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ORIGINAL RESEARCH PAPER

# Simplified engineering tools for seismic analysis and design of traditional Dhajji-Dewari structures

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**Abstract** This paper presents simplified engineering tools for seismic analysis of traditional Dhajji-Dewari structures, a concentrically braced timber frame with masonry infill, within the context of vulnerability assessment of existing stock, strengthening and restoration of historical heritage and feasibility analysis of future construction projects. Similar like structures can be found in various parts of America, Asia, Europe and the Middle-East. The study included tools for the evaluation of lateral force-deformability characteristics of Dhajji walls using non-linear static pushover analysis, simplified models for nonlinear dynamic time history analysis of Dhajji wall structures subjected to ground shaking, and simplified models for seismic performance evaluation of Dhajji wall structures using hand calculations. Three full scale Dhajji walls tested quasi-static-cyclically, with additional 18 tension and bend tests on timber frame connections, at the Earthquake Engineering Center of Peshawar are analyzed to understand the damage mechanism and salient features of the system in resisting lateral load, retrieve lateral force-deformability behavior, hysteresis response and viscous damping (energy dissipation) of Dhajji walls in order to calibrate tools for nonlinear static and dynamic seismic analysis of Dhajji wall structures. Applications are shown on the seismic performance assessment of example structures and design of new construction schemes. The findings from the present research study can provide help on the seismic performance evaluation of similar like concentrically braced timber frame masonry wall structures.

**Keywords** Traditional structures · Dhajji-Dewari · Concentrically braced timber frame · Damage mechanism · Simplified models · Seismic analysis · Earthquake-resistance

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# 1 Introduction

## 1.1 The need for engineering research on traditional structures

Traditional structures generally refers to the residential construction systems employing locally available cheap construction materials and traditional construction techniques of building (which may have evolved in the region over hundreds of years), resulting in low-cost dwellings, yet resilient against environmental forces. The excellent performance of these traditional structures in large earthquakes not only provided important findings for the historic preservations, which manifests the cultural identity of community, but essentially demonstrated lessons for the construction of affordable earthquake-resistant structures in regions with high seismicity where advanced construction techniques may not serve an alternative for saving lives. It is due to improper design and/or unregulated constructions of advanced systems e.g. gravity designed or unregulated poorly detailed reinforced concrete structures. This fact is evidenced during various moderate and large earthquakes in recent past in Europe and other parts of the world, in which traditional constructions have performed relatively better (Fig. 1) where at the same time modern building systems (both engineered and

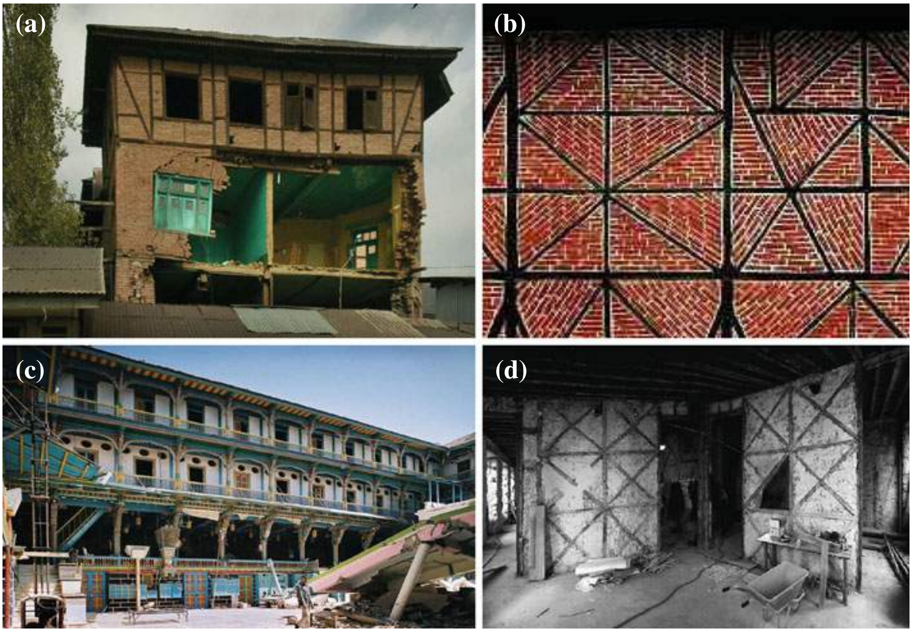



**Fig. 1** Traditional timber braced masonry construction that performed better in recent and past large earthquakes where other modern structures (engineered and non-engineered) were either heavily damaged or completely destroyed. **a** Half timber frame structure damaged in 2005 Kashmir earthquake. The unreinforced masonry wall collapsed *out-of-plane mode* whereas the timber frame masonry part is still standing despite the fact that inertia is high at the upper stories, adopted from Langenbach (2007). **b** Detail of traditional *himiş* construction near Duzce, Turkey that survived the 1999 Duzce earthquake where many modern structures were found collapsed in the region, adopted from Gülkan and Langenbach (2004). **c** Traditional timber-laced masonry structure found in Bhuj, India that survived the 2001 Bhuj earthquake while a modern concrete structure can be seen collapsed in the foreground, adopted from Langenbach (2007). **d** Timber frame masonry construction employed for the reconstruction of destroyed buildings in 1755 Lisbon earthquake, Portugal due to good performance of timber-masonry structures in this earthquake, adopted from Langenbach (2007)

non-engineered) collapsed leading to the enormous loss of lives (Avrami 2010; Doğangün et al. 2006; Güçhan 2007; Gülhan and Güney 2000; Gülkan and Langenbach 2004; Spence 2007).

No doubt modern structures i.e. engineered one, when designed with due consideration of the earthquake loading and the construction process is regulated under the supervision of trained engineers employing good quality material can perform excellent in large earthquakes as demonstrated by the recent 2010 earthquakes in Chile and New Zealand, among others. Alternatively, the new innovative construction techniques for precast/prestressed concrete and timber structures can perform exceptionally well in large earthquakes that are capable to deform laterally to large displacement with no or less damage and tolerable economic losses (Buchanan et al. 2008; Newcombe et al. 2008; Pampanin 2005; Priestley et al. 1999; Smith et al. 2008). In few other cases, even the non-engineered structures if constructed with modest effort and good sense can perform better, as evidenced in the past European earthquakes (Magenes 2006) and the recent 2010 Haiti's earthquake (Lang and Marshall 2011) among others.

Many national and international organizations recently have shown interest in the construction, conservation and promotion of traditional structural systems in the reconstruction of destroyed buildings in recent earthquakes in many developing parts of the world where primarily owner-driven housing construction programs are iterated and where modern techniques cannot be successfully implemented due to the reasons as mentioned e.g. in the re-construction and restoration projects of UN-Habitat, UNDP, UNESCO, among others. Although past earthquakes has demonstrated relatively better performance of traditional structures, the understanding of its seismic behavior and response from engineering standpoint (using simplified engineering tools, yet sufficiently accurate) is of equal importance which can provide tools for the seismic performance evaluation of existing stock within the context of vulnerability assessment, strengthening and restoration of historical heritage and feasibility analysis of future construction projects (Aktas et al. 2010; Bulleit et al. 1999; Burnett et al. 2003; Ceccotti et al. 2006; Kouris and Kappos 2012; Popovski et al. 2002, 2003; Tsakanika-Theohari and Mouzakis 2010). All the above point to the need for developing engineering tools for seismic performance assessment of traditional structural systems which can provide quantitative bases for the design and assessment of these structures.

### 1.2 Traditional Dhajji-Dewari structures

Dhajji or Dhajji-Dewari means patchwork quilt wall derived from Persian word. Dhajji structures are traditional construction type of concentrically braced timber frame with masonry infill, a single wythe thin wall. Timber frame is formed using vertical timber posts, horizontal timber beam (at the top) and horizontal and diagonal timber braces. These timber members are connected using mortise and tenon connections, supplemented with mild steel nails. This construction is largely practiced in parts of northern regions of Pakistan, India and Kashmir, including other nearby mountainous regions, for many years. However similar like structure schemes can be found also in various American, Asian, European and Middle-East countries. In this type of construction, a timber frame is first erected which is braced with timber stud and provided with timber roof truss and G.I. sheet. The wall frames are then filled with masonry in weak mortar, which is then plastered with coating, see Fig. 2. Further information on construction and detailing of Dhajji-Dewari structures can be found in the construction manual prepared by Schacher and Ali (2010). This structural system has performed relatively better in past earthquakes (Jain 2010; Jain and Nigan 2000; Rai and Murty 2005) including the

**Fig. 2** Dhajji construction type in northern parts of Pakistan. *From left to right* construction of Dhajji wall, completed structure walls and completed Dhajji housing unit

recent devastating earthquake of 2005 Kashmir with $M_W$ 7.6 (Mumtaz et al. 2008; Schacher and Ali 2008; Langenbach 2007, 2008, 2010).

Following the poor performance of modern structures in Kashmir earthquake (Javed et al. 2008; Naseer et al. 2010; Rossetto and Peiris 2009), ERRA (ERRA 2006a), *the official body of the government of Pakistan responsible for the reconstruction and rehabilitation in the earthquake affected areas*, avoided the construction of reinforced concrete buildings and consequently recommended the use of Dhajji structures for the re-construction of housing units destroyed in the event. At present, about 120,000 housing units are constructed now in the major seismic zone of Pakistan (Schacher and Ali 2008). The ERRA has prepared pictorial construction catalogues highlighting main features of Dhajji construction schemes for its onward use in the construction industry and owner-driven housing schemes (ERRA 2006b).

## 1.3 Scope and limitations of the research study

The current recommendation for the construction of Dhajji structures in the major affected areas of Pakistan is based on the qualitatively good performance of few Dhajji construction in the past earthquakes and not on a quantitative scientific rationale. Also, in the recent large earthquake of 2005 Kashmir no Dhajji structures was found in the worst affected regions but rather in some remote areas (Langenbach 2007) where the ground shaking intensity was not very high. Also, till this date no engineering tools or guidelines are widely available to provide help on the appropriate layout and structural detailing of Dhajji systems for a given region and seismic loading or validate a given structural scheme against the future expected large earthquakes.

Thus, this paper presents experimental investigation carried out on the performance assessment of three full scale Dhajji walls, with additional 18 tension and bend tests on main connections, at the Earthquake Engineering Center of Peshawar (Ali et al. 2012) in order to understand the behavior of Dhajji walls under lateral loading and develop simplified engineering tools for seismic analysis of Dhajji wall structures. Simplified methods are developed for the nonlinear static pushover analysis and nonlinear dynamic time history analysis of Dhajji structures which are computationally efficient and appealing to the practicing engineers. Furthermore, analytical nonlinear static direct displacement-based method, *called DDBD* (Priestley et al. 2007) which is well developed in Europe and has been released recently in Model-code format (Sullivan et al. 2012), is developed for future applications (restoration of historical heritage and design of new construction projects) in the performance-based assessment and preliminary seismic design of Dhajji wall systems using hand calculations. The findings from the present research study can also provide help on the seismic response evaluation of similar like traditional timber braced masonry infill frame wall structures.

## 2 Experimental investigation of Dhajji-Dewari walls

This section is largely based on the experimental investigation carried out on full scale Dhajji walls and main connections of timber braced frame, meeting the current construction practice and the design recommendations of ERRA in the region, at the Earthquake Engineering Center of Peshawar (Ali et al. 2012). A comprehensive detail of the testing program and test results were previously presented by the authors, the following sections briefly describes overview of the tests and the main findings important within the scope of the present research work.

### 2.1 Experimental tests program

The experimental investigation included in-plane quasi-static cyclic test on three full scale Dhajji walls that represent interstorey wall panel at the ground floor; gravity load applied on the wall (on the main vertical posts) represents typical loading on wall in single storey structures common in the region. Additionally, 18 tension and bend tests were performed on main connections; three test on each connection for each tension and bend test, since connections play important rule in seismic behavior of timber-braced frames (Popovski et al. 2003). The tested wall specimen and the connection detailing are shown in Fig. 3. The joint connectivity in this structural system is achieved through the use of tenon and mortise joint scheme connected by mild steel nails. The joints were tested to obtain the capacity of connections in tension and bending. The walls were deformed to large lateral displacement during the test till the peak strength of wall is developed and further, significant degradation is noticed in lateral stiffness and strength of wall. The following sections briefly describe the lateral response, damage mechanism and salient features of Dhajji walls observed during cyclic tests.

### 2.2 Response of Dhajji-Dewari walls under in-plane quasi-static cyclic lateral load

#### *2.2.1 Response mechanism of Dhajji-Dewari walls*

A typical cyclic lateral force-displacement response, viscous damping, and lateral capacity curves of the tested walls are shown in Fig. 4. The lateral cyclic loading caused the opening and closing of horizontal and diagonal braces and corner main vertical posts. The force-displacement response of the walls is found to be nonlinear at the very beginning of the lateral movement. The lateral force kept on increasing nonlinearly with increased deformation due to the closing/opening of the connections and deformation of timber panels. The deformation capacity is provided by the widening of tenon holes and bending of nails, where the tenon holes tore in tension at the ultimate limit state of the connection. The horizontal and diagonal posts pullout of the intermediate connections upon the tension failure of the tenon, i.e. tearing of holes. The lateral load carrying capacity of wall tended to stabilize which is significantly reduced upon the failure of tenon at the bottom of end vertical posts.

The damage to the tenon and pullout of the horizontal and vertical braces were mainly concentrated at the bottom and lower level connections. Table 1 provide information on the detailed description of local damages and their impacts on the global behavior of wall.

#### *2.2.2 Observed behavior of Dhajji-Dewari walls: salient features*

It is widely believed that timber-braced masonry infill wall perform better than counterpart unreinforced ordinary masonry wall due to the fact that in-plane lateral forces (which can form large diagonal cracks in unreinforced wall, susceptible then to both in-plane and out-of-

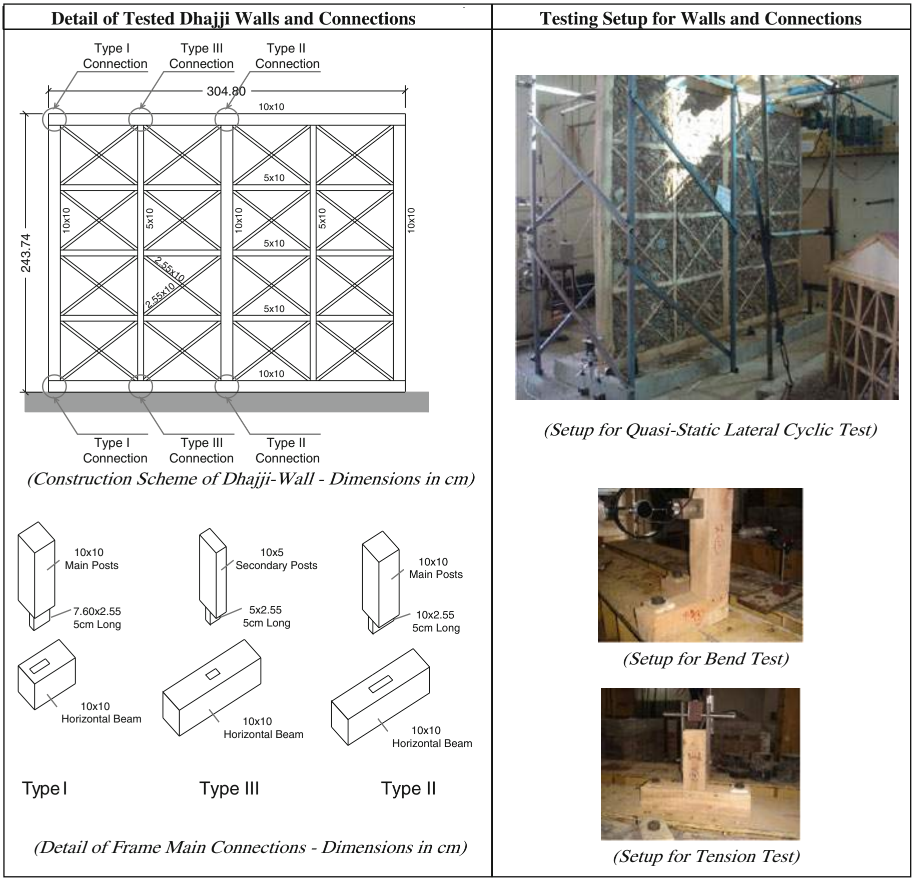



**Fig. 3** Tests on Dhajji walls conducted at the Earthquake Engineering Center of Peshawar. *From left to right* construction scheme of Dhajji wall, connection detailing, test setup for Dhajji wall and timber frame main connections

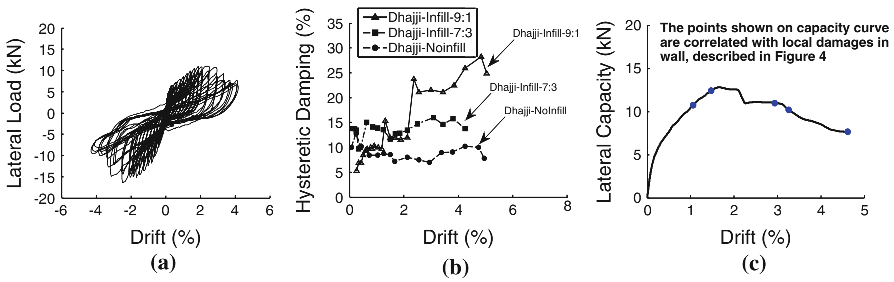



**Fig. 4** Tests results of full scale Dhajji walls. *From left to right* cyclic response of wall with masonry infill, viscous damping of walls and mean lateral capacity curve of masonry infill Dhajji walls. The *points* shown on capacity curve are correlated with local damages in wall in Table 1. **a** Hysteresis response of wall, **b** viscous damping of walls, **c** force–deformation behavior of walls

plane collapse) in the former case is mobilized by large number of small panels which localize the lateral force demand due to the provision of timber posts that break down the load transfer path from a diagonal strut mechanism, as common for ordinary masonry walls, whereby

**Table 1** Damage mechanism of Dhajji wall with increasing lateral displacement

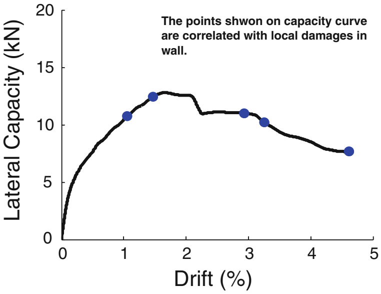
| | | |
|---|---|---|
|  | | The lateral capacity curve *mean* corresponds to the average of both positive and negative loading for each of the Dhajji wall with masonry infill |
|  | $F = 0.84F_{max}$ | Many horizontal and diagonal braces exhibits opening of joints due to tensile action |
| | $k_{eff} = 0.28k_i$ | Vertical cracks are performed at the masonry and frame interface, due to timber panels distortion |
| | $\theta = 1.05\,\%$ | All main vertical posts are intact<br>Peak strength is not developed in the system |
|  | $F = 0.97F_{max}$ | Few horizontal and diagonal braces are pulled out of joints due to tensile failure of tenon |
| | $k_{eff} = 0.23k_i$<br>$\theta = 1.47\,\%$ | Vertical cracks widened at the masonry-frame interface |
| | | Corner vertical posts exhibits opening of joints due to tensile action |
| | | Peak strength is not developed in the system |
|  | $F = 0.86F_{max}$ | Many horizontal and diagonal braces are pulled out of joints due to tensile failure of tenon |
| | $k_{eff} = 0.10k_i$ | Vertical cracks widened significantly at the masonry-frame interface |
| | $\theta = 2.93\,\%$ Supp. | Corner vertical posts pulled out of joints due to tensile action |
| | $\theta = 3.25\,\%$ at $F = 0.80F_{max}$ | Peak strength of the system is passed. Lateral force exhibits degradation |
| | | The wall is capable to provide stability under the gravity and lateral load |

**Table 1** Continued

| | | |
|---|---|---|
|  | $F = 0.60F_{max}$ | Most horizontal and diagonal braces are pulled out of joints due to tensile failure of tenon |
| | $k_{eff} = 0.05k_i$ | Vertical cracks widened largely at the masonry-frame interface |
| | | Corner vertical posts tenon are completely out of mortise |
| | | Peak strength of the system is passed. Lateral force exhibits significant degradation |
| | | Masonry infill at the bottom panels exhibits material disintegration |
| | $\theta = 4.61\ \%$ | The wall is highly susceptible to both in-plane and out-of-loading, consequently may lead to collapse of the wall |

*From left to right*: damage state of wall and timber frame components; the state of lateral force, deformation, and stiffness degradation of wall; damage description of wall and frame components

friction forces are developed at the infill-frame interface and at the infill material contact surfaces. Due to this behavior timber-braced masonry infill frame wall provide significant energy dissipation capacity to the system to resist earthquake induced cyclic loading.

The present experimental investigations revealed that the lateral force-deformability behavior of Dhajji wall depends largely on the lateral force-displacement response of timber braced frame (which is the characteristics of main connections of frame) with less contribution from infill to lateral stiffness and strength in the elastic state and negligible contribution in the inelastic state of response; the infill provide additional energy dissipation capacity (damping) to the system through infill-frame interaction (due to friction at the infill-frame interface); the provision of masonry infill do not affect the peak strength significantly, however the stiffness and strength degrade (which is not observed in timber-braced frame without infill) once the peak strength of wall is developed; the diagonal braced frame behave like a pin-ended truss structure system; the overall global ultimate behavior is governed by rocking of the wall; the lateral load carrying capacity is provided by the tension (by larger part) and bending capacity of main connections of vertical posts; the vertical gravity load helped in counteracting the overturning of wall; the tensile capacity of braces can be ignored for lateral strength evaluation, these rather primarily carry compression forces; due to the global rocking behavior of wall and pin-ended characteristics of connections, the wall can deform to very large lateral displacement without jeopardizing the stability of frame. These findings can provide help on the seismic response evaluation, modelling and performance assessment of similar like timber frame masonry wall structures.

## 3 Simplified engineering tools for nonlinear static (Pushover) and dynamic time history seismic analysis of Dhajji structures

Simplified models are developed for nonlinear static and dynamic seismic analysis of Dhajji wall structures, with the objective to provide simplified engineering tools, yet sufficiently accurate, for seismic performance assessment of Dhajji wall structures. This included the development of tools for non-linear static pushover analysis and models for nonlinear dynamic time history analysis. Furthermore, analytical models are derived, based on various case study analyses, for lateral strength-deformability evaluation of Dhajji walls using hand calculations. The following sections describe methods for static pushover analysis and dynamic time history analysis of Dhajji structures. The analytical models for hand calculations are presented in the following sections.

## 3.1 Pushover analysis of Dhajji walls

The SAP2000 v.08 Package (CSI 1999) is considered in the present study for pushover analysis of Dhajji walls, due to the user-friendly interface of the tool which makes it appealing to the practicing engineers and its wide applicability in the field of structural analysis. Nevertheless, the modelling approaching can be easily extended to other computing tools. The following consideration are made for lateral load analysis of Dhajji walls, based on the experimental response and observed damage mechanism: masonry infill does not affect the lateral stiffness and peak strength of wall, thus it can be ignored in the mathematical modelling; the peak strength of Dhajji wall depends primarily on the capacity of connections (tension *by larger part* and bending capacity) of vertical posts; the tensile capacity of horizontal and diagonal braces can be ignored.

### *3.1.1 Modelling of concentrically braced timber frame*

A complete idealization of Dhajji wall is performed where all vertical and horizontal posts and horizontal and diagonal braces are modeled, using equivalent frame modelling approach. Figure 5 provides detail on the mathematical modelling of Dhajji walls. The main timber frame i.e. vertical and horizontal posts, is modeled using beam-column type element. This is modeled using elastic bending element which is provided with lumped plasticity hinges at the ends (for vertical posts only due to the characteristics of frame connections). The beam-column element is completely defined by timber Young modulus (3345 Mpa in the present study), timber post sectional area and moment of inertia. The lumped plastic hinges are assigned with force-deformation (moment-rotation and axial tension/compression-displacement) constitutive law of connections. The diagonal and horizontal braces are modeled as truss element with tension-compression response, however the tension capacity is ignored whereby braces carry compression only.

### *3.1.2 Test and validation of the proposed modelling approach*

The proposed mathematical modelling approach is examined against the experimental tests performed on full scale Dhajji walls. The experimental tests performed on connections for tension and bending capacity evaluation are analyzed to define the constitutive law of plastic hinges for each connection type. The connection capacity (both in bending and tension) is considered as elasto-plastic, using the bi-linear idealization of experimental capacity curve as proposed by Magenes and Calvi (1997). Table 2 provides the moment-rotation and tension-deformation constitutive laws of main connections. The capacity in compression is considered as the crushing strength of timber posts.

For horizontal and diagonal braces, moment releases were applied at the ends of bracing elements to simulate the free rotation of connections and achieve pin connectivity of joints. Braces are assigned with no tensile capacity whereby only compression forces are carried by braces. The mathematical model prepared for tested Dhajji walls (Fig. 6) was analyzed in SAP2000 Package for static pushover analysis in order to compare the response (mechanism,

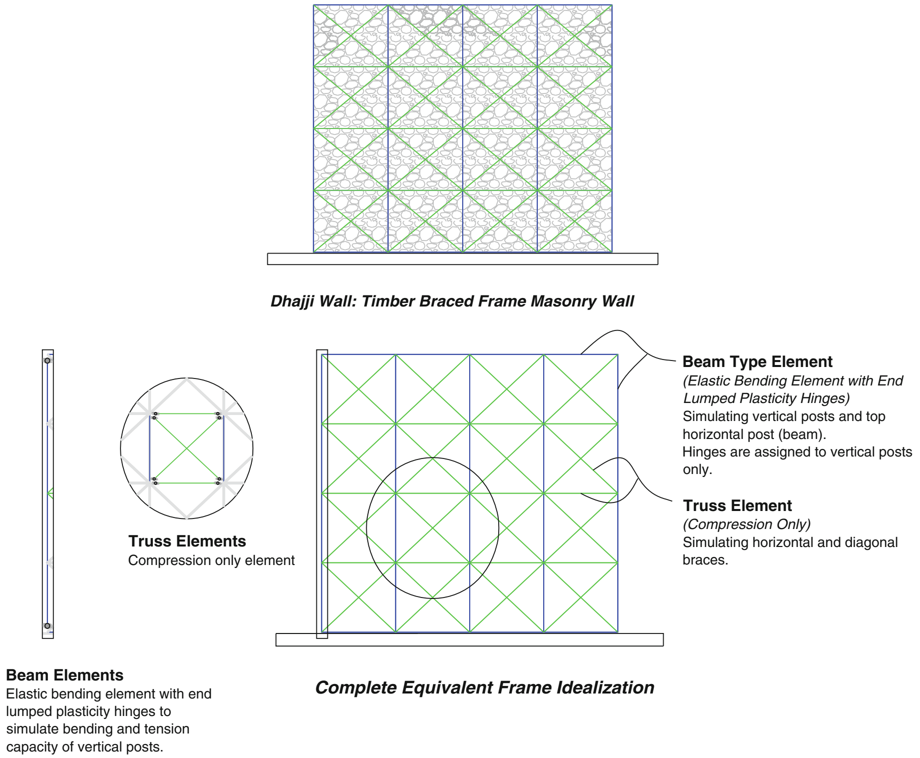



**Fig. 5** Mathematical modelling of Dhajji wall for nonlinear static pushover analysis. *From top to bottom* Dhajji wall, complete equivalent frame idealization, type of elements for analysis and detail of frame element connectivity

**Table 2** Moment-rotation and tension-deformation of hinges defined for main vertical posts of timber frame

| Connection | Limits | Moment–rotation | | Tension–deformation | |
|---|---|---|---|---|---|
| | | Moment (kN-m) | $\theta$(rad $\times$ $10^3$) | Tension (kN) | $\Delta$ (mm) |
| Type I (Corner Joints) | Yielding | 0.37 | 10 | 3.30 | 1.50 |
| | Ultimate | 0.37 | 35 | 3.30 | 12.0 |
| Type II (Central Joint) | Yielding | 0.58 | 13 | 6.67 | 4.50 |
| | Ultimate | 0.58 | 38 | 6.67 | 36.0 |
| Type III (Inner Joints) | Yielding | 0.21 | 14 | 4.90 | 2.20 |
| | Ultimate | 0.21 | 42 | 4.90 | 26.4 |

The capacity of hinge in compression is defined by the crushing strength of timber

lateral stiffness, peak strength, yield and ultimate displacement capacity) of Dhajji wall obtained using the proposed modelling approach with the experimentally observed response.

Once all the properties of mathematical model are assigned, the frame is applied with pre-compression loading at the top of main vertical posts (as applied in the test) and gravity analysis is performed first. Lateral displacement is applied at the top left (as applied in the test) to displace the wall laterally. The analysis approach considered the material as well as

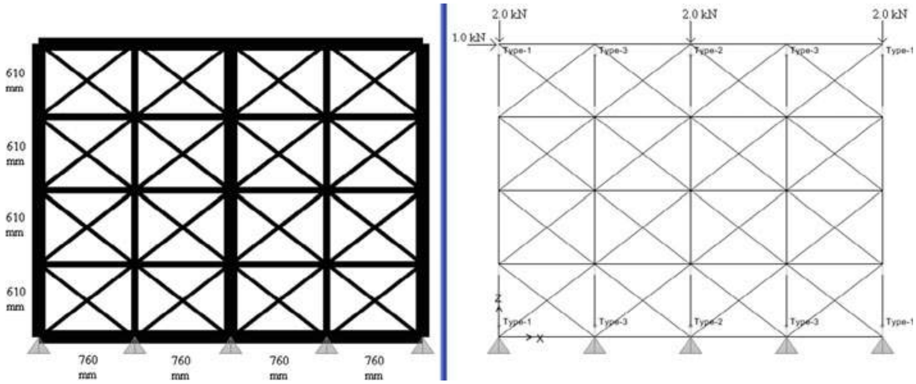



**Fig. 6** Detail of mathematical model prepared in SAP2000 Package for tested Dhajji wall using the proposed complete equivalent frame idealization

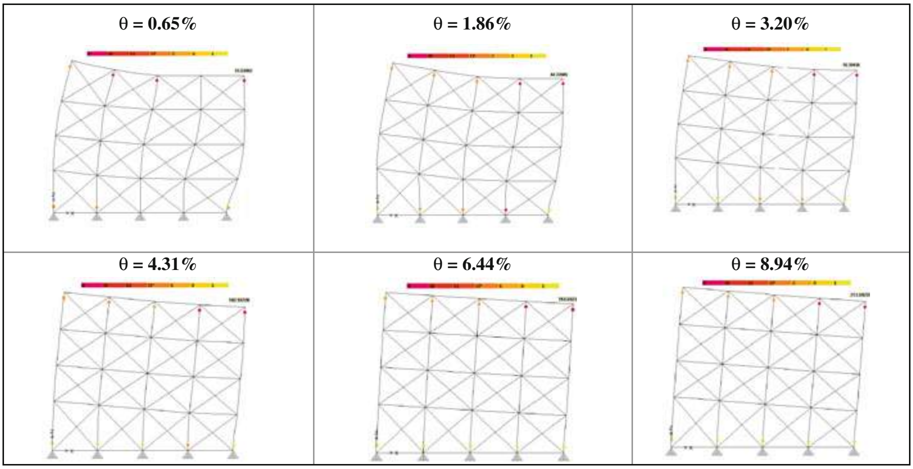



**Fig. 7** Deflected shape and mechanism of Dhajji wall at increasing drift demand, from pushover analysis, using proposed complete equivalent frame mathematical model

the geometric nonlinearity (P-Delta effect) of the system. Figure 7 shows the deflected shape of mathematical model at increasing drift demand. Figure 8 report forces demand in frame members and load transfer mechanism at various stage of the analysis.

Initially, various plastic hinges were formed in the main frame when the bending capacity of vertical posts connections is reached. However, it corresponded to very low lateral capacity of wall which indicates the lower contribution of bending capacity of connection to lateral strength. On increasing lateral drift demand, the tension capacity of vertical posts is mobilized. The wall behaved in a rocking mode on further increase in the lateral drift demand whereby the lateral capacity was largely dependent on the tension capacity of vertical posts. The pre-compression load helped in counteracting the wall overturning. These findings are within very good agreement of the observed behavior of tested walls which shows reasonable performance of the mathematical modelling approach and the considerations made for nonlinear behavior idealization of timber frame connections, frame elements and braces.

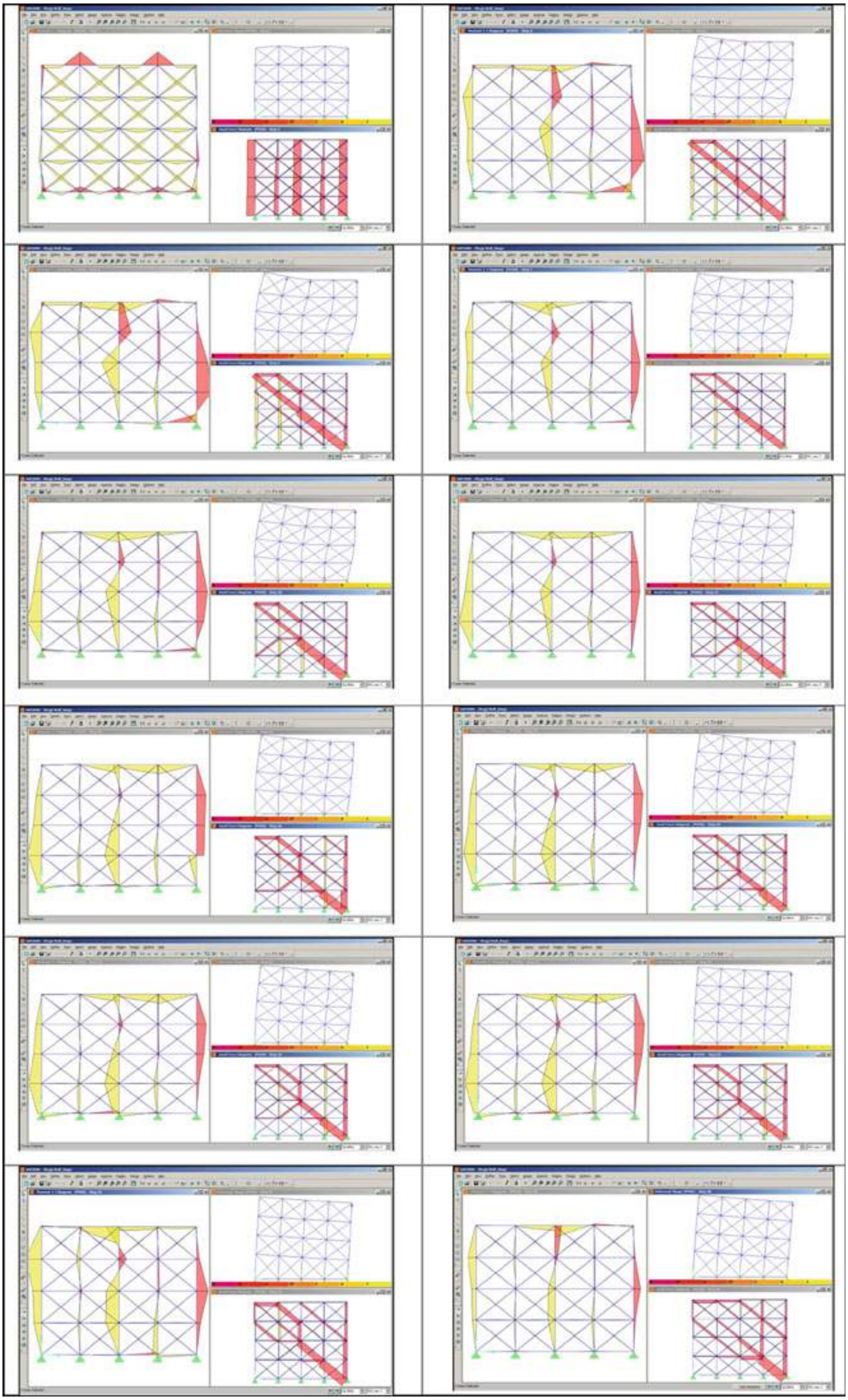


**Fig. 8** Shots from nonlinear static pushover analysis performed using SAP2000 Package showing deformed shape, bending moment demand and axial force demand in timber posts and braces. The shots are arranged from *left* to *right* and *top* to *bottom*. Step 1 shows the state of system under gravity loading only

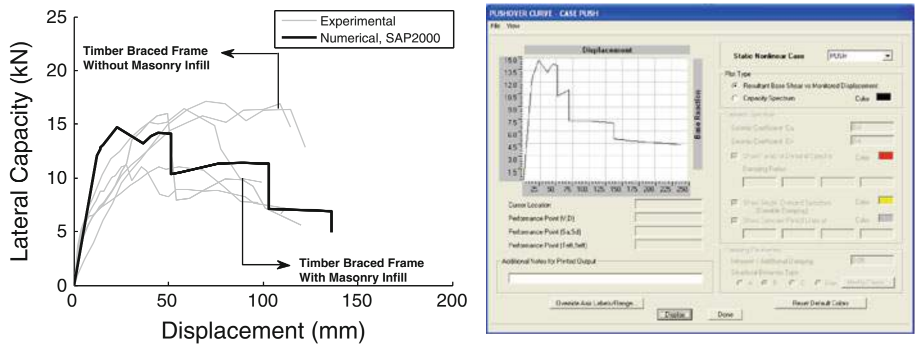



**Fig. 9** Comparison of the lateral capacity curve derived using SAP2000 Package against the experimentally obtained force–displacement curves (both positive and negative loading are shown) for full scale Dhajji walls

Figure 9 shows the comparison of lateral force-displacement response calculated using the proposed modelling approach with the experimentally obtained lateral capacity curves of Dhajji walls. The calculated capacity curve is found in a reasonable agreement with the test results. On average, the peak strength is overestimated by 7 %; the initial stiffness is underestimated by 27 %; the yield displacement capacity is underestimated by 50 %; the ultimate displacement capacity is underestimated by 8 %. The overall performance of the modelling approach seems to be very reasonable and conservative (*positively*) when compared with the test results.

## 3.2 Nonlinear time history analysis of Dhajji walls

For dynamic time history analysis of Dhajji wall structures a relatively simplified approach is proposed for mathematical modelling, keeping in view the computational ease and efficiency of the modelling technique in response analysis. The macro modelling approach and equivalent frame method as common for unreinforced masonry wall structures (Galasco et al. 2002; Kappos et al. 2002; Magenes and Fontana 1998) is adopted herein for simplified equivalent frame idealization of Dhajji wall panel. In this modelling approach, a wall is idealized with single beam-column type element (also referred as wide column analogy). In the present study, it is modeled using stiff elastic element which is provided with lumped plasticity moment-rotation hinges at the ends (Fig. 10). On lateral translation, the element simulate rocking behavior of Dhajji panel whereas the lateral force-displacement behavior of wall is simulated through the moment-rotation plastic hinges assigned to the element. The modelling approach can be easily extended also to multistory and 3D Dhajji structures.

### *3.2.1 Force-deformation constitutive law of Dhajji wall panel*

Considering the equilibrium condition of the element, the lateral force developed in the system is provided by the overturning moment resistance of the element whereas the lateral displacement of element is calculated as the element chord rotation times the panel height. The element moment resistance and chord rotation depend on the moment-rotation characteristics of plastic hinges. Two constitutive laws are proposed and calibrated in the present study: trilinear constitutive law for which the force-deformability behavior (back-bone curve) of wall closely match the capacity curve obtained experimentally and bilinear constitutive law for which the force-deformability behavior is idealized using the procedure of Magenes

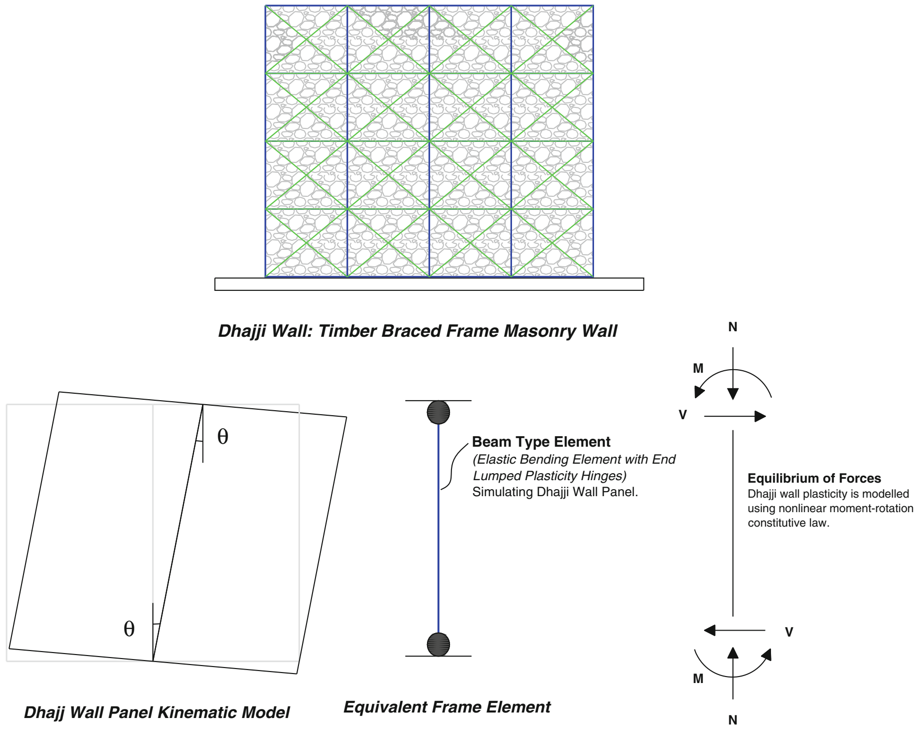



**Fig. 10** Mathematical modelling of Dhajji wall for nonlinear dynamic seismic analysis. *From top to bottom and left to right* Dhajji wall, Dhajji panel idealization using beam type element, and equilibrium of forces in beam element

and Calvi (1997), see Fig. 11a for the idealization. For trilinear constitutive law, the strength degradation, after the peak strength of wall is developed, is due to the masonry infill. This degrading behavior is not observed in the counterpart timber-braced frame without masonry infill, for which the third branch of constitutive law may be approximately considered as flat (see Fig. 9).

Investigating, the hysteretic response of Dhajji walls (Fig. 4), a pinching cyclic behavior of plastic hinges are proposed and calibrated with the experimental test results to reasonably simulate the hysteretic response of wall. Figure 11b shows the detail of hysteretic rule (for both trilinear and bilinear cases) proposed in the present study. An attempt is made in modelling accurately the unloading and reloading stiffnesses of the hysteretic rule, however essential approximation is performed due also to the limitation of the available hysteretic rules for cyclic analysis of civil structures.

### *3.2.2 Test and validation of the proposed modelling approach*

The proposed modelling approach and constitutive approximation is compared with the cyclic response of one of the tested walls. For this purpose, the peak strength of wall is considered as observed in the test (Fig. 4) whereas for drift limits the average estimate is considered, Fig. 11a. A mathematical model is prepared in OpenSees (McKenna et al. 2008) for Dhajji wall panel using the proposed modelling hypothesis: elasticbeamcolumn element is used to

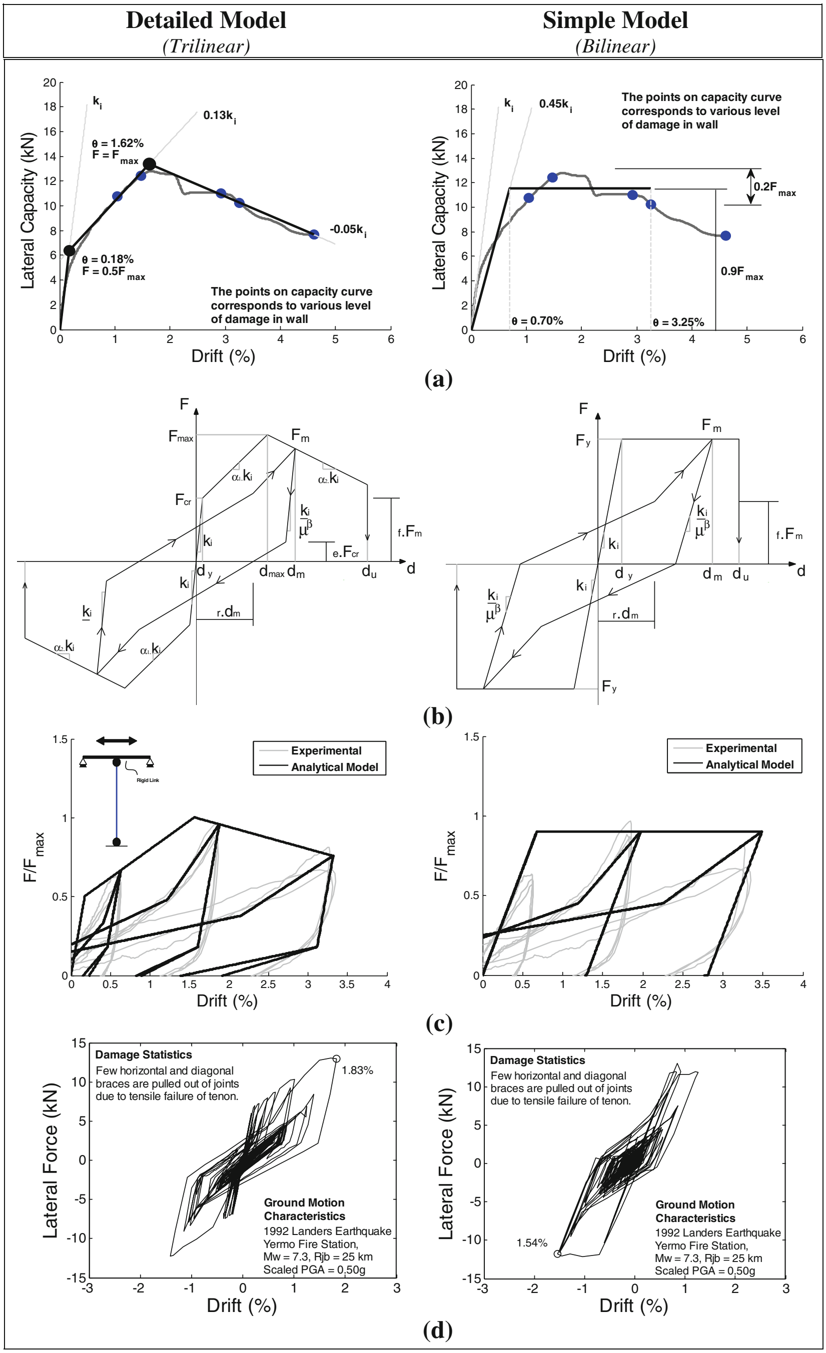



**Fig. 11** Analytical models for dynamic seismic analysis of Dhajji wall panel. *From top to bottom* lateral force-deformation constitutive law, hysteretic response of wall, comparison of analytical models with test results and test dynamic analysis of Dhajji wall. **a** Dhajji-wall lateral force-deformation constitutive law – idealization. **b** Dhajji-wall panel hysteretic rule—idealization. **c** Comparison of proposed analytical models with experimental results. **d** Comparative dynamic analysis of tested Dhajji-wall

model panel, it is provided with zerolength element at both ends, the zerolength elements are provided with stiff elastic element for translation degrees of freedom (horizontal and vertical) whereas a plastic element (assigned with the moment-rotation nonlinear behavior) is considered for the rotational degree of freedom.

In OpenSees, the moment-rotation trilinear rule is defined using *Pinching4 Material* available in the tool whereas *Hysteretic Material* of the tool is used to define the bi-linear rule, considering r = 0.65, f = 0.50, e = 0.30, $\beta = 1.0$, as defined in Fig. 11b. A rigid horizontal link is provided at the top of the beam-column element, which is provided with roller at both ends in order to allow the lateral translation of the element and develop the forces mechanism as shown in Fig. 10. It is worth to mention that this formulation is considering the material nonlinearity of the panel whereas the geometric nonlinearity (P-Delta effects) is neglected to avoid double counting, due to the fact that this effect is already implicitly considered in the constitutive law of plastic hinges, and which is a reasonable approximation for considered low-rise Dhajji structures. Nevertheless, future research is required in this regard whereby the proposed formulation can be extended then for seismic analysis of multi-storey Dhajji structures. To check the model it is subjected to lateral cyclic displacement at the top, the corresponding shear force developed in the model is calculated. Figure 11c shows the cyclic response of model under specified target drift demand and its comparison with the test results. The trilinear rule can simulate the hysteretic response of wall very much closely where the bi-linear rule seems to be reasonable when considering the post yield response.

The modelling approach and constitutive idealization is also investigated through dynamic time history analysis using accelerogram, to comparatively analyze the two models for performance assessment. The models were assigned with 1.30 Ton mass, representing the total mass of timber, infill and the mass contribution from the floor tributary area. The models are assigned with 2 % elastic damping only, as common for wood type structures (Filiatrault et al. 2002). Also, approximation is made to simulate the true pinching behavior of wall whereby the numerical model may result in hysteretic energy relatively greater than the actual. Thus, using a small fraction of damping as the source of energy dissipation in the system in elastic stage is a reasonable choice. The Rayleigh damping model with tangent stiffness proportionality is employed. Time history analysis is performed for ground motion with peak ground acceleration of 0.50 g, representing the shaking level for highest seismic zone in the country, as per BCP (2007). The system of nonlinear equations in the analysis is solved using the KrylovNewton algorithm (Scott and Fenves 2010), that accelerate the solution convergence, using the average acceleration Newmark time-stepping method (Chopra 2003; Newmark 1959), with $\gamma = 0.5$ and $\beta = 0.25$. A time step equal to the sampling size (0.02 s) of the ground motion record is considered in the analysis. Figure 11d shows the response of the model against the real ground motion excitations. The trilinear rule provide estimate of drift demand relatively higher than the bi-linear, due to the fact that the relative energy dissipated in the bi-linear case is higher than that of tri-linear rule. However, both of the model provide comparative global performance (similar damage state) of the system.

## 4 Dhajji structures design application

This section describes the design of Dhajji wall structures using direct displacement-based design (DDBD) method for seismic design of structures using simplified approach. The DDBD method can provide help on the minimum requirement of structural detailing to resist a given seismic demand without exceeding a specified performance state of the structure or alternatively check a given design scheme for specified seismic demand. The following

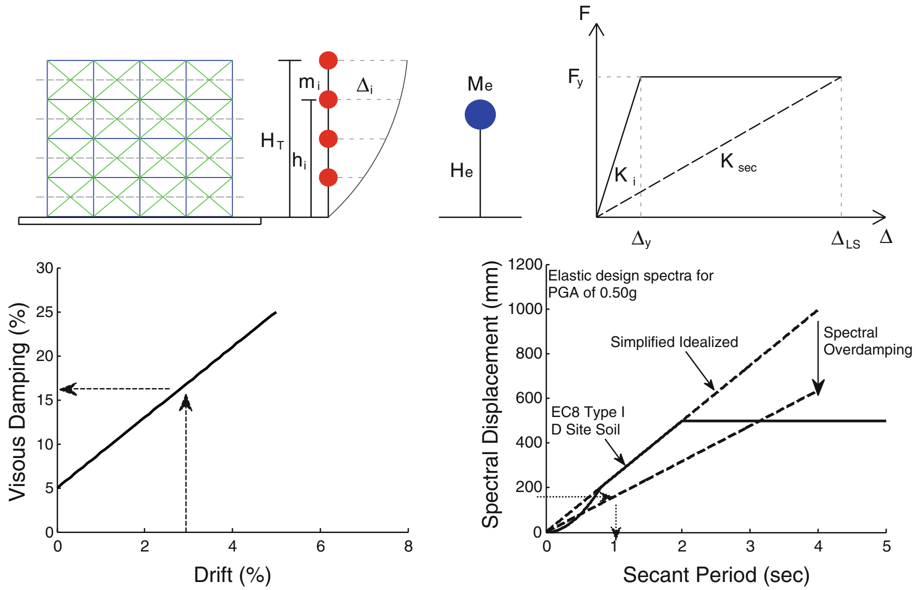



**Fig. 12** DDBD framework for Dhajji wall structures. *From top to bottom* SDOF idealization of Dhajji wall, viscous damping model and design displacement target spectra (CEN 1994). The code spectra is conservatively idealized following the current recommendations for linearization (Faccioli and Villani 2009; Faccioli et al. 2004)

section thus provides detail on the design of Dhajji structures for the highest seismic zones in Pakistan i.e. Zone 4 (PGA > 0.40 g).

## 4.1 Displacement-based design of Dhajji structures

### *4.1.1 Fundamentals of the displacement-based methodology*

The displacement-based design and assessment method used herein is based on the pioneering methodologies developed by Priestley et al. (2007) which has been released recently in Model-code format for different type of structures (Sullivan et al. 2012), however essential modification is performed to the basic method in order to be conveniently applied to traditional Dhajji structures. This method employ an equivalent single degree of freedom system (SDOF) system to simulate the nonlinear response of structure in terms of its stiffness, strength, ductility and energy dissipation capacity which are the fundamental parameters demonstrating the seismic response of the structures (Elnashai and Di-Sarno 2008). Figure 12 depicts the DDBD framework for Dhajji wall structures.

In this figure, $H_T$ represents the total wall height; $h_i$ represents the ith sublevel of wall, $\Delta_i$ represents the lateral displacement and $m_i$ represents the ith floor mass for a given deformed shape of Dhajji wall; $M_e$ and $H_e$ represent the mass and height of the equivalent SDOF system; $\Delta_y$ and $\Delta_{LS}$ represent the equivalent yield and ultimate limit state displacement that represents the displacement capacity of the actual system at the center of seismic force for a specified deformed shape; $K_i$ represents the initial pre-yield stiffness; $F_y$ represents the yielding force; $K_{sec}$ represents the secant stiffness. For seismic assessment or design of a structure, the static SDOF system is completely defined by secant vibration period, limit state displacement capacity and viscous damping:

$$T_{LS} = T_y\sqrt{\mu} \tag{1}$$

$$\Delta_{Y/LS} = \theta_{Y/LS}\, kH_T \tag{2}$$

$$\xi_{eq} = \boldsymbol{f}\left(\xi_{el}, \xi_{hyst}\right) \tag{3}$$

$$\eta_{old} = \sqrt{\frac{7}{2+\xi_{eq}}} \text{ or } \eta_{new} = \sqrt{\frac{10}{5+\xi_{eq}}}. \tag{4}$$

where $T_{LS}$ represents the secant vibration period of structure at specified target state; $T_y$ represents the vibration period at the idealized yield limit state; $\mu = \Delta_{LS}/\Delta_y$ represents the limit state ductility; $\Delta_y$ represents the idealized yield displacement; $\Delta_{LS}$ represents the maximum displacement capacity at specified target limit state; $\theta_Y$ represents the yield drift; $\theta_{LS}$ represents the ultimate state drift; $H_T$ represents the height of the structure; k represents the coefficient to convert structure to equivalent SDOF system and simulate the displacement capacity at the center of seismic force; $\xi_{eq}$ represents the equivalent viscous damping of the system; $\xi_{el}$ represents the elastic damping of the system; $\xi_{hyst}$ represents the hysteretic contribution of system damping; $\eta$ represents the spectral overdamping factor proposed by Eurocode 8 where $\eta_{old}$ is the earlier recommendation of Priestley et al. (2007) and CEN (1994) for displacement based design of structures and $\eta_{new}$ is recent of CEN (2004). However, recently Pennucci et al. (2011) showed that the expression for spectral overdamping must be compatible with the spectral sensitivity to damping of the accelerograms used to develop the calibrated equivalent viscous damping expression. The present study considered both overdamping models for the design of Dhajji structures, which are later comparatively investigated in the design examples.

### *4.1.2 Preliminary seismic design of Dhajji structures*

The primary objective of traditional Dhajji construction technique is to save lives in expected large earthquakes in the region. For this purpose, the target limit state of the system may be considered when the lateral load carrying capacity of the system reduced by 20 percent i.e. whereby Dhajji structural walls may attain sever damage to connections through tension failure of horizontal and diagonal braces and vertical main posts (Table 1).

*Design Option 1:* For a given site, the seismic demand is represented as elastic 5 % damped displacement response spectrum which is overdamped using system viscous damping and overdamped factor. The seismic demand at the secant vibration period can be compared with target specified displacement capacity of the system in order to predict whether a given structural scheme will exceed the target limit state or not. This can provide guidance on the appropriate selection of preliminary design schemes, when the displacement capacity is higher than the demand with certain level of safety factor. The secant period, for a target ductility limit, can be obtained from the modal analysis of structural schemes by computing the initial yield period of structure and Eq. (1). However, for this purpose simplified model will be required to estimate the yield vibration period of structures e.g. as a function of the structure height like the building code, like as the analytical period-height model developed by Ahmad et al. (2011).

*Design Option 2:* The above approach is iterative in nature and is best suitable to check the performance of already selected design and layout. Alternatively, the displacement-based method can be developed to give direct estimate of structural layout and detailing. For example selecting the target displacement of Dhajji wall and computing the corresponding secant period from the overdamped spectrum (Fig. 12) which can be used to estimate the minimum strength required for the structure to satisfy the design criterion.

$$F_{yeq} = \frac{4\,\pi^2\,\Delta_{LS}}{T_{LS}^2} \tag{5}$$

$$F_{yeq} = \boldsymbol{f}(L) \tag{6}$$

where $F_{yeq}$ (m/s$^2$) is the minimum equivalent design strength of the structure for the specified limit state. This design strength can be related to the length of Dhajji wall for structures of different storeys and loading condition (Ali et al. 2012).

A design scheme is proposed for Dhajji structures which can provide help on the appropriate length and arrangement of walls for a given structure and site seismicity using hand calculations. The design steps are provided in Fig. 13 and outlined briefly below, all the parameters and terminology are defined earlier.

- The design start with an arbitrary selection, although roughly judged, of Dhajji walls total length.
- Once the length is assumed, the corresponding lateral strength of the structure can be computed. Also, the ultimate displacement capacity of the structure can be computed.

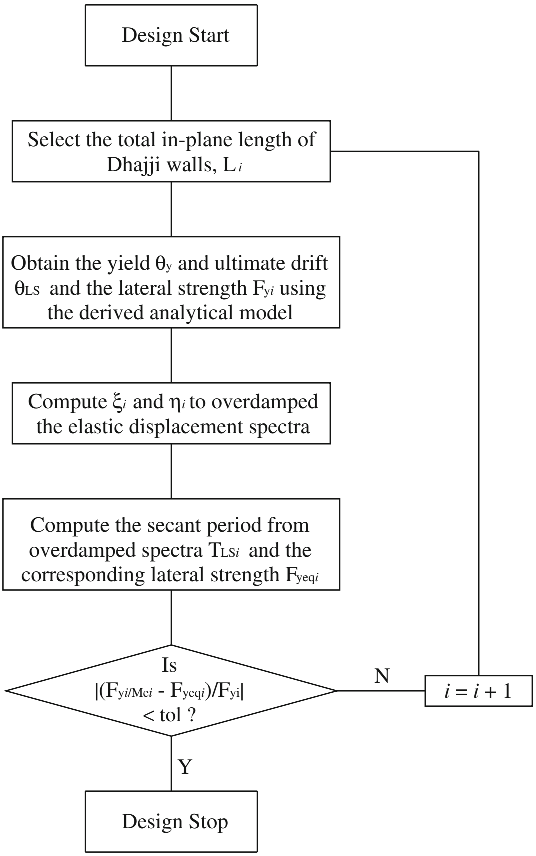



**Fig. 13** Design chart for displacement-based seismic design of Dhajji structures

- The corresponding viscous damping and spectral overdamping is computed which is used to overdamped the displacement response spectrum for target design.
- The secant period corresponding to the displacement capacity is obtained from the overdamped spectra and which is used to estimate the required strength for the design and the appropriate length.
- The required length is thus considered and the process is iterated from the initial step which converges in few iterations. The final total length defines the minimum requirement of in-plane Dhajji walls to resist the earthquake without collapse.

The following section provide the application of the proposed design method for single storey Dhajji wall structures for target ground motions of 0.50g which is a highest seismic demand that may be expected in Zone 4 (PGA > 0.40 g) of Pakistan.

#### *4.1.3 Parametric study for strength deformability evaluation of Dhajji walls*

The aforementioned DDBD framework require models to estimate the strength and deformability of Dhajji structure of a given length, if wall panel of other dimensions is employed than the tested configuration. For this purpose seven cases of Dhajji wall panels (0.76–6.0 m, see Fig. 14), representative of field practice, are analyzed through static pushover analysis in SAP2000 Package to derive simplified models for strength-deformability evaluation. This included the derivation of lateral pushover capacity curve which is idealized as elasto-plastic to compute the lateral strength, yield drift and ultimate drift of wall. Figure 15 shows the calculated parameters and fitted models. The following analytical models are developed for Dhajji wall panels.

$$F_y = 1.54L^{0.70} \times SCF;\ SCF = 0.80 + 0.034P \tag{7}$$

$$\theta_y = 0.82L^{-0.30} \tag{8}$$

$$\theta_u = 3.70L^{-0.30} \tag{9}$$

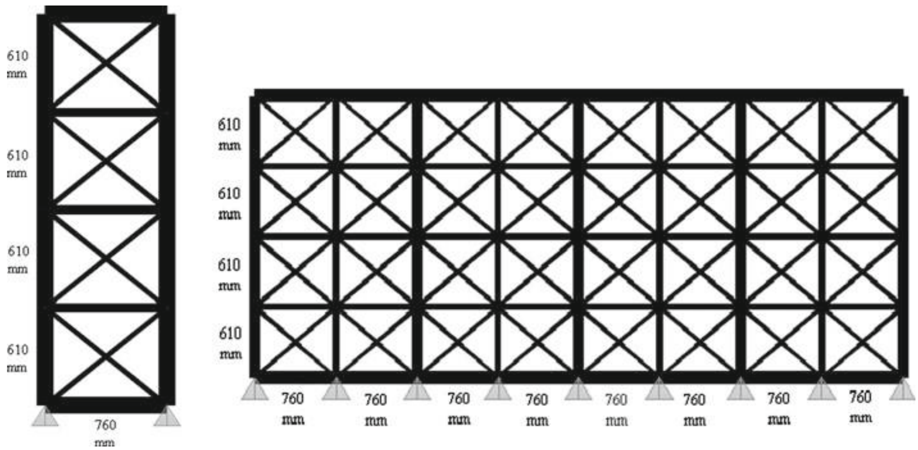



**Fig. 14** Geometric details of Dhajji wall panel analyzed through static pushover analysis in SAP2000 Package. *From left to right* minimum and maximum limiting Dhajji panels studied

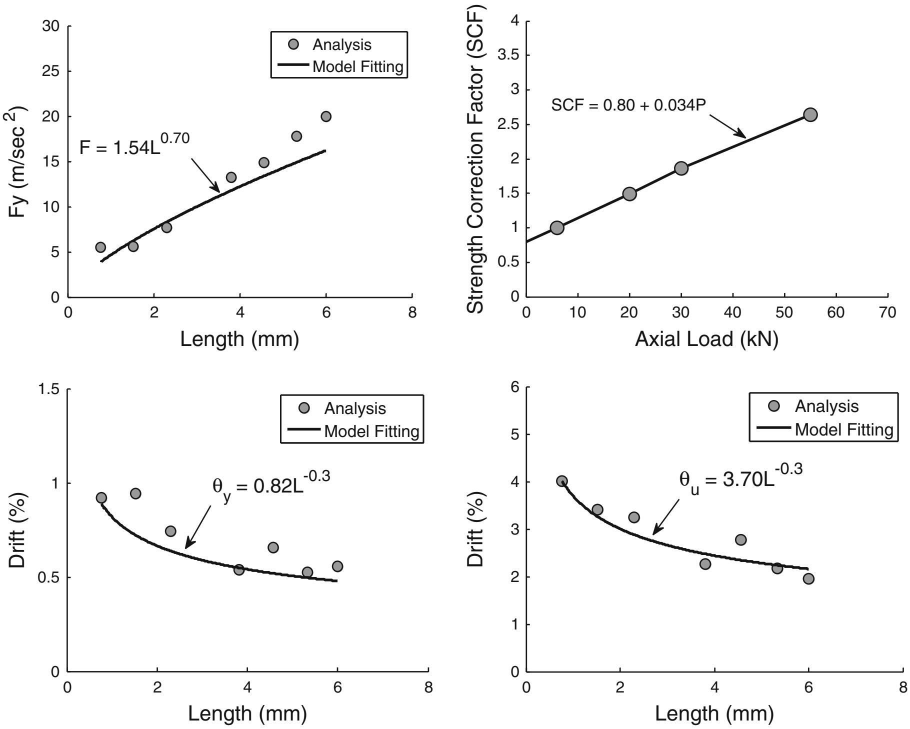



**Fig. 15** Analytical strength-deformability models derived for Dhajji wall panel. *From left to right and top to bottom* lateral strength, axial load strength correction factor, yield drift and ultimate drift capacity

where $F_y$ ($m/s^2$) represents the yield strength of wall; L (m) represents the in-plane total length of walls; P (kN) represents the total axial load on walls; SCF represents the strength correction factor; $\theta_y(\%)$ represents the yield drift of wall; $\theta_u(\%)$ represents the ultimate drift of wall. It is worth to mention that these simplified models may not be extendable to similar walls employing weaker/stronger timber members.

### *4.1.4 Parametric study for calibrating viscous damping model*

The present DDBD framework requires also the viscous damping model of structures to estimate the overdamping factor that is employed in the computation of secant period and the yield strength demand from the overdamped spectrum (Fig. 12). Generally, the viscous damping of system is represented as the combination of elastic and hysteretic damping in the current DDBD procedure developed by Priestley et al. (2007):

$$\xi_{eq} = \xi_{el} + \xi_{hyst} \tag{10}$$

Generally, the hysteretic part of damping of structural system is obtained from the hysteretic force-displacement response using the area-based Jacobson approach (Jacobsen 1960):

$$\xi_{hyst} = \frac{A_h}{2\,\pi\,F_m\,\Delta_m} \tag{11}$$

where $A_h$ represents the area off the complete stable hysteretic loop at a given lateral displacement; $F_m$ represents the maximum force developed in the cycle; $\Delta_m$ represents the

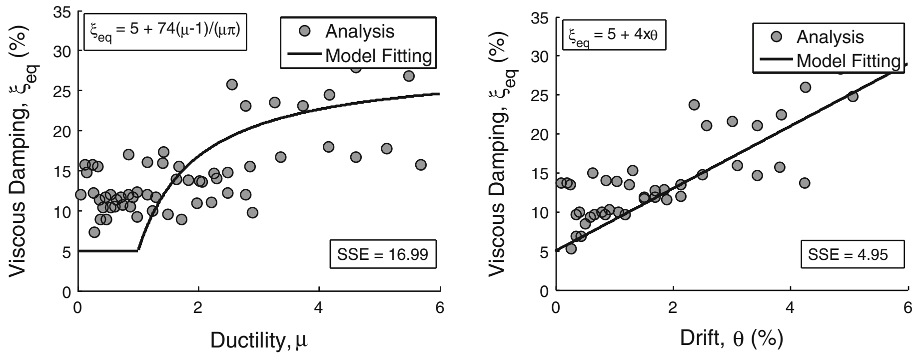



**Fig. 16** Viscous damping computed from the observed hysteresis using the Area approach and derivation of analytical model using the Priestley et al. (2007) (*left*) and Ali et al. (2012) (*right*) proposals. The linear model provides estimate with relatively less residual (sum of square of error, SSE) and best fitting. The linear model should be used confidently for drift less than 5 % and should be considered as constant beyond 5 % drift

maximum lateral displacement attained in the cycle. The above equation is used to compute the hysteretic damping of Dhajji wall panel. Furthermore, two options of analytical models, after Priestley et al. (2007) and Ali et al. (2012), are used to develop viscous damping model used in the DDBD framework (Fig. 12). Figure 16 shows the area-based derived hysteretic damping and fitting of the two analytical models. It can be observed that the linear model provide relatively best fit to the data with less SSE (sum of square of error). The difference in the damping values of the nonlinear model i.e. Fig. 16 (left), is due to the fact that additional 2 % damping is added to the data, as per the model requirement (Priestley et al. 2007).

The above derived damping consider the fact that the cyclic loading push the structure back and forth to equal displacement both in positive and negative loading direction i.e. steady-state-response of system. However, area-based equivalent viscous damping expressions are not always sufficiently accurate, because of the fact that earthquakes do not impose steady state excitations (Chopra and Goel 2001; Dwairi et al. 2007). It has been shown by Priestley et al. (2007) that for structures with wide hysteresis loops the area-based damping approach overestimate the actual damping. For example the relative overestimation of damping is higher in case of steel structures (Elasto-Plastic hysteresis) as compared to concrete wall structures (Thin-Takeda hysteresis). Thus a correction factor chart (Fig. 5) derived by Priestley et al. (2007) should be employed to correct the viscous damping used in the DDBD process.

### *4.1.5 Design examples*

Four case study examples of single storey structure are considered for the verification of the proposed method, with different design options. The structures are considered with characteristics as common in the field. The design is carried out for site seismicity with peak ground acceleration (PGA) of 0.50 g, which is the possible hazard demand in the northern parts of Pakistan, considering Zone 4 (PGA > 0.40 g). The design displacement response spectrum is conservatively considered as linear, as discussed earlier. Table 3 shows the preliminary calculated length for Dhajji structural walls along with the structure design parameters, obtained using the aforementioned design framework (Fig. 12). The co-efficient k is considered in displacement model as 0.80.

In Table 3 S1 represents the first design option considering the nonlinear viscous damping model proposed by Priestley et al. (2007) and the old version of spectral overdamping

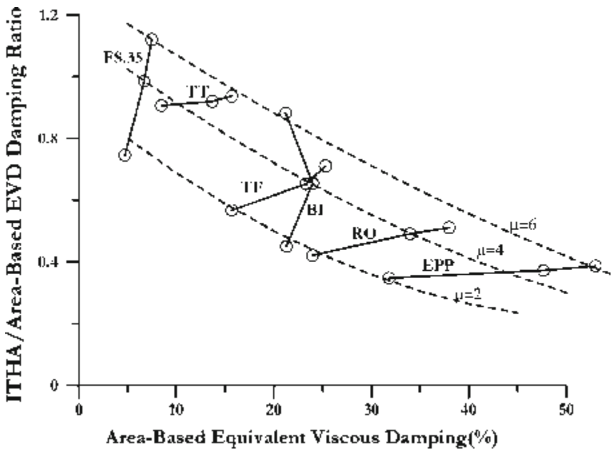



**Fig. 17** Correction factors to area-based equivalent viscous damping, reproduced from Priestley et al. (2007) with permission

**Table 3** Preliminary design of Dhajji structures for design PGA 0.50g and code specified design spectrum (BCP 2007), estimate of total wall length

| Structure type | Limit state drift (%) | | Secant period | Estimated length (m) | |
|---|---|---|---|---|---|
| | Yield | Ultimate | (s) | Total length | Lw |
| S1 | 0.75 | 3.37 | 0.74 | 8.23 | 1.37 |
| S2 | 0.64 | 2.90 | 0.58 | 13.55 | 2.26 |
| S3 | 0.63 | 2.85 | 0.56 | 14.31 | 2.39 |
| S4 | 0.51 | 2.30 | 0.39 | 29.23 | 4.87 |

factor; S2 represents the second design option considering the linear viscous damping model proposed by Ali et al. (2012) and the old version of spectral overdamping factor; S3 and S4 represents design options considering the new version of spectral overdamping factor and viscous damping model of Priestley et al. (2007) and Ali et al. (2012), respectively. The estimated total length required for a given structure scheme is divided in six in-plane panels with three in-plane panels for each wall.

### *4.1.6 Validation of the DDBD method through NLTHA*

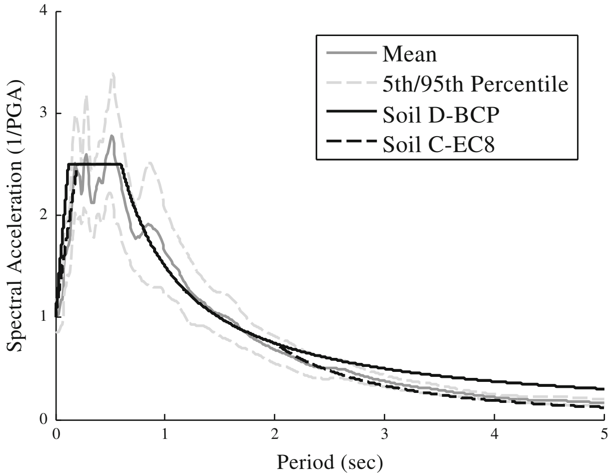
The above preliminary designed schemes are investigated through NLTHA in order to check the efficiency of the DDBD method and the design solutions using various options. The required design length per structure are used to estimate the lateral strength, stiffness, seismic mass, and initial period of the structural models. Table 4 shows the fundamental structural parameters required for the development of mathematical model for the case study structures.

The structural models are prepared in OpenSees (McKenna et al. 2008) using the simplified modelling approach for dynamic seismic analysis (Fig. 10), which are subjected to ten real accelerograms extracted from the PEER NGA database with mean spectrum compatible to the code specified spectrum for Type-I C soil in case of EC8 and design spectrum for D type soil in case of BCP (2007). Table 5 shows the response spectrum of acceleration records and their

**Table 4** Fundamental structural parameters for NLTHA of case study structural models

| Structure type | Yield strength (kN) | Yield stiffness (kN/m) | Mass (Ton) | Yield period (s) |
|---|---|---|---|---|
| S1 | 20.54 | 1, 140.86 | 3.57 | 0.35 |
| S2 | 47.93 | 3, 120.54 | 5.87 | 0.27 |
| S3 | 52.59 | 3, 478.24 | 6.20 | 0.27 |
| S4 | 177.11 | 14, 469.44 | 12.67 | 0.19 |

**Table 5** Mean acceleration response spectrum of ten natural accelerograms used for NLTHA




**Accelerograms scaling for analysis**

For structural analysis, all the accelerograms are anchored to the design PGA 0.50g considering linear scaling, resulting in the compatibility of acceleration response spectrum with the design spectrum. The linear scaling of accelerograms can reasonably provide estimate of seismic response parameters, however with relatively higher dispersion (Hancock et al. 2008)

**Table 6** Details of the accelerograms used for nonlinear time history analysis in the present study

| Date | Event | Station/Component | Mw | $R_{jb}$ (km) | $t_d$ (s) | PGA (g) |
|---|---|---|---|---|---|---|
| 04/25/1992 | Cape Mendocino | Fortuna-Fortuna Blvd | 7.1 | 23.6 | 44.00 | 0.12 |
| 06/28/1992 | Landers | Desert Hot Springs | 7.3 | 23.2 | 50.00 | 0.15 |
| 06/28/1992 | Landers | Yermo Fire Station | 7.3 | 24.9 | 44.00 | 0.15 |
| 10/18/1989 | Loma Prieta | Hollister Diff. Array | 6.9 | 25.8 | 39.64 | 0.28 |
| 01/17/1994 | Northridge | Beverly Hills | 6.7 | 19.6 | 29.99 | 0.42 |
| 01/17/1994 | Northridge | Canoga Park-Topanga Can | 6.7 | 15.8 | 24.99 | 0.36 |
| 01/17/1994 | Northridge | LA-Hollywood Store FF | 6.7 | 25.5 | 40.00 | 0.23 |
| 01/17/1994 | Northridge | Sunland-Mt Gleason Ave | 6.7 | 17.7 | 29.99 | 0.16 |
| 11/24/1987 | Superstitn Hills | El Centro Imp.Co.Cent | 6.7 | 13.9 | 40.00 | 0.26 |
| 11/24/1987 | Superstitn Hills | Plaster City | 6.7 | 21.0 | 22.23 | 0.19 |

comparison with the code-specified design spectrum. Further details on the ground motion records are provided in Table 6, after Menon and Magenes (2011) and Pampanin et al. (2002).

Figure 18 report the results of the dynamic NLTHA of case study structures for both bi-linear and trilinear pinching rules for the comparison of inter-storey drift demand with

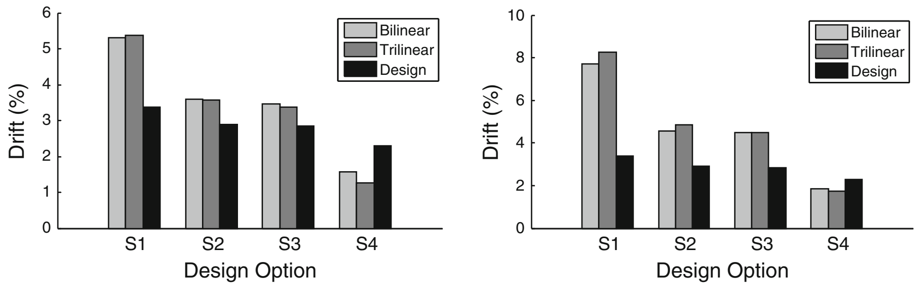



**Fig. 18** Comparison of the seismic demand through NLTHA using both bi-linear and tri-linear model, with the design capacity of case study Dhajji structures. *From left to right* comparison of drift capacity with 50 % chances of drift demand exceedance (*left*) and 95 % chances of drift demand exceedance (*right*)

the design drift capacity of Dhajji structures. The analysis shows that the displacement-based design technique can perform satisfactory for the case study structures considering both linear and non-linear viscous damping models and both the EC8 old and recent recommendations for spectral overdamping. However, the nonlinear viscous damping model and the EC8 old recommendation of overdamping factor can provide unconservtaive design solutions whereby the structure cannot meet the design requirements for specified ground motions. The linear viscous damping and the EC8 new recommendations for spectral overdamping can provide efficient design solution whereby the structure can perform relatively better; meeting the design requirements both at 50th and 95th percentile seismic demand i.e. drift demand exceeded with 50 % chances and 95 % chances. Also, it can be observe that low-rise Dhajji structures provided with minimum requirements of total wall length can survive large earthquakes expected in the region and that traditional Dhajji structures can save lives during future expected large earthquakes, given the conditions considered herein.

## 5 Dhajji structures assessment application

The present study also included an assessment example for Dhajji wall structures within the context of vulnerability assessment of structures. A fully probabilistic and dynamic method is presented herein to assess an existing structure. The method is employed to calculate ground shaking level capable of exceeding specified performance state with different chances of performance exceedance.

### 5.1 Seismic performance assessment of Dhajji structures

#### *5.1.1 Nonlinear dynamic reliability based seismic assessment method (NDRM)*

A nonlinear dynamic reliability based approach NDRM is included for probabilistic-based seismic performance assessment of example structure (Fig. 19). The methodology include the incremental dynamic analysis (IDA) technique for structural analysis as proposed by Vamvatsikos and Cornell (2002), to obtain demand (drift demand) on a structure in a probabilistic fashion which is convolved with the probabilistic capacity (drift capacity) to obtain the exceedance probability of a specified physical damage state. The exceedance probability of a specified damage state for a given ground motion is calculated using the classical reliability formulation that includes the integration of joint probability density function of demand

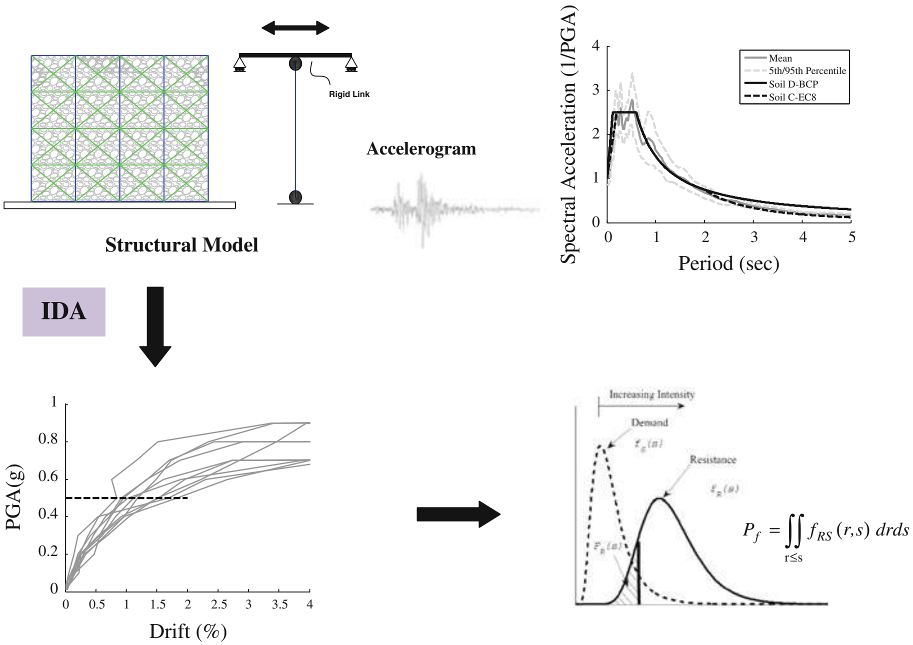



**Fig. 19** Probabilistic framework for seismic assessment of structures, after Ahmad (2011). *From left to right and top to bottom* mathematical modelling of structure system, selection and scaling of accelerograms for NLTHA, derivation of response curves through IDA and estimation of exceedance probability of ground motions for specified performance level

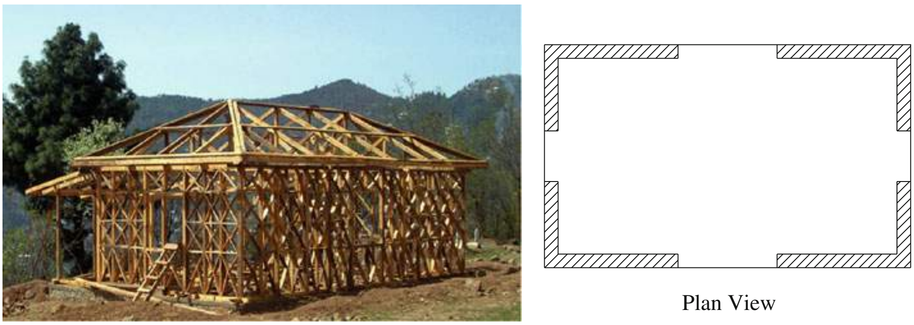



**Fig. 20** Example Dhajji structure considered for seismic assessment. *From left to right* typical configuration and plan view of the structure

and capacity, which is nevertheless inconvenient to evaluate numerically. The present study thus employed the *First Order Reliability Method* FORM approximations (Der Kiureghian 2005; Pinto et al. 2004) to obtain the damage state exceedance probability. This procedure has been recently investigated against moderate and large magnitude earthquakes for damage and collapse assessment of existing structures in Pakistan (Ahmad 2011; Ahmad et al. 2012) which shows reasonable performance of the approach.

**Table 7** Structural parameters for NLTHA of example structural models

| Direction | Yield strength (kN) | Yield stiffness (kN/m) | Yield drift (%) | Ultimate drift (%) |
|---|---|---|---|---|
| Weaker | 32.48 | 2, 114.58 | 0.64 | 2.90 |
| Stronger | 105.52 | 8, 455.13 | 0.52 | 2.35 |

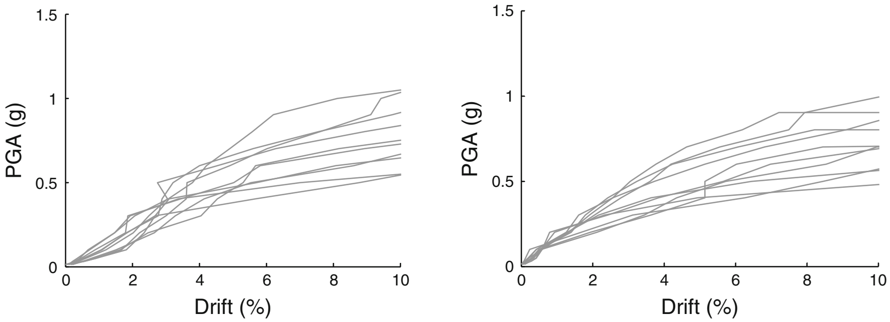



**Fig. 21** Response curves for Dhajji structures derived through NLTHA and IDA. *From left to right* using bi-linear pinching rule and tri-linear pinching rule

### *5.1.2 Example structure for assessment*

The example structure for assessment is a single storey structure in the northern high seismicity area of Pakistan. A typical configuration and plan of the structure is provided in Fig. 20. The structure is assessed in the weaker direction (short panels i.e. with less total in-plane length). The aforementioned analytical strength-deformability models, Eqs. (7) to (9), are used to compute the in-plane strength, drift limits and stiffness of the structure. Mathematical model is prepared in OpenSees for NLTHA and fragility analysis (Table 7).

### *5.1.3 Fragility analysis of example structure*

The structure response curve is derived by multiple scaling of accelerograms and incremental dynamic analysis, correlating drift demand with shaking intensity which is employed to calculate the probability of exceedance of specified limit state given the shaking intensity and derive the structure fragility functions. The fragility functions can be used to assess the performance of structures for any earthquake given the ground motion characteristic of the event. Figure 21 shows the IDA curves obtained for example structure using both bi-linear and tri-linear pinching rule for hysteresis. The response curves include only the record-to-record variability in the drift demand at a particular ground motion.

For each of the target PGA, the drift demand is convolved with drift capacity for each limit state using the FORM approximation. The structure fragility functions and collapse resilient curves are derived (Fig. 22). The collapse resilient curve can provide help on identifying the critical ground motions with different confidence level. It shows that the example structure can survive ground shaking intensity of 0.50 g with 50 chances of exceedance whereas ground shaking intensity well above 0.80 g can cause the collapse of structure (with 95 % confidence). Although, different ground motions will have different chances of collapse exceedance. The structure can survive significant large ground shaking even in the weaker direction (since ground shaking well above 0.80 g will cause the collapse of structure).

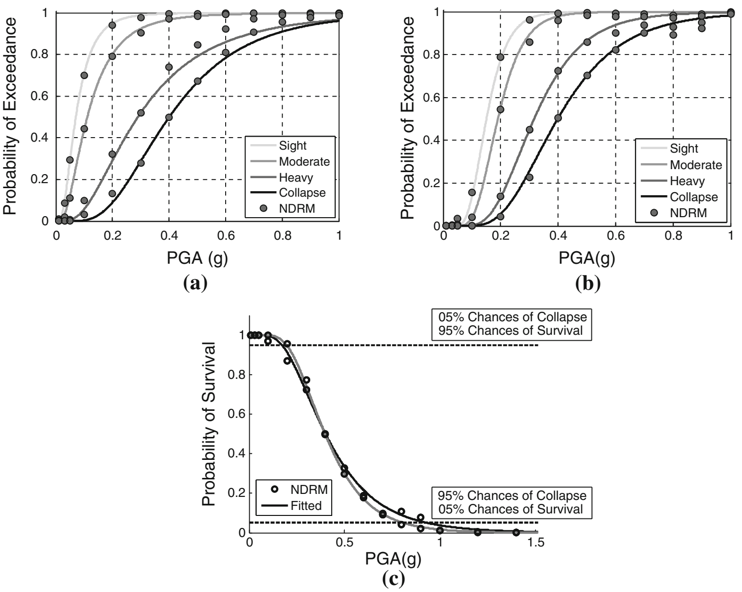



**Fig. 22** Fragility functions derived for case study Dhajji structure in weak direction using NDRM. *From left to right and top to bottom* fragility for physical damage states using bi-linear and tri-linear pinching rule and collapse resilient curve, showing chances of ground motion capable of causing collapse., **a** bi-linear pinching rule, **b** tri-linear pinching rule, **c** structural collapse resilience curve

## 6 Conclusions and recommendations

### 6.1 Conclusions

Research is of utmost importance on the seismic behavior and performance assessment of traditional construction types because of the fact that many reconstruction programs especially in the developing parts of the world employ these construction techniques to provide safety against large earthquakes to vulnerable communities, where primarily owner-driven housing schemes are iterated. The understanding of these construction from engineering standpoint is paramount. This paper thus presents, simplified engineering tools for the seismic performance evaluation of traditional Dhajji-Dewari structures, timber-braced frame masonry wall structures. This included the development of mathematical formulation and tool (SAP2000 Package) for the nonlinear static pushover analysis of Dhajji wall panels towards evaluation of lateral force-deformability behavior (lateral force–displacement curve) of wall. Simplified models are formulated for nonlinear dynamic time history seismic analysis of Dhajji wall structures subjected to ground shaking. Furthermore, analytical models are derived for seismic performance evaluation of Dhajji wall structures using hand calculations. The above tools are developed and validated in light of full scale Dhajji walls tested quasi-static-cyclically, with additional 18 tension and bend tests on timber frame connections, at the Earthquake Engineering Center of Peshawar. Similar like structures can be found in various parts of America, Asia, Europe and the Middle-East. The observed damage mechanism of Dhajji wall and salient features of the system in resisting lateral load can provide help on the further extension of the findings to similar like structures.

Additionally, a direct displacement-based design (DDBD) framework is developed for Dhajji wall structures within the context of strengthening and restoration of historical heritage, design and feasibility analysis of future construction projects, which is validated against NLTHA of designed case study structures. The DDBD framework can provide design solutions, using various approaches, that can perform satisfactorily in surviving the design level ground motions. Possible, optimization of the method can result in excellent performance of the design framework. Application on the seismic assessment of existing Dhajji structure is shown in the high seismicity area of Pakistan using fully dynamic and probabilistic based assessment approach for fragility and collapse resilience analysis. An attempt is made to generalize the findings from the present study, there is a need to carry out further experimental investigation on Dhajji walls of various geometry with the aim towards developing (*supporting*) force-deformation constitutive law which can be extended to many possible cases for seismic analysis. Also, research is required on the investigation of full Dhajji structural models in order to quantify the 3D effects on the seismic behavior of system. Extension of the research study to structures with multiple storeys is required.

### 6.2 Recommendations

The following conclusions are derived based on the investigation of traditional Dhajji structures, concentrically-braced timber frame with masonry infill, practiced in Pakistan. The Dhajji walls are formed using vertical timber posts, horizontal timber post (top beam), horizontal and diagonal braces. The connectivity of the framing members is achieved using mortise and tenon connections, supplemented with mild steel nails. The lateral force-deformability behavior (pushover curve) of Dhajji walls can be reasonably obtained analytically considering the in-plane capacity of concentrically-braced frame without including the effect of masonry infill. The braced frame behave like a pin-ended truss structure system, where the horizontal and diagonal braces primarily carry compression while their tension capacity can be ignored. The bending capacity of main connections is of less importance because the ultimate resistance is provided by the tension capacity of main vertical posts. The vertical gravity load provide help in counteracting the overturning of wall. A simplified Mohr-Coulomb like constitutive law can be developed for Dhajji panel strength to take into account the axial loading effect, as may be observed for walls in multistory structures. The Dhajji wall can deform to very large lateral displacement without jeopardizing the stability of frame. The lateral stiffness and peak strength of Dhajji panel obtained through pushover analysis is not affected by not considering geometric nonlinearity (P-Delta effects, P-Delta effects and large displacement) in the analysis, however lateral strength is reduced relatively faster (when considering P-Delta and/or P-Delta effects and large displacement). These findings can provide help on the seismic performance evaluation of similar construction types.

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