown in Figure 18, the applied strengthening techniques have
305 clearly improved the behavior of the buildings activating IP mechanism of
lateral resisting walls, typical of effective box-type buildings. However, from
global experiments, the detailed mechanical characteristic of the connections
was difficult to identify, therefore, local tests were needed. Further numerical
analyses were recommended to evaluate different scenarios through calibrated
310 models.

Three-leaf URM building models have been studied by Vintzileou et al.
[63] through half-scale shake table tests. After initial shake, the damaged
model was retrofitted (the timber floor was stiffened and accurately linked
to the wall, reinforced through hydraulic lime based grout injections) to
315 study the effect of widely applied intervention techniques. The wall-to-floor

reinforcement was made by connecting the stiffened deck to the masonry with steel plates and grouted bar bolted each other. The selected scheme of interventions was very efficient, and it made the model able to sustain strong input motion, relying on the injections to enhance the OOP resistance of the external masonry leaf and its mechanical properties.

Dazio [64] investigated the effect of different top and bottom boundary conditions ranging from fully fixed to simply supported, on the OOP behavior of the URM brick wall, applying different levels of initial axial load and varying the wall slenderness. Shaking table test results showed that boundary conditions could have larger effect on the lateral stability of a URM wall than its slenderness, and it was observed that simply supported walls could reach higher displacement capacity in comparison to the restrained one.

Simsir et al. [65] performed a shake table test set-up to investigate the response of OOP wall component as an integral part of the building system, where the floor mass was supported on the walls by means of a pin connection, while timber diaphragm (floor or roof) was simulated using different steel profiles connected to the walls through almost perfect pin connections. Mid-height collapses of the wall resulted only for low level of axial load and significant mass of the wall, corresponding to upper floors in real buildings. The authors measured peak acceleration values at the top and mid-height of the wall up to 4.5 times the peak base accelerations. In this study, the influence of adjacent IP walls and the deformability of the diaphragm-to-wall connection are neglected.

4.2. Wall-diaphragm anchor tests

340 Accurate accounting for the connections into the numerical model requires their mechanical static and dynamic characterization and this is not usually obtained or addressed after shake table tests on building or local wall scale. Detail-scale experiments (pull-out or shear tests) can be useful to address this problem aiming at force-displacement curves of existing or improved/repaired
345 prototype connections.

In New Zealand the dynamic behavior of two types of URM walls anchors was performed by Jacks and Beattie [66]. Both through-bolt and epoxied-in anchors were investigated through pull-out tests. From the dynamic and static tests of the through-bolt anchors, no pull-out failure of the URM panel
350 was observed even under high load levels (96 kN of dynamic and 200 kN of static loads) indicating the effectiveness of the practice for anchoring URM walls. From the static tests of the epoxied-in anchors, the anchorage failure was related to the crack occurred across the full width of the test panels.

Lin and LaFave [15] conducted static monotonic, as well as static and
355 dynamic cyclic tests on two different types of typical brick wall to wood diaphragm connection specimens, with and without nailed strap anchors (Figure 19(a)). The authors considered the contribution of friction and of strap anchor nails loaded in shear separately and together. A value of friction coefficient was suggested as the average measured, well compared with his-
360 torically published values. The behavior under dynamic cyclic loading was more brittle than behavior under monotonic or quasi-static cyclic loading, for the case of connections with nails and friction (Figure 19(b)). Results obtained from those tests can be used to calibrate nonlinear finite element

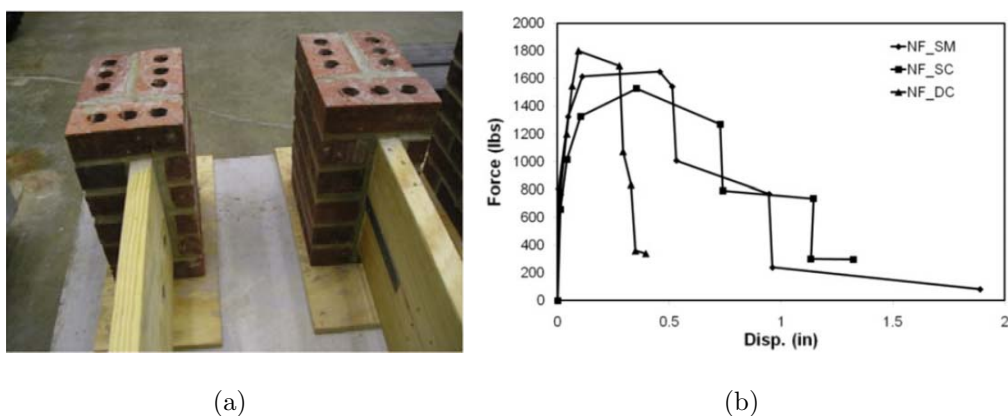


Figure 19: Experimental campaign performed by Lin and LaFave [15]: (a) test specimens of wall-diaphragm connections; (b) comparison between static monotonic (SM), quasi-static cyclic (SC) and dynamic cyclic (DC) average curves, considering the contribution of friction and nailed straps (NF)

models of these types of connections.

365 Campbell et al. [67] studied typical existing wall-diaphragm anchor plate connections, extracted post-earthquake by sorting through the demolition debris from URM buildings damaged in the 2010 M7.1 Darfield (Christchurch) earthquake. The connections consisted of long, roughly 25 mm diameter rod, with a rectangular or circular steel plate (about 5 mm thick) attached to the wall end that is about 50 mm wide \times 450 mm long and fastened to the rod and positioned either inside the brick wall or in the center of a masonry pier or wall (Figure 20(a)). The authors established at least seven plausible failure modes associated with the wall-diaphragm anchor plate connections that were axially loaded to rupture (Figure 20(b)) (A. Punching shear failure of masonry; B. Yield or rupture of connector rod; C. Rupture at joint between masonry; B. Yield or rupture of connector rod; C. Rupture at joint between connector rod and joist plate; D. Splitting of joist or stringer; E. Failure of fix-

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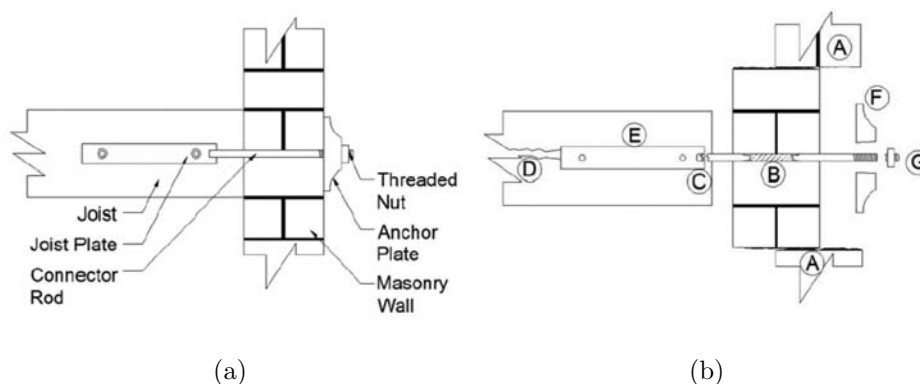


Figure 20: Typical New Zealand wall-diaphragm anchor plate connections [67]: (a) connection assembly; (b) location of failure modes

ing at joist plate; F. Splitting or fracture of anchor plate; G. Yield or rupture at threaded nut), based on 2010/11 Canterbury earthquake sequence damage observations and on a small amount of anchor samples tested (only six anchor plate connections). A lower 5th percentile characteristic ultimate tensile strength of 63.9 *kN* was suggested for connections having anchor plates of approximately 200 *mm* diameter and connector rods of approximately 17 *mm* diameter. For degraded connections a 5th percentile characteristic ultimate stress of 269.5 *MPa* was recommended, but further testing to be undertaken was suggested to increase the test dataset.

Karim et al. [68] studied experimentally the wall-diaphragm connection focusing on the failure of the timber joist bolted connection. They recommended a set of design equations to assess the strength of the connection related to the minimum strength value that will govern the capacity of the whole connection. This procedure could quantitatively and quickly assess the performance level of wall-diaphragm connection, which could be reinforced,

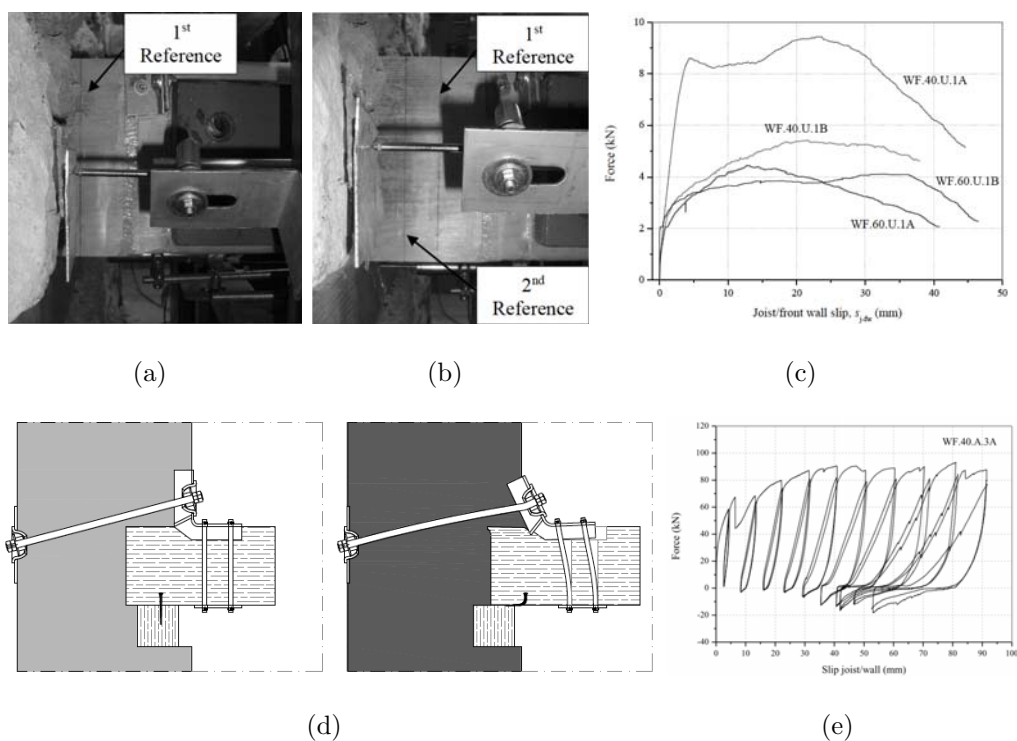


Figure 21: Experimental campaign performed by Moreira [49]: (a) and (b) sliding of the timber joist caused by failing of nails before and after; (c) monotonic force-displacement curve of unstrengthened specimen; (d) deformation of pulled-out strengthened specimen; (e) cyclic curves for strengthened tests

if necessary.

Moreira [49] carried out an experimental campaign on both unstrengthened and strengthened wall-to-floor connections typical of late 19th century in Lisbon. The strengthened connection relied on anchoring the timber floor to the masonry wall through the use of steel tie-rods with anchor plates. Hinges at both ends reduced the bending force induced by the timber beam to the anchor, diagonally infixed into the masonry (Figure 10(b), Section 3). Both quasi-static monotonic and cyclic pull-out tests were carried out. In

400 all unstrengthened specimens monotonic tests were performed and the two
nails connecting the joists to the wall-plate failed causing clear sliding (Fig-
ure 21(a), (b)) and the capacity was highly influenced by the rotation of the
joist. For the strengthened connections, yielding of the steel angle, crushing
and shearing of the timber was observed (Figure 21(d)), and the tensile ca-
405 pacity of the connections was approximately 19 times greater than the one
of the unstrengthened ones and the ultimate displacements were much larger
by comparison. However, a decrease in ductility could be observed due to
increase of the elastic limit and cyclic tests degradation (Figure 21(c),(e)). A
retrofit design proposal was recommended for design at component level, and
410 future works are necessary to establish hysteretic rules useful to implement
the connection as an element in numerical models of whole buildings.

Ismail [69] tested 40 grouted/epoxied anchors for pull-out-capacity (POC)
of which 30 were installed in salvaged heritage material assemblages and 10
in-situ at a heritage URM building. Two types of anchors were consid-
415 ered in this study: 1) helically shaped stainless steel 10mm anchors; and
2) self-cutting stainless steel anchors with 12mm diameter. The author in-
vestigated the influence of embedment length, installation quality, anchor
location, condition of masonry and condition of substrate materials on an-
chor performance. The highest probability of failure during an earthquake is
420 associated to the failure of anchor-diaphragm connection and thus limits the
achievable POC of the wall to diaphragm anchorage system. However, this
type of failure was observed to be reasonably ductile.

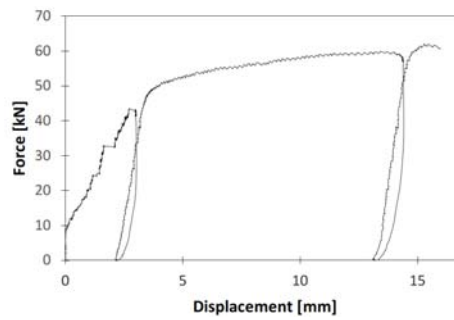
A consistent experimental campaign (almost 400 specimens) was under-
taken by Dizhur et al. [70] on adhesive anchor connections between unre-

425 infomed clay brick URM walls and roof or floor diaphragms. No conical
masonry failure surface was observed in any of the tests. Cementitious grout
was a suitable anchor adhesive and 16mm anchor rod diameter was consid-
ered to be the optimum rod size for the POC of the adhesive anchor. Low
overburden weight negatively influences the POC and accurate installation
430 process is critical to achieving adequate POC. The authors proposed a design
equation of the POC of adhesive anchors considering relevant parameters and
based on the concrete breakout strength of an anchor in tension as proposed
by Fuchs et al. [71]. No strength reduction factor is required, except when
ultra-weak masonry, or particularly low overburden loads, are encountered.

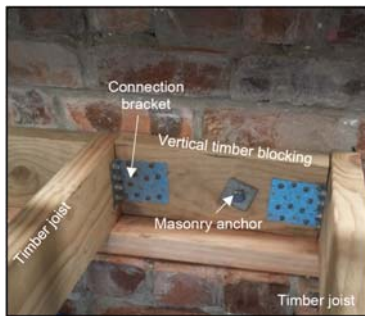
435 Recently, Dizhur et al. [72] undertook experimental tests on original
vintage wall-to-diaphragm plate anchor connections installed in an URM
buildings, built in 1913. Two different types of anchorages between the
masonry anchor and the timber diaphragm were studied: (i) metal connector
directly fixed to the timber joist through a bolted joist plate (Figure 22(a)),
440 and (ii) connections making use of timber blocking (vertical or horizontal
timber elements interconnected between the joists) (Figure 22(c)). Failure
modes and force-displacement curves were provided for the plate anchor,
while ultimate capacity was established for the timber blocking systems with
different configurations (Figure 22(b), 22(d)). Among the main outcomes
445 from the experimental campaign results, authors highlighted that timber joist
splitting was the most commonly observed failure mode for joist plate anchor
connections, while positioning the timber blocking horizontally allowed the
highest performance to be achieved. Finally, when a small washer was used,
timber bearing failure was observed in timber blocking systems.



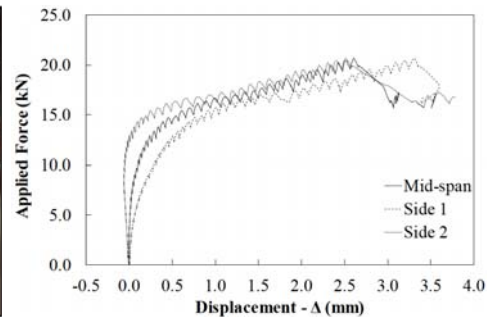
(a)



(b)



(c)



(d)

Figure 22: Wall-to-diaphragm experimental tests by Dizhur et al. [72]: (a) joist plate connection; (b) joist plate connection exhibiting anchor rod rupture; (c) timber blocking connection; (d) vertical timber blocking connection exhibiting screw withdrawal

450 5. Assessment approaches and code standards

Basically, an existing URM building can be seismically assessed comparing its capacity (C) to the seismic demand (D) through the following inequality:

$$C > D. \quad (1)$$

Different definitions of both the capacity and the demand lead to several
455 approaches, which can be mainly grouped into two big families: force-based-approaches (FBA), and displacement-based-approaches (DBA) [73, 74, 75]. Energy-based-approaches (EBA) were also recently proposed by Sorrentino et al. [73] and Giresini [76] as a promising tool, but still less conservative procedure for assess the seismic response of local OOP masonry walls, if
460 compared to current code verifications. A considerable number of modelling techniques have been developed for the seismic analysis and assessment of URM constructions in a building scale [e.g. 77, 78] and local scale (i.e. OOP mechanism) [e.g. 73, 74], but only few included the behavior of the connections between vertical and horizontal members interfaces [e.g. 79, 65, 80].
465 Moreover, authors often considered the connection assuming simplified hypothesis (i.e. perfect hinges or linear elastic springs), neglecting the cyclic nonlinear behavior, typical for many typologies of links and reducing the number of the degrees of complexity of the dynamic problem.

The most common modelling techniques comprise rigid block approaches,
470 numerical finite elements models (FEM), and distinct (or discrete) elements models (DEM). For rigid block approaches, the capacity can be computed through static or kinematic analysis, while the demand is usually derived from the displacement response spectrum, based on the period of the equiv-

alent SDOF system. When NLTH analysis is used, however, the demand
475 can be related to the PGA (or other Intensity Measures, IMs) of the ground
motion leading to a certain level of damage or collapse. Static pushover and
dynamic analysis with time-step integration under artificial or real accelero-
grams are common tools when FEM or DEM techniques are adopted [81, 82].
For the former analysis the capacity can be defined as the peak of the re-
480 sponse curve, while for the latter the maximum response value in terms of
acceleration can be considered. It must be noted that FBA are preferred
by practitioners as simple and direct methods, in spite of high conservative
results, while DBA appears to be adequate for cracked walls falling into the
nonlinear range [73].

485 *5.1. Global scale approaches*

Priestly [83] proposed a DBA for the assessment of URM buildings, for
which the author identified a seismic-load path: the earthquake excitation at
the footing was filtered by the IP response of shear walls and floor diaphragms
and reached OOP walls (Figure 23). The acceleration capacity was based on
490 an "equal energy" criterion, while the acceleration demand was based on
the height of the building, ratio of the period of the floor and shear walls.
The analyses produced expected results, but assumptions were made, such
as that the walls were properly connected to floor and roof (even if generally
this was not the case).

495 The influence of the floor on the seismic response of URM structures was
studied by Tena-Colunga and Abrams [84] through discrete MDOF dynamic
models, based on linear masonry behavior and flexible and linear diaphragms
(elastic springs). Mainly two structures served as case-study (Figure 24), and

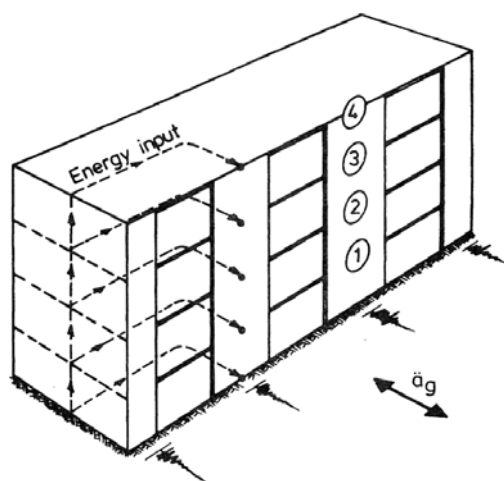


Figure 23: Seismic load path in URM according to Priestly [83]

results demonstrated that, in some cases, diaphragm and shear-wall accel-
500 erations can increase with the flexibility of the diaphragm. Moreover, tor-
sional forces could reduce considerably as diaphragm flexibility increased.
Finally, the authors stated that the period of vibration of systems with flex-
ible diaphragms could be underestimated if calculated through approximate
expressions prescribed in current seismic codes.

505 An unconventional procedure for the simulation of the OOP dynamic be-
havior of URM buildings, considering the stiffness of the roof, was performed
by Costa [85], where the local mechanisms were modelled as kinematic chains
of masonry portions (normally assumed as infinitely rigid bodies) whose non-
linear behavior is concentrated at the contact regions (Figure 25). The au-
510 thor considered a Coulomb-type sliding friction law and energy dissipation
through the restitution coefficient at the impacts. Springs simulated the IP
stiffness of the horizontal diaphragm. Lumped mass located at the ridge and

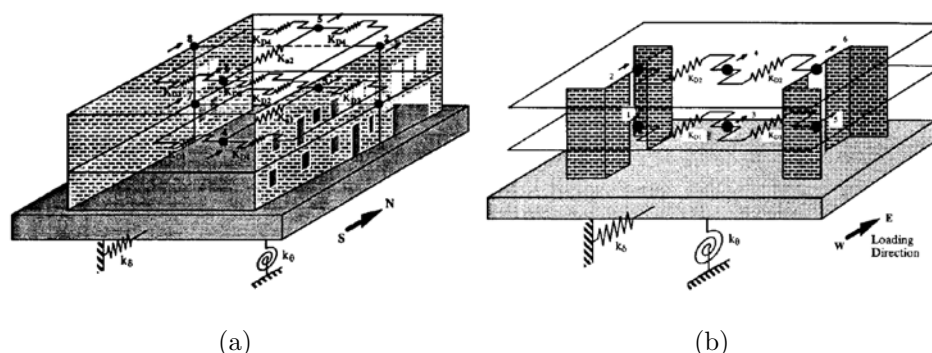


Figure 24: Modelling of buildings through discrete dynamic models by Tena-Colunga and Abrams [84]: (a) firehouse; (b) office building

top spreader beams represented the roof mass. Similarities of the method can be found in the modelling technique used by Oliveira et al. [86] based on DEM method for the interpretation of the post-seismic damage in URM structures. This methodology was adopted to simulate the experimental behavior of a full scale URM unstrengthened building tested at EUCENTRE (described in Section 4), for which elastic springs were placed at the top of each roof pitch (with a stiffness of $1.49MN/m$ defined according to results obtained by Brignola et al. [1]). The adopted numerical approach allowed simulating the experimental failure mode with sufficient accuracy, but further studies were suggested to obtain an adequate and suitable method for the simulation of the OOP behavior of masonry walls. As highlighted by Ferreira et al. [74], the main advantages of such methodology rely on the time efficiency and the mechanical parameters (using only geometric parameters, mass, friction coefficient and restitution coefficient), while the predefinition of the formed local mechanisms need further analysis or judgement to define a realistic overturning mechanism, setting the limits and the drawback of the

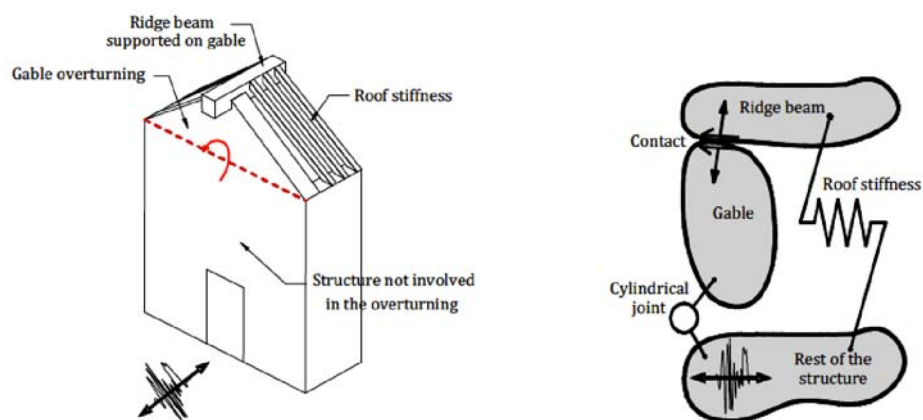


Figure 25: Schematic representation of the equivalent Multi-body dynamics-based approach [85]

method.

530 Ortega et al. [87] developed a parametric study through a detailed FEM and pushover analyses aiming at the evaluation and quantification of the influence of the type of diaphragm in the seismic behavior of vernacular buildings (Figure 26(a)). Among the different parameters that have been analysed, the connection between the timber beams and the walls, and the
535 connection between the timber deck and the masonry were studied, considering different embedment and connection conditions. Results clearly showed the important increase in seismic resistance of the building if proper connections were adopted (i.e. timber beams fully embedded inside the whole thickness of the wall, and cross-board sheathing infixed in the walls), see
540 Figure 26(b), avoiding the prematurely cone failure on the masonry and a better redistribution of the forces to the IP walls. The adoption of rigid diaphragms with low levels of connections between the timber beams and the walls resulted to be ineffective. Besides FE approaches is rarely adopted

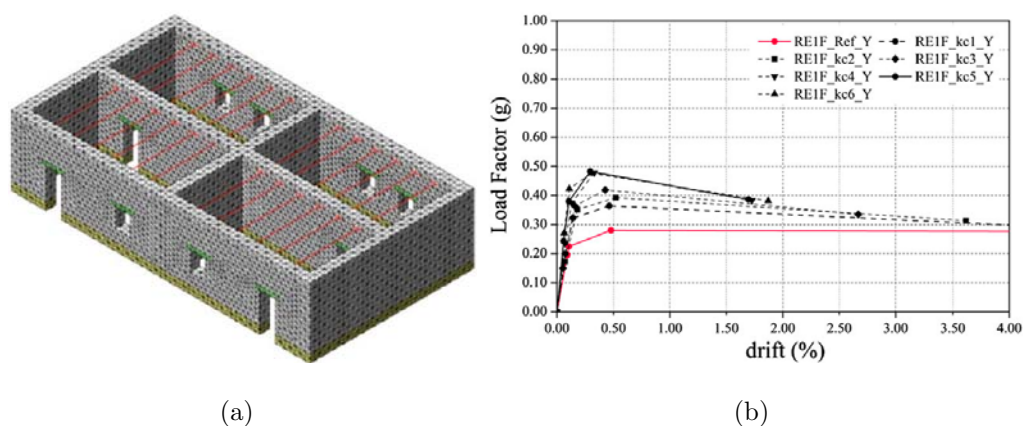


Figure 26: Detailed FEM developed by Ortega et al. [87] in order to evaluate the influence of beam-to-wall connection: (a) one of the three case study buildings; (b) positive influence of proper beams-to-walls connections

from practitioners for masonry constructions, several examples demonstrate
 its capabilities also for large complex structures [88]. Nonlinear mechanical
 545 characteristics of the masonry and flexibility of the floor are few of the im-
 portant parameters playing a crucial role on the vulnerability index, usually
 obtained through pushover analyses [89, 90]. Because of the local behav-
 ior of structural elements, frequently observed in historical buildings due to
 550 the poor connections between each other, timber roofs or floors are often
 neglected as structural elements (included only as additional masses on top
 of the walls) [91]. When a good quality of the connection may be justified,
 the diaphragm timber beams can be included in the global 3D model, but
 the connection is usually perfect (acceptable only for properly retrofitted
 555 connections or new buildings).

Senaldi et al. [53] simulated the experimental results by using an equiv-
 alent frame approach implemented in the program TREMURI [92]. The

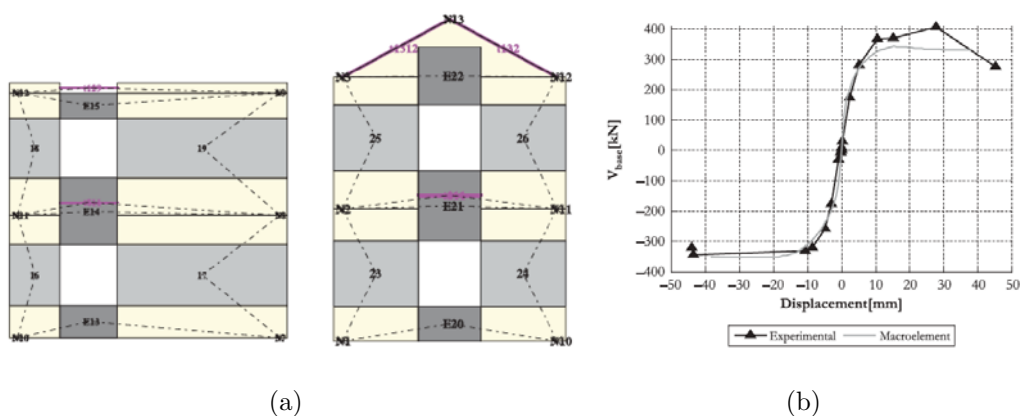


Figure 27: Experimental model implemented in TREMURI program [53]: (a) geometry of the model; (b) experimental vs. numerical pushover curves

macro-element model (Figure 27(a)) allowed for the shear-sliding damage evolution and rocking mechanism, with toe-crushing effect. Nonlinear beam
 560 elements have been introduced to account for the presence of the RC ring beam and of the wall to diaphragm connections at floor level. Pushover curves were compared to the experimental resistance curve, showing well similarities (Figure 27(b)). The equivalent frame modeling may be considered as a simplified approach, preferred by professional, for a first conservative approach for the seismic assessment of existing URM buildings. This approach,
 565 however, is not recommended for buildings with irregular opening patterns, high flexible floors and poor wall-to-floor and wall-to-roof connections [93].

A simplified modeling approach was presented based on three dimensional discrete elements as an extension of a plane macro-element [94]. The
 570 proposed approach lies in between the more complex, computationally cumbersome and more detailed nonlinear FE method and the classical limit approach and shows the capability of simulating the nonlinear response of mon-

umental structures. The dynamic behavior of the structural connections is not usually taken into account.

575 *5.2. Local scale approaches*

Cross et al. [95] provided a useful tool for the design of seismic retrofit details developing a FEM that accounts for friction and impact behavior at the diaphragm-to-wall interface. The effectiveness of the model was demonstrated through comparison to some MDOF systems and applying this approach to a historic brick building shaken during the Loma Prieta 1989 earthquake. 580

The influence of strengthening techniques (e.g., insertion of tie-rods, of rigid diaphragms, etc.) was studied by Giuffrè [96] through a rigid-body limit analysis on a comparative basis. The safety check was given between the maximum horizontal load multipliers causing OOP mechanisms, and an expected PGA normalized to gravity. A response spectrum for the bilinear non-dissipative model was proposed allowing to evaluate the maximum displacement expected as function of the PGA and several parameters concerning the static strength of the model to horizontal forces. On the same 585 bases, D'Ayala and Speranza [97] developed a simple but realistic mechanical model, based on post-earthquake damage observations, capable of handling a large number of buildings and performing statistical analysis. The model allowed to consider the influence of seismic strengthening (insertion of ring beams and ties), proving their efficiency, especially for slender vulnerable buildings. Several load factor equations were given for each overturning 590 mechanisms (e.g. Figure 28) without and with the presence of strengthening devices (for which the arch effect was taken into account), and implemented

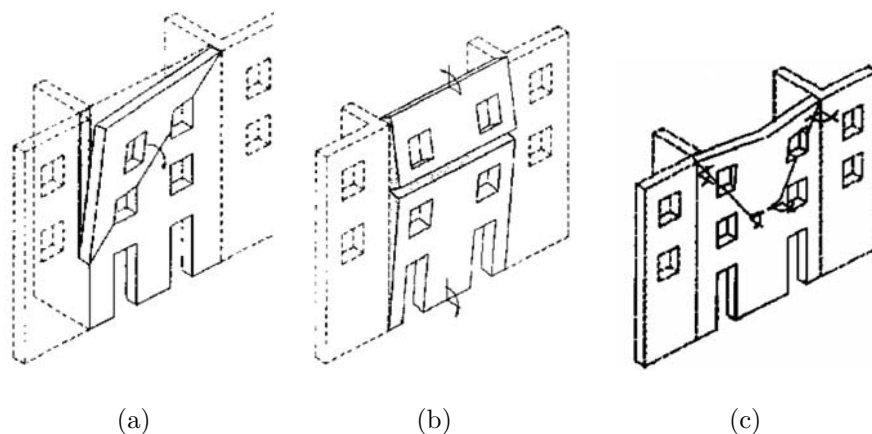


Figure 28: Mechanisms for overturning failures studied by D'Ayala and Speranza [97]

in the FaMIVE (Failure Mechanism Identification and Vulnerability Evaluation) procedure for the online evaluation of a façade or building seismic
600 vulnerability.

Lawrence and Marshall [98] proposed a new approach for the design of masonry OOP panels based on the knowledge of the likely crack lines, through application of virtual work principles. A design equation was given for different boundary conditions (the top of the masonry panel was considered either
605 free or simply supported) and results were compared with several tests available in literature providing closer estimates with low variability. The method also allowed for the presence of door and window openings.

A two-degree-of-freedom model was introduced by Simsir et al. [65] as an assemblage of two rigid bars interconnected by rotational springs, simulating the cracked wall tested experimentally. The connection on the top was
610 modelled as elastic spring representing the stiffness of the IP wall and the diaphragm connected in series (Figure 29(b)). The authors also compared a SDOF (Figure 29(a)) to a MDOF (Figure 29(c)) model to study the influ-

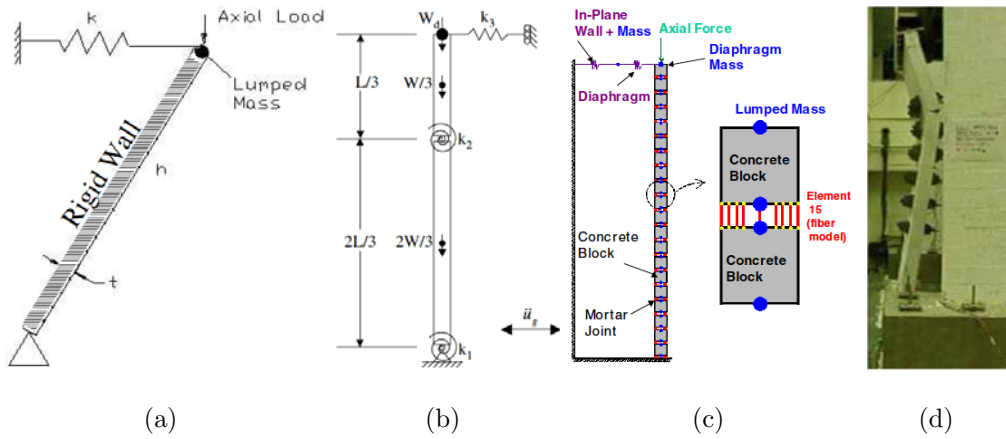


Figure 29: Numerical and experimental studies carried out by Simsir et al. [65]: (a) SDOF model; (b) 2DOF model; (c) MDOF model; (d) damaged specimen after tests

ence of OOP bending caused by cracks at the bed joints. The SDOF model
 615 seemed to be inaccurate if compared to MDOF, as could not capture higher
 mode response observed experimentally (i.e. it could not model the forma-
 tion of the mid-height wall crack as shown in the experimental test result in
 Figure 29(d)), while the two-degree-of-freedom model revealed to be a simple
 and efficient tool for the evaluation of the seismic vulnerability of OOP walls
 620 showing sufficient experimental validation.

When a monolithic behavior is a reliable assumption for the walls in OOP,
 they can be regarded as rigid blocks, and possible reliable approaches can be
 rocking and kinematic dynamics analysis simulating the local behavior.

Basis on rocking blocks can be found on the pioneering work of Housner
 625 [99], who considered the motion of a SDOF free rigid block, a rigid prism
 with longitudinal rectangular cross section, rocking about the two corners O
 and O' (Figure 30(a)) subjected to free vibrations, constant and sinusoidal
 acceleration, and earthquake motion. Important contribution was given by

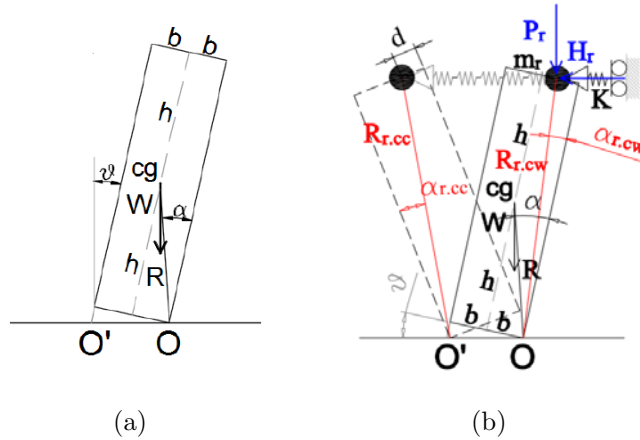


Figure 30: Rocking rigid block: (a) Housner free block model; (b) restrained rocking block [101]

Aslam et al. [100] who considered the rocking vibrations of the free rigid
 630 block under ground motions through a computer program. Considering the
 rotation θ (> 0 if counter-clockwise) as a Lagrangian coordinate, the equation
 of motion takes the form: The energy dissipation is accounted for through
 the “restitution coefficient” $e_H = 1 - \frac{3}{2} \sin^2(\alpha)$, where α is the arctangent of
 the thickness to height ratio and is valid for slender and rectangular blocks.
 635 Giresini et al. [101, 102] modified the equation of motion to consider the
 influence of possible roof mass, thrust and horizontal restraint through sin-
 gle or bed spring with stiffness K , or K' , respectively, simulating a tie-rod,
 a flexible diaphragm or transverse walls (Figure 30(b)). Comparison with
 experimental results demonstrated the accuracy of the method, contrary to
 640 the non-linear kinematic approach suggested by the Italian code for OOP
 mechanism. An improvement of the approach was performed by AlShawa
 et al. [103] who considered the plasticity of tie rod. The model is used to

investigate the response to variations of wall geometry and tie characteristics. The most relevant parameter was the steel strength, whereas other
645 characteristics play minor roles allowing to recommend reduced values for pre-tensioning forces.

In cases when the degree of freedom cannot be assimilated to one (flexural bending failure modes), a MDOF dynamics could be a more realistic representation. Studies on 2DOF system consisting of two blocks, one placed on
650 the top of the other, and free to rock without sliding (Figure 31(a)), were developed by Psycharis [104], who derived the equations of motion for each mode of vibration and analysed the redistribution of kinetic energy of the system in the blocks. Kounadis and Papadopoulos [105] extended the study to a 3DOF system (Figure 31(b)), neglecting sliding, aiming at assessing the
655 overturning instability with or without impact either between blocks or between the lower block and the ground. All possible configuration patterns that may lead to rocking instability through an escaped motion were properly discussed. None of the latter studies took into consideration lateral boundary conditions.

660 Landi et al. [80] simulated the behavior of slender façades of URM buildings with flexible diaphragms subjected to OOP bending by means of a 2DOF model assembled as two rigid bodies connected by an intermediate hinge and restrained at the top by a spring (Figure 32(a)), while the damping has been modelled through the introduction of the coefficient of restitution. The calibrated model was used for parametric analyses (Figure 32(b)) and compared
665 to simple models confirming its effectiveness. NLTH analyses of a set of walls subjected to recorded ground motions have been performed in order to com-

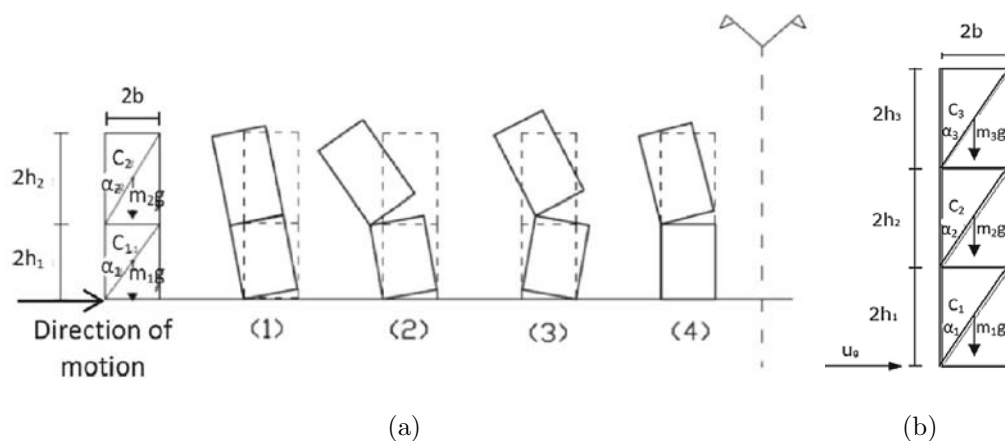


Figure 31: MDOF configurations of rigid blocks [105] : (a) 2DOF system with 4+4 possible configurations; (b) 3DOF system

pare the response of the simply supported wall with that of the restrained wall. Four different geometric conditions described by four corresponding sets of equations were studied, and the results compared to a simple 1DOF model, showing that with the increase of the spring stiffness, the response of the 2DOF model tended to reproduce well the response of the 1DOF model. On the contrary, with the decrease of the spring stiffness, the top displacement increased and tended to be larger than the mid-height displacement.

675

A general settlement of the rocking procedure for the vulnerability of the OOP masonry wall, is still far from the professional practise. Currently for masonry structures, standard response spectra are used.

A simple design method for assessing the maximum rocking displacement, using equivalent elastic characteristics and equivalent response spectrum was presented by Priestley et al. [106] and compared to results from simulated seismic excitation of the model using an electro-hydraulic shake-table. Do-

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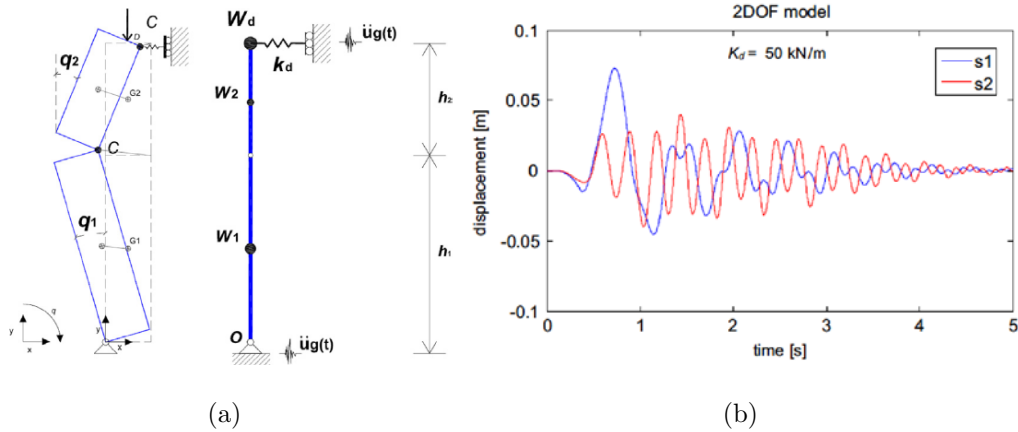


Figure 32: 2DOF model analysis of the wall in OOP bending [80]: (a) illustration of the model; (b) mid-height (s_1) and top (s_2) displacements results for $k_d = 50 \text{ kN/m}$

herty et al. [107] presented a simplified linearized DB procedure based on a trilinear force-displacement relationship of the URM walls (Figure 33(a)).
 685 The authors considered both the free and simply supported condition on top (Figure 33(b)) and compared the procedure with NLTH analyses results and with quasi-static analysis showing that less scatter could be reached through DB approach rather than FB procedures. The selection of a proper substitute structure was recommended for each case and the secant stiffness was defined, which was found to correlate well with the predominant natural
 690 frequencies observed during experimental testing.

However, distinct differences were noted between rocking spectrum and the classic design response spectrum. Makris and Konstantinidis [108] concluded that the rocking structures cannot be replaced by “equivalent” SDOF
 695 oscillators.

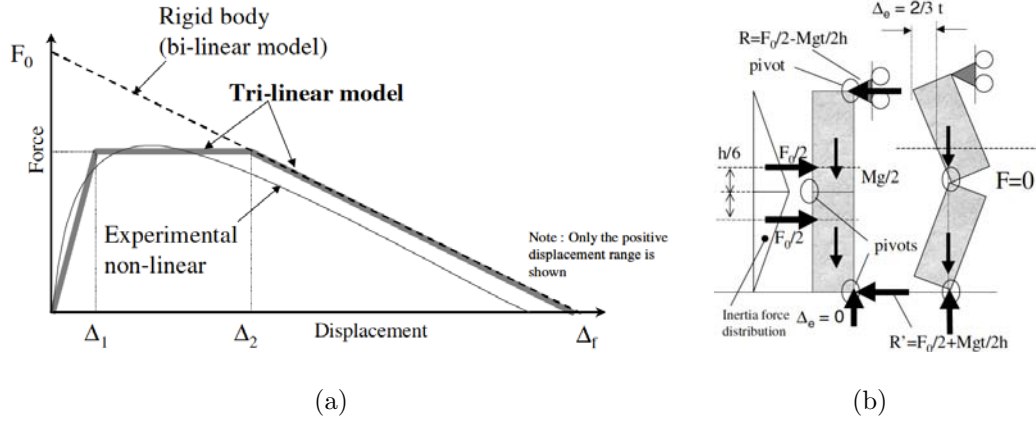


Figure 33: Linearized DB approached proposed by Doherty et al. [107]: (a) force-displacement relationship of URM walls; (b) simply supported masonry wall on top

5.3. Code Standards

The seismic capacity of a local mechanism (i.e. OOP wall failure) can be assessed in terms of ultimate displacement through a non-linear kinematic analysis, considering forces given by possible restrains (steel tie rods, beams, etc.) as recommended in the Italian code [11, 12], if the monolithic behavior of the block is reasonably assumed. This approach is based on the equilibrium limit analysis and allows for the calculation of the horizontal force which the structure is able to resist, during the evolution of the mechanism. This force-displacement curve can be transformed into the equivalent SDOF system capacity curve (Figure 34(a)) where the Ultimate Limit State (ULS) capacity, d_u^* is defined in terms of spectral displacement d^* and compared to the spectral seismic displacement demand, $S_{De}(T_s)$ based on the secant period, obtained from the capacity curve:

$$d_u^* \geq S_{De}(T_s). \quad (2)$$

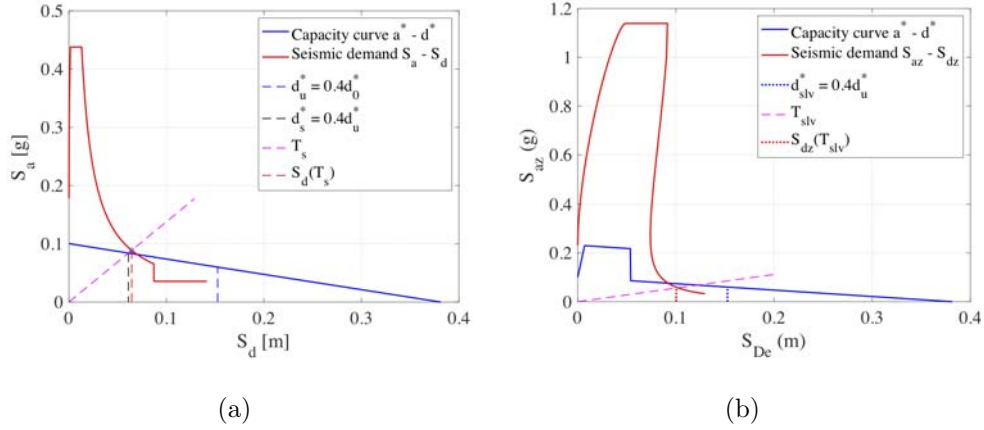


Figure 34: ULS assessment through the kinematic approach of a masonry wall restrained by a elastic-perfectly plastic spring on top, according to Italian code: (a) NTC2008 [12]; (b) NTC2018 [14]

The Italian code standard also allows for the Damage Limit State (DLS)
 710 assessment through the following:

$$a_0^* \geq a_g S, \quad (3)$$

where S accounts for the type of soil and geographic conditions, while a_g is the reference acceleration for the specific geographic coordinates, and a_0^* is the spectral acceleration that activates the mechanism (Figure 34(a)). Moreover the normative was found to be too conservative for some case studies
 715 [109]. On the bases of recent research works [110, 111, 112] the new Italian standard [14] allows to consider the amplified floor Response Spectrum in case of local mechanisms placed at higher levels of the structure. The seismic demand is defined in terms of overdamped elastic Acceleration Displacement Response spectrum (ADRS), to be compared to the multi-linear
 720 capacity curve, accounting for the initial dynamic elastic response (Figure

34(b)). The previous standard [12] considers the amplification assessing simplified expressions, (e.g. C8A.4.12 for ULS),

$$d_u^* \geq S_{De}(T_1) \cdot \psi(Z) \cdot \gamma \frac{\left(\frac{T_s}{T_1}\right)^2}{\sqrt{\left(1 - \frac{T_s}{T_1}\right)^2 + 0.02 \frac{T_s}{T_1}}}, \quad (4)$$

where, S_{De} is the elastic displacement response spectrum; T_1 is the period of the first mode shape of the structure in the interested direction; $\psi(Z)$ is the normalized mode shape assumed as Z/H , where Z is the level of the local mechanism, while H is the total height of the structure; γ is the coefficient of modal participation and can be assumed as $3N/(2N + 1)$, where N is the number of floors of the building. The spectral demand obtained through the new standard based on floor spectra may be significantly higher if compared to the previous guideline.

The evaluation of load factors through the kinematic method of limit analysis was recently updated by using rigid-block modelling. An original criterion to account for the frictional resistance in the rocking-sliding failures of URM walls was proposed to assess their in-plane and out-of-plane behaviour [113]. This criterion was validated and applied e.g. to the analysis of the local failure of masonry corners accounting for the role of thrusting timber roofs [114].

If compared to FBA, DBA provide a more rational means of determining seismic design actions for unreinforced masonry walls, avoiding inevitable material mechanical characterization [74]. The limitation of such formulations relies on the quantification of the displacement limit parameters which are calibrated based only on experimental data.

FEMA 547 [115] addresses inadequate or missing wall-to-diaphragm tie

deficiencies suggesting the use of tension and shear ties. Such anchors should
745 transfer OOP inertial loads perpendicular to the face of the masonry back
into the diaphragm or shear loads from the diaphragm into the wall where
they are resisted by IP action. For tension anchors the use of through bolt
anchored to an external steel plate and welded to an internal steel strap,
nailed to the joists and timber blockings is suggested in the case of connec-
750 tions with joists parallel to wall. On the other hand, a steel angle connection
is suggested for connections where joists are perpendicular to the wall. Ad-
hesive anchors are also recommended for aesthetic reasons giving details of
the connection procedure and material to use. A threaded rod is commonly
suggested as ATSM A36 all-thread rod, relatively ductile material with high
755 strength capacity. As highlighted from the guideline, holes need to be drilled
with a rotary drill with the percussion setting turned off to limit vibration
into the wall. The use of a diamond tipped blade is suggested. Eccentric-
ity issues, together with other aspects are discussed and some examples are
shown of different types of anchorages.

760 The ASCE/SEI 41-13 guidelines [116] covers both analysis for the evalu-
ation of existing buildings and design of retrofit measures. Tier 3 systematic
procedure is suggested as the most reliable and complete, for which the anal-
ysis must include the entire structural system. Four analysis procedures are
proposed: linear static procedure (LSP), linear dynamic procedure (LDP),
765 nonlinear static procedure (NSP), or nonlinear dynamic procedure (NDP).
Adoption of linear procedures is limited to regular and simple buildings,
while nonlinear procedures are encouraged to account for non linearities,
usually present in existing constructions. The guidelines set forth analysis

requirement in §7.2.11.1 regarding the OOP wall anchorage to diaphragms.

770 Walls shall be anchored to diaphragms at horizontal distances not exceeding $8ft$ ($2.44m$) and anchorage shall positively transmit the seismic forces generated into the OOP wall to the diaphragm:

$$F_p = 0.4 S_{XS} K_a K_h \chi W_p, \quad (5)$$

where S_{XS} is the spectral response acceleration for the selected hazard level and damping, adjusted for the site class; K_a is a factor accounting for the flexibility of the floor (not exceeding 2.0), for different Structural Performance Level (SPL); K_h is a factor accounting for the variation in force over the height when all diaphragms are rigid; χ is a factor accounting for different SPL and W_p is the weight of the wall tributary to the wall anchor. According to this standard the pullout and shear strength of expansion anchors and adhesive anchors shall be verified by approved test procedures, while for other types of anchors, the lower-bound values for strengths with respect to pullout, shear and combination of both shall be calculated according to approved building code and analysed as force-controlled action. The guideline refers to ACI [117] TMS 402 and FEMA [118] for the design and analysis of anchors.

785 New Zealand guidelines recently published [35] allow for both force-based and displacement-based procedures for the assessment of existing retrofitted URM building. The evaluation is based on the value of the earthquake rating for the building expressed as %NBS (Percentage of new building standard as assessed by application of these guidelines) obtained by dividing the calculated ultimate seismic capacity of the component by the ULS seismic demand

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with the following expression:

$$\%NBS = \frac{Capacity}{Demand} \times 100. \quad (6)$$

A building is defined as earthquake risk building (ERB) if $< 67\%NBS$. These standards define the probable capacity of diaphragm to wall connections as the lowest among 7 different failure modes. These latter failure modes are based on damage observations during 2010/11 Canterbury earthquake sequence and on studies conducted by Campbell et al. [67] (illustrated in Section 4). Probable failure mechanisms are also reported for adhesive anchorages, based on Canterbury earthquake sequence, too. These include, among the others: brick work cracking of the masonry in the proximity of the anchorages loaded in flexural tension; incorrect installation; bed-joint shear failure of the brick around its perimeter. Based on the above considerations, these standards provide tables in lieu of specific testing showing probable shear strength capacity and tension pull-out capacity for different anchorage details. The demand is calculated as the support reactions of the seismic inertial forces generated from the tributary OOP walls. Retrofitted connections are recommended to be stronger than the walls.

6. Conclusions

This paper presents a literature review on wall-to-horizontal diaphragm connections in URM buildings, considering the typical connections found in traditional buildings and the main solutions to enhance the seismic behavior of such connections. Both experimental and numerical research studies aimed at mechanically characterizing these connections are reported and discussed.

This study reveals the necessity of accounting for wall-to-floor or wall-to-roof connections in the seismic analysis of existing URM buildings and stresses the need of simple but clear guidelines for their assessment and recommendations
815 on their numerical modelling.

Considering the peculiarity of the connections, the calibration of numerical models based on experimental data is necessary for a reliable evaluation of the seismic capacity of historical constructions. Despite pull-out monotonic and cyclic tests on detailed connections for the axial and shear capacity
820 are appropriate for this scope, they are costly and complex to set-up. Failure mechanisms and ultimate dynamic capacity can be assessed through shaking table tests on building-scale and wall-scale specimens to understand the influence of the strengthening strategy on the original poor building.

Complex and detailed finite element models reveal to accurately simulate
825 the global behavior of the URM building but they are hardly used from professionals who look for simpler procedure (equivalent frame or macro models). Uncertainties can be found on the definition of the mechanical characteristics of masonry elements and diaphragm are either not-considered as structural elements or they are modelled as rigidly connected to the perimeter walls.
830 From the literature review, it is clear how the structural nonlinearities are important, especially when dealing with connections. They affect the energy dissipation of the system, and therefore they influence the assessment of both global and local mechanisms. Pushover curves are widely used in the field
835 of masonry constructions to evaluate the ultimate force/displacement capacity taking into account those nonlinearities. Non-linear time-history analysis is also considered to be a valid tool able to better simulate the structural

response under real seismic input. Selection of appropriate input motions, essential for predicting reliable results is, however, a topic requiring clearly
840 further research.

Many code standards still indicate force-based approaches for the assessment of local failure mechanism and only few refer to displacement-based approaches, in many cases more reliable in presence of local URM wall mechanisms. Currently, kinematic analysis proposed in the Italian code is the only
845 one recognized as completely reliable in European Standards, and it can be performed in nonlinear range. Furthermore, the rocking approach considers the evolution of motion during time and dissipation properties at the plastic hinge around which the masonry wall rotates are not neglected. Limitations of such approach rely on the monolithic characteristic of the block, modelled
850 as rigid, not always true for masonry walls or façades. However, improvements of the equation of motion have been done to account for the formation of an intermediate hinge, frequently developed during OOP bending of the walls horizontally restrained on top.

In literature, only few experimental data at connection level are available
855 involving uncertainties on the seismic evaluation of the original or reinforced buildings. Common practise is to perform parametric analyses based on the calibrated model, in order to investigate a large value set of the interesting parameters. This study reveals the necessity of clear and simple ready-to-use research-based procedures grounded on experimental and numerical
860 approaches useful for professionals to assess existing wall-to-horizontal diaphragm connections and design strengthening solutions in historical constructions.

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