SAHC2014 – 9th International Conference on
Structural Analysis of Historical Constructions
F. Peña & M. Chávez (eds.)
Mexico City, Mexico, 14–17 October 2014

IN-PLANE EXPERIMENTAL TESTING ON HISTORIC QUINCHA WALLS

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Keywords: Quincha, racking test, timber frames

Abstract. *The historic centre of Lima contains many large residential buildings known as casonas, constructed during the Peruvian Viceroyalty and early Republican periods (1542-1850). The ground floor walls of a typical casona are constructed with adobe, while the upper storeys are built using quincha. Quincha is a traditional Peruvian construction technique consisting of a timber frame infilled with a weave of canes and mud. Although it is widely believed that quincha has positive seismic properties, little experimental data is available to quantify this numerically.*

In order to gain a better understanding of the lateral behaviour of quincha and support results from numerical analysis, a series of racking tests were carried out with the aim of determining the stiffness, ultimate capacity and localised failure mechanisms of these walls. Two distinct geometrical variations of quincha were identified in a building in central Lima, each differing in height and lateral bracing technique. Identical frames for each variation were tested, with and without the infill, allowing the contribution of the infill alone to be quantified from the experimental results. An additional frame was also tested with its connections fixed using an epoxy glue to evaluate the impact of any strengthening work on the connections.

The results suggest that although the walls are flexible, they can withstand considerable deformations without any significant loss of resistance or failure of the load bearing system. The infill increased the stiffness of the frame by around 2.3 times, while the yield strength was more than six times greater for the infilled frame than the bare frame. During one of the tests, the nailed connection between a diagonal timber brace and the frame failed, reducing the stiffness. However, the wall was still capable of supporting significant lateral loads relying on the contribution of the infill. The information obtained can be used to develop a global numerical model of a typical casona to assess its seismic vulnerability and consider the need for local or global strengthening.

1 INTRODUCTION

Quincha is a technique found in historic buildings in towns and cities along the coast of Peru. It is comprised of a timber frame infilled with a weave of canes covered with mud and plaster (see Figure 1 and Figure 2). Although a rudimentary form of quincha was used prior to the arrival of the Spanish, the technique was developed and altered extensively between the 16th and 19th centuries. The technique reached its peak after the 1687 earthquake, when a law was passed ruling that quincha must be used for the upper storeys of any building greater than a single storey in height [1]. Today a number of these buildings survive, most dating from the 18th and 19th centuries, with quincha in the upper storeys, and the first storey in adobe or fired brick, but many are in a state of deterioration and their numbers are dwindling.

Figure 1: Exposed quincha on a building in central Lima

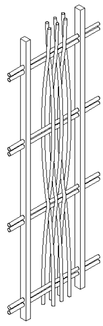

Figure 2: Detail of weave of canes

The present research is part of the Seismic Retrofitting Project (SRP), a collaboration between University College London (UCL), The Getty Conservation Institute (GCI), Pontificía Universidad Católica del Peru (PUCP) and the Ministry of Culture of Peru. The ultimate aim of the SRP is to assess the behaviour and design appropriate retrofitting systems for four case study buildings, selected because they are representative of typical historic construction systems found in Peru [2]. One such case study is a 19th century building in central Lima called Hotel El Comercio (HC). HC represents a typical casona with the first storey constructed in adobe and fired brick masonry, while the upper two storeys are quincha. The building is particularly interesting as it has three storeys rather than two, as is more common, and it has been the focus of several insitu testing campaigns to obtain information on its dynamic properties [3, 4]. A numerical model of this building is being developed which can be used to assess the global behaviour of similar structures, and evaluate possible retrofitting options. The primary issue in developing the numerical model is how to realistically model the behaviour of the quincha. The mechanical properties of the infill and its interaction with the timber frame is unknown and little experimental data is available to verify the models. The only other experimental work on historic quincha is a series of in-plane cyclic tests carried out by PUCP as part of the SRP, where 12 replica quincha frames and one original frame extracted from HC were tested [5]. In order to develop an accurate numerical model it is necessary to fully understand the behaviour of the connections, quantify the contribution of the infill to the stiffness of the frame, and identify where simplifications can be made.

This paper reports a series of tests on quincha frames constructed with and without the mud and cane infill in order to quantify the contribution of the infill for the purpose of developing a detailed numerical model. Six specimens were tested, two with infill and four without infill. The connections of one specimen were strengthened with glue to quantity the effect of rotation and pull-out of the joints. Preliminary results on the initial stiffness phase of testing were previously presented in [6] so are not discussed in detail, although the final outputs are presented.

2 DETAILS OF THE TEST SPECIMENS

2.1 Configuration of the Quincha Frame

In a typical quincha frame, the vertical timber posts are spaced between 0.5 and 1.0m, and joined at the top and bottom to horizontal timber elements by cylindrical mortice and tenon joints. The wall can be up to 4.6m in height. In addition to the mud and cane infill, the frame also contains an additional lateral bracing, which can be in the form of short diagonal wooden struts known as *citara* in between posts, or a single diagonal bracing member extending the whole height of the frame and crossing several posts, both of which are found in HC. Frame A, found on the second storey directly supported on the adobe wall, uses short diagonal struts to brace the lower portion of the frame. Adobe blocks, or in some cases, small fired bricks, are placed between these struts to provide a modest increase in stiffness and add mass. This is the most common arrangement found in Lima. Frame B, less common, is found on the third storey and does not contain the struts or bricks, but has a diagonal bracing member extending across three or four bays. In HC Frame A is around 4.4m in height, while Frame B is 3.2m.

2.2 Geometry of Test Specimens

The present series of tests were carried out by the authors in 2011 and 2012 in the structural laboratory at the University of Bath. The test specimens were designed to half scale based on architectural drawings [7]. Figure 3 andFigure 4 show the dimensions of Frame A and B respectively.

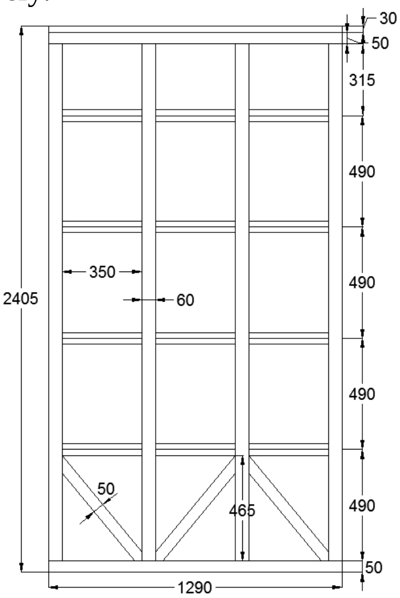

Figure 3: Frame A (dimensions in mm)

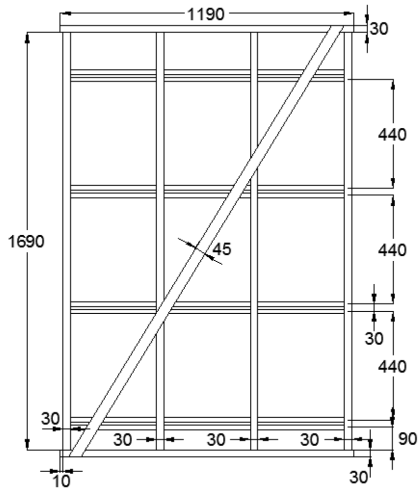

Figure 4: Frame B (dimensions in mm)

In the original building, the mortice and tenon joints that connect the vertical and horizontal elements are cylindrical and do not contain a dowel. Due to a request from the carpenter in the UK, square tenons were used, but this is not believed to significantly affect the behaviour [8]. On site observation showed that the minimum distance between the end of the beam and the post was not less than 100mm. It was not possible to extend the beam so to prevent an unrealistic failure of the mortice, the tenon was offset inwards as shown in Figure 5 and Figure 6. From onsite observations, the diagonal is connected to the beam by a lap joint, where the beam has been cut away to a depth of half the thickness of the diagonal, although the diagonal itself retains its full cross section (see [9] for more information). This set up was replicated in two of the Frame B tests but in the other two, the diagonal was simply nailed to the beam with no scarf joint. This was so the canes could be inserted without pressing on the diagonal.

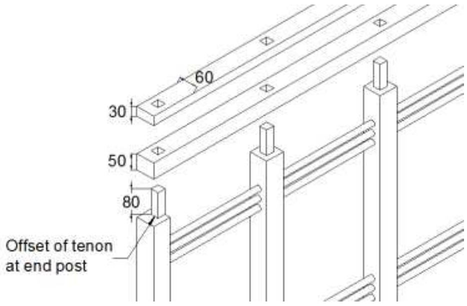

Figure 5: Frame A - mortice and tenon detail

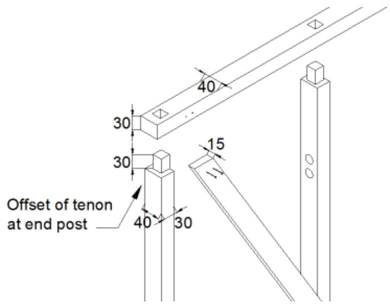

Figure 6: Frame B - connection detail

2.3 Material Characteristics

Testing to determine the species of timber used in HC concluded that two species were used, Sapelli (Entandrophragma Cylindricum) for the horizontal beams, and Cypress (Taxodium Distichum), for the vertical posts, struts and bracing [10]. The mechanical characteristics are shown in Table 1. Unfortunately this data was not available at the time of testing and since literature suggested that hardwood may have been used, English Oak conforming to D30 was sourced and used to construct the test frames. The most significant difference between the test specimen and the timber onsite is the modulus of elasticity in bending of the vertical posts. The value for the test oak is substantially higher than the cypress, leading to a lower lateral displacement in the test, and possibly more rotation occurring at the joints.

Table 1: Mechanical Characteristics of Timber used in Tests

Mechanical Characteristics (Average Values)	D30 (ref. BS EN 338)	Test Oak	Sapelli [10]	Cypress [10]
M.O.R. (MPa)	30	48 (s.d. = 6.4)	60	48
Compressive strength parallel to grain (MPa)	23	42.8 (s.d. = 1.8)	33	31
Compressive strength perpendicular to grain (MPa)	8	8.3 (s.d. = 0.6)	5	5
M.O.E Bending (MPa)	9 000	9 100 (s.d. = 3.6)	8 000	5 000
Density (kg/m^3)	640	630 (s.d. = 4.7)	490	470

Canes with a representative diameter of 12mm were threaded horizontally through preformed holes in the timber. The space between the struts in Frame A is filled with adobe blocks. Since much of the lateral restraint is taken by the timber struts, the adobe mechanical characteristics are not thought to be overly significant so cob blocks, which were easily obtainable were used in its place. The mortar was made from crushed cob blocks sieved using a 4mm sieve and mixed with fine builder's sand at a ratio of 1 part clay to 2 parts sand by volume to control shrinkage. Water was added at a ratio of 1:6 by volume.

The mud used for the infill is believed to provide a large portion of the lateral capacity, but during lateral loading the infill may detach, causing a significant loss of stiffness of the frame. The extent of this depends on the cohesion of the mud. Due to the historic nature of the building, it is not possible to obtain mud used for the infill from site and the only original material that could be extracted was an adobe block. It is probable that the mud used for the infill had slightly different properties to the adobe, but in absence of other information, the granulometry of the mud for the test infill was designed to be as similar as possible to this, although the test mud had a slightly higher clay content making it more cohesive and less prone to detachment. The mud for the infill of the test frames was made from crushed blocks of cob sieved to remove any particles larger than 5mm, although straw was retained to improve binding and reduce shrinkage. Water was added until the mud had a plastic consistency, which was reached at a ratio of four parts soil to one part water. The mud was applied to the weave of canes and left to dry for ten days. Due to the high clay content, a high level of shrinkage occurred and a gap of a few millimetres appeared between the frame and the infill. Additional mud was used to fill in the gaps caused by shrinkage. No plaster was added to the frame.

3 TEST SETUP AND PROCEDURE

3.1 Test Program

The testing program was based on the procedure outlined in BS EN 594 [11], with some modifications due to a differing geometry and lack of prior knowledge on quincha behaviour. The aim of the tests was to obtain parameters for the numerical model, specifically the contribution of the infill to the stiffness of the frame, quantitative data on the uplift or rotation of the mortice and tenon joints, and determine whether failure of any timber elements or connections occurred. Six specimens were tested, two representing Frame A, and four representing Frame B. These are outlined in Table 2. The capability was not available to push and pull the frame so the test did not consist of full cyclic loads, but rather half-cyclic loads. The lateral load was applied, then removed, and applied again in the same direction. Since full cyclic test could not be performed, Frame B was tested in each direction separately, indicated in Table 2.

Table 2: List of Test Specimens

Specimen No.	Bare/Infill	Diagonal in Tension/Compression	Strengthening
A1	Bare	N/A	None
A2	Infill	N/A	None
B1	Bare	Compression	None
B2	Bare	Tension	None
B3	Infill	Tension	None
B4	Bare	Tension	Glued Tenons

Figure 7 and Figure 8 show the test setup of Frame A and Frame B respectively. The horizontal load was applied via a hydraulic jack connected to a 10kN load cell, while the vertical load was applied to a spreader beam on rollers through two hydraulic jacks. For Frame A, a total of 11 LVDT (linear variable differential transformer) transducers were used to measure the deformation at various locations. A dial gauge was also installed to verify that no sliding occurred. Transducers 1 to 6, on the sides of the frame measured the lateral deformation, while transducers 7 to 11 measured vertical or diagonal relative displacement. Readings were taken at a rate of 1 per second throughout the duration of the tests. In addition, angle readings were taken at ten locations along the frame at 10mm displacement intervals throughout the test. The test setup for Frame B was similar to that of Frame A, but additional transducers were used to observe the deflected shape of both external posts.

3.2 Test Setup and Loading Regime

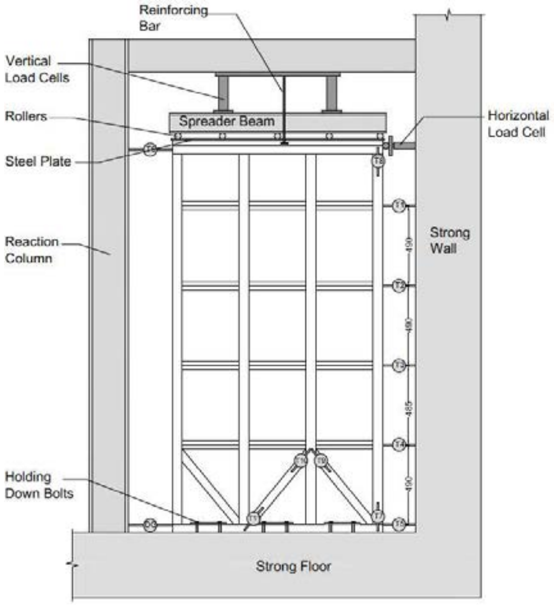

Figure 7: Loading Setup and Transducer Locations Frame A

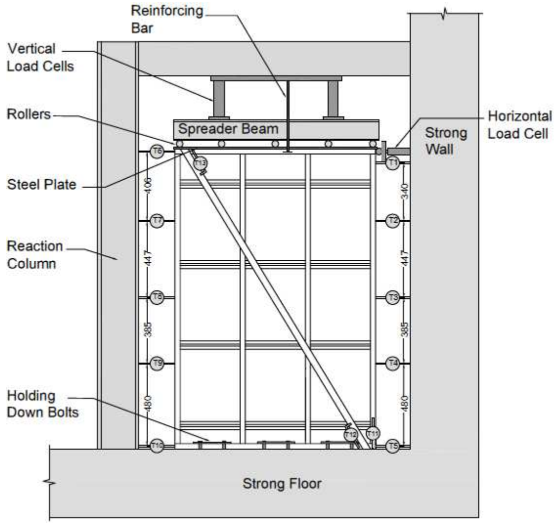

Figure 8: Loading Setup and Transducer Locations Frame B

Vertical loads of 6.4kN and 4.4kN were applied to Frame A and B respectively, representing the dead load of the floors and walls above. The lateral load was applied in three phases; Phase 1: a stabilising load cycle, Phase 2: a stiffness load cycle and Phase 3: a strength test. Phase 1 consisted of one cycle of 0.2kN, Phase 2 consisted of three cycles of 0.5kN, three of 0.7kN and three of 0.9kN, and in Phase 3 the frame was pushed to failure or 180mm, whichever occurred first. The load was applied at a rate of 40N per second, and maintained for 300s. Between each cycle the load was removed for 600s to allow the frame to settle.

4 RESULTS

4.1 Frame A

The bare frame was very flexible from the start of the test, undergoing a relatively large displacement when the load was first applied. Due to the combination of adobe and citara struts, the angle between the base of the frame and the vertical posts remained at a right angle, with the posts bending above this region. During the strength phase, considerable rotation of the mortice and tenon joints between the top beam and posts was observed and one of the tenons failed. In contrast, the infilled frame was much stiffer, with very little lateral deformation occurring during the stiffness phase and the frame remaining undamaged. However, during the strength phase, failure also occurred in one of the top tenons. Despite this failure, the infilled frame still had residual capacity due to the infill which did not detach.

Figure 9 compares the load-drift curves of the infilled and bare frames for the stiffness phase. The hysteresis loops for the infilled frame are quite wide, indicating that it is dissipating a large amount of energy through small cracking in the mud, and friction between mud and frame. This is confirmed by Figure 10, which shows the cumulative energy dissipated by both frames for the same lateral drift.

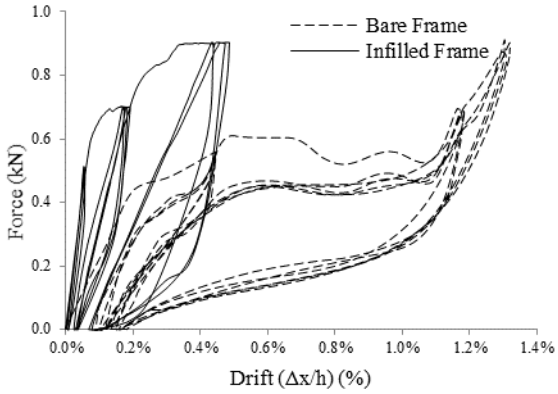

Figure 9: Comparison between the Stiffness Phase for infilled frame and bare frame

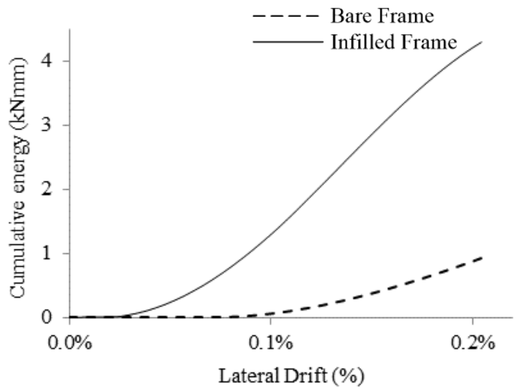

Figure 10: Cumulative energy dissipated for each frame vs. lateral drift for 0.9kN cycle 1.

Figure 11 shows the load-drift curves for Phase 3 of both tests. The maximum lateral load taken by the infilled frame is 3.8kN, compared to 1.6kN for the bare frame. For the bare frame, after the initial 0.3kN load, the relationship remains linear up to a load of 1.5kN. Then the frame yields and has very little further uptake of lateral force, although lateral capacity is maintained up to a drift of 7.5%. The infilled frame maintains a linear relationship between load and drift up to a lateral load of 1.5kN, similar to the yield load of the bare frame, before the stiffness reduces. However, the stiffness does not reduce to the same extent as the bare frame.

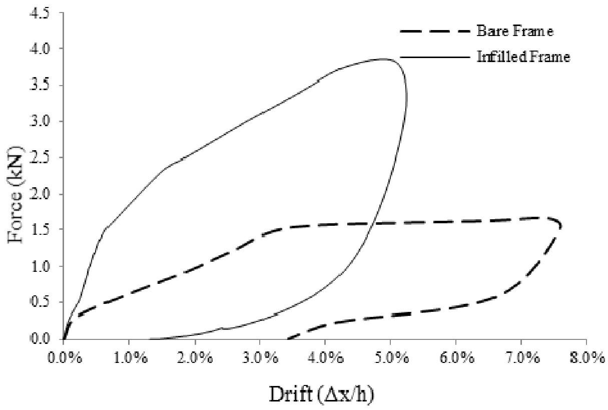

Figure 11: Comparison between the strength phase for Frame A1 and A2

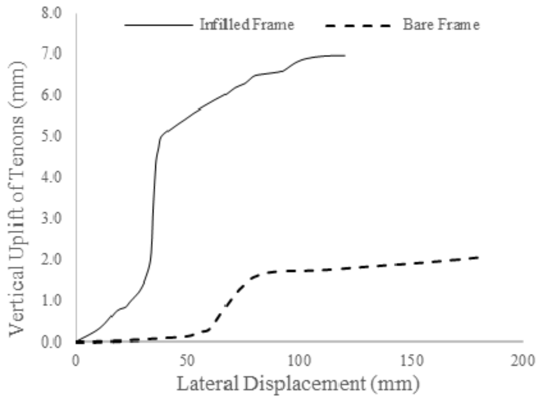

Figure 12: Vertical displacement of the post against lateral force for Frame A1 and A2

A major difference between the two frames is the uplift of the posts (shown in Figure 12). The infilled frame behaviour is dominated by the uplift of the posts, while this is not the case for the bare frame. The maximum uplift of posts measured for the bare frame was just less than 2mm, whereas the same tenon on the infilled frame pulled out over 7mm for the same lateral drift. The angle between the posts and the top of the frame remained relatively unchanged for the infilled frame, with a maximum rotation of 5 degrees, as the infill restrains relative rotation between the beam and post. This is evidence of the prevalent shear deformation of the panel as a whole rather than the flexural deformation shown by the post in the bare frame, reducing the relative rotation between mortice and tenon. In this case, rotation of the posts is resisted by the infill, not just by the tenon, reducing the moment applied to the joint. The bottom tenons on the side of the load pulled out of the mortice while remaining straight. This indicates the formation of a global strut, and a corresponding enhanced response in shear, rather than in flexure, as was the case for the bare frame.

Figure 13:Frame A2 at 4.2% drift

Figure 14: Failure of tenon in A2 (top) and A1 (bottom)

In both tests, a shear failure occurred in one of the top tenons (see Figure 14). This occurred for an estimated drift of 4.9% for the infilled frame and 3.0% for the bare frame. Since the failure occurred in the end tenons, which had been offset, further investigations should be carried out to assess whether this is a realistic failure mode that could occur onsite or exacerbated by the geometry of the tenon used in the tests.

Table 3 summarises the results for Frame A. The racking stiffness was calculated based on BS EN 594:1996 to compare the elastic stiffness of the frames while the yield point was calculated according to ASTM E2126 – 11. The maximum capacity was calculated according to BS EN 594:1996. Yield deformation refers to the displacement at the yield point.

Table 3: Summary of Results for Frame A

	Racking Stiffness (kN/m)	Yield Force (kN)	Maximum Capacity (kN)	Yield deformation (mm)	Deformation at failure (mm)	Ductility
Bare Frame	21.5	0.31	1.67	4.7	72	15.4
Infilled Frame	97.8	1.98	3.85	19.3	120	6.2

The racking stiffness of the infilled frame is more than four times that of the bare frame. There is also a significant difference between the yield points of the two frames, highlighting the substantial contribution made by the infill not only to the stiffness, but also tom the capacity of the frame. The infilled frame has a strength capacity of around two and a half times higher than the bare timber frame.

4.2 Frame B

Four specimens representing Frame B were tested; an infilled frame with the diagonal in tension, a bare frame in tension, a bare frame in compression and a strengthened bare frame in tension. The strengthening involved fixing the mortice and tenon joints with an epoxy glue to prevent uplift. Figure 15 shows envelopes for four tests for the stiffness phase. Due to the diagonal brace, and its shorter height, Frame B is stiffer than Frame A and the maximum lateral drift measured during the stiffness phase was less than 1%. It is also stiffer in compression than in tension and the reinforcement slightly increases the stiffness of the frame. All the tests had a residual drift of less than 0.2% after the stiffness phase, indicating that they were all still within their elastic range. Figure 16 shows the cumulative energy dissipation for each frame for the first 0.9kN cycle. The infilled frame dissipates more energy than the bare frames, but up to a 0.1% drift, there is little difference in energy dissipation between the strengthened frame and the bare frame in tension, as any energy dissipated by friction in the tenons is not accounted for at such a low drift.

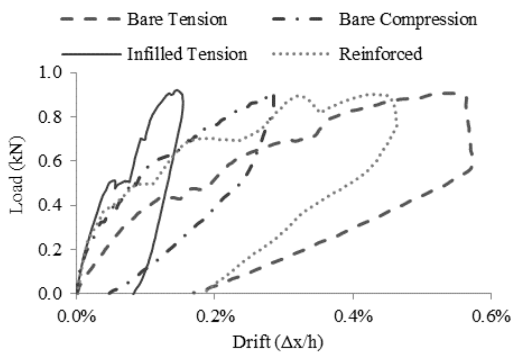

Figure 15: Envelopes for four tests.

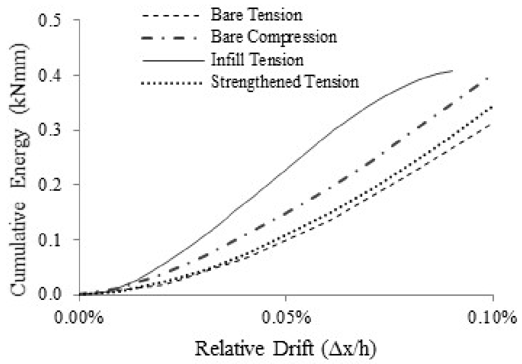

Figure 16: Cumulative energy dissipated for each frame vs. lateral drift for 0.9kN cycle 1.

The strength test load-drift curves for all frames are shown in Figure 17. The infilled frame has the highest capacity and the highest initial stiffness. The bare frame in compression has a higher yield strength than the same frame in tension but buckled suddenly at a drift of 2.3% and had no residual capacity. Failure of the frame in tension occurred due to gradual yielding of the nails or timber at the diagonal connection. No tenons were damaged in any of the tests.

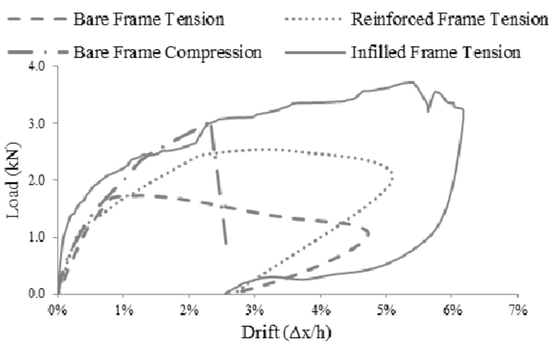

Figure 17: Comparison between the Strength Phase for all frames

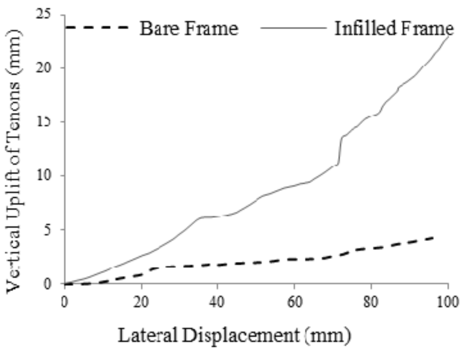

Figure 18: Vertical displacement of the post against lateral displacement for specimens B2 and B3

Figure 18 shows the relationship between lateral displacement and vertical uplift of the posts. The external post of the infilled frame uplifts a distance 4 times larger than the bare frame for the same lateral displacement. This is because the infilled frame behaves more like a rigid body, rocking as the external post lifts up, rather than deforming in flexure like the bare frame. This is further demonstrated in Figure 19 and Figure 20 which show the deflected shapes of both frames for the same drift. The post closest to the load is relatively straight for the infilled frame, whereas for the bare frame it is bowing outwards. The diagonal has much more of an influence on the deflected shape in the bare frame and it is bending itself and causing bending of the internal posts. The external post on the opposite side of the load is bowing outwards, showing that it is in compression, in the infilled frame while it has a slight S shape in the bare frame.

Figure 19: Frame B2 at 5% drift

Figure 20: Frame B3 at 5% drift

The strengthened frame has a 46% increase in capacity compared to the bare frame but the initial stiffness was not significantly increased. In addition to preventing uplift, rotation of the tenons was also prevented, which probably contributed to the increased capacity more than the prevention of uplift. As a consequence of its higher capacity, the damage caused to the strengthened frame at the end of testing was greater than the bare frame. As well as failure occurring at the diagonal connection, one of the mortices was severely damaged due to the glue preventing pull out or rotation.

In all tests, failure occurred in the diagonal connection (see Figure 21). Due to the additional depth of the top beam later observed onsite, it is thought that the connection that exists in reality is slightly stronger than was represented in the tests. Assuming that the timber members are all in good condition it is believed that the most likely failure mode involves the diagonal moving downwards along the groove in the top beam, and yielding of the nails as shown in Figure 22.

Figure 21: Failure of diagonal connection in test B3

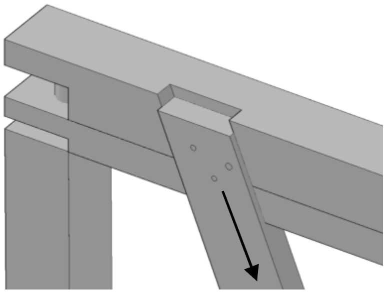

Figure 22: Predicted failure mode of connection onsite showing yielding of nails

Negligible rotation of the diagonal with respect to the top beam was observed so the connection can be assumed to be flexurally rigid, with only the ability to move axially. The stiffness depends on the elastic moduli of the timber, and failure is governed by the yielding of the nails, assuming that the timber is in good condition. If the timber has any deterioration or weakness (as in reality), timber failure should also be considered.

Table 4 summarises the results for Frame B. The racking stiffness indicates that the frame is stiffer in compression than in tension, but the strengthened tenons did not increase the racking stiffness of the frame. The racking stiffness of the infilled frame is more than 4.5 times greater than the bare frame, indicating that the contribution of the infill to the stiffness is greater than the contribution of the diagonal.

Table 4: Summary of Results for Frame B

	Racking Stiffness (kN/m)	Yield Force (kN)	Maximum Capacity (kN)	Yield deformation (mm)	Deformation at failure (mm)	Ductility
Bare Frame in Tension	180	1.71	1.73	11.4	23.6	2.1
Bare Frame in Compression	270	2.04	3.04	12.3	39.3	3.2
Infilled Frame in Tension	820	1.51	3.53	2.3	108.0	47.0
Strengthened Frame	180	1.18	2.52	6.0	62.9	10.5

5 CONCLUSIONS

A series of tests on quincha frames have been presented with and without the infill of mud and canes. The outcomes of these tests can be used to quantify the contribution of the infill and develop a realistic numerical model of quincha to assess its behaviour. The major outcomes are summarised in Table 5.

Table 5: Outputs for Numerical Model

		Experimental Observations	Conclusions for Numerical Model
FRAME ONLY	Pull-out of tenons	Tenons at bottom pull out of mortices. No vertical movement measured for top tenons	Nonlinear transversal spring calibrated from test results to model pull-out of bottom tenons. Vertical movement does not need to be considered for top tenons.
	Rotation of tenons	If tenons unable to rotate, the stiffness of the frame is overestimated.	Rotation of tenons modelled using semi rigid springs.
INFILL	Contribution of infill to stiffness and capacity	Infill increases stiffness by 4.5 times for both frames, and doubles capacity.	The mechanical characteristics used for the infill in the model can be calibrated from this data.
	Effect of the infill on the deformed shape	The overall response of the frame is fundamentally different when infill present.	Infill modelled using shell elements rather alternative of using a spring or brace to model contribution of infill to strength stiffness. The latter would give different deformed shape and misleading failure mechanism.
CITARA (FRAME A ONLY)	Connection between citara struts and frame	Negligible rotation occurs and no pull out occurs.	Citara struts modelled continuous with frame.
	Contribution of adobe blocks to behaviour	Adobe blocks undamaged. The citara prevents rotation of bottom tenons.	Not necessary to include adobe blocks in model, simply accounting for their weight. Rotation of bottom tenons prevented by using rotational spring with high stiffness value.
DIAGONAL (FRAME B ONLY)	Compression of diagonal brace	Frame stiffer in compression than tension but buckling of diagonal occurred	Buckling capacity of diagonal considered.
	Diagonal connection to post	Failure at the connection. No rotation measured, but diagonal moved axially when nails yielded.	Connection modelled using nonlinear axial spring. Stiffness and capacity of spring calculated depending on geometrical and material characteristics of particular joint
	Connections to internal posts	Relative rotation was observed between the diagonal and posts	End-releases inserted between diagonal and posts in model to allow relative rotation

In addition, the experimentally obtained quantitative data relating to drift, deformed shape and capacity can be compared to the numerical model to ensure the results obtained from the model are reliable. It should be also noted that the tests performed do not reflect frame behavior under full cyclic loading.

ACKNOWLEDGEMENTS

The authors would like to acknowledge Daniel Moore and Will Bazeley of the University of Bath for their work on the tests. They would also like to thank Daniel Torrealva and Erika Vicente for their valuable input.

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