DISPLACEMENT-BASED SEISMIC PERFORMANCE EVALUATION OF
DHAIJJI STRUCTURAL SYSTEMS

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Abstract

The paper presents the derivation of mechanical models, using experimental and numerical investigations, for the development of displacement-based seismic performance evaluation of Dhajji wall, timber-braced frame with masonry infill, structural systems. The mechanical model uses secant vibration period, limit state displacement capacity, viscous damping and overdamped displacement spectrum to assess the global seismic performance of structures. Three full scale Dhajji walls typical of current construction practice in Pakistan are tested quasi-static-cyclically to obtain drift limits, hysteretic response, viscous damping and strength envelope of Dhajji walls which are analyzed to develop a simplified method for the nonlinear dynamic time history analysis of Dhajji structures using equivalent frame approach. Thirty-six case study 2D structural models, representing regional building stock, are analyzed using natural accelerograms in order to develop limit state vibration period and displacement capacity formulae. Controlled Monte Carlo simulation is used to generate random populations of regional building stock, taking into account the uncertainties in the geometrical and mechanical properties of the systems, to develop analytical fragility functions for Dhajji buildings, which, along with the fragility functions for other building typologies, can be used for seismic risk map development, regional code development, modeling insurance schemes, earthquake preparedness and planning and decision making in the region. Case study application is performed to derive damage probability matrices for scenario earthquakes in the region in order to estimate the number of Dhajji buildings required to be retrofitted.

Keywords: Displacement-Based; Nonlinear Static; Seismic Risk Assessment; Dhajji; Pakistan.

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Introduction

Dhajji is a traditional construction type practiced in parts of northern regions of Pakistan, India and Kashmir for many years. In this type of construction, a wooden frame is first erected and provided with wooden roof truss and G.I. sheet. The wall frames are then filled with rubble stone masonry in mud mortar, which is then plastered with mud coating, see Figure 1.

![Figure 1. Construction of Dhajji Wall (Left), Completed Building Walls (Middle) and Completed Dhajji Housing Unit (Right).](image)

This structural system has relatively performed better in past earthquakes including the recent devastating earthquake of 2005 Kashmir with \( M_w 7.6 \) [Mumtaz 2008, Schacher 2008]. Consequently the ERRA (www.erra.gov.pk), the official body of the government of Pakistan responsible for the reconstruction and rehabilitation in the earthquake affected areas, recommended the use of Dhajji structural systems for the re-construction of housing units in the region. The ERRA has prepared pictorial construction catalogues highlighting main features of Dhajji construction schemes for its onward use in the construction industry (http://www.erra.pk/Reports/Housing/Dhajji%20Poster/English/DhajjiPoster.pdf). However, this recommendation is based on the qualitatively good performance of Dhajji construction in the past earthquakes and not on a quantitative scientific rationale. Also, the number of Dhajji housing units were very few in the affected region as compared to brick, block and/or stone masonry buildings which makes the relative comparison less reliable. Thus, the present paper assess the seismic performance of the considered building system using experimental and numerical investigation. A simplified method is developed for the nonlinear dynamic seismic analysis of Dhajji systems using equivalent frame approach which is computationally efficient and practically appealing to the practicing engineers and designers. Also, analytical nonlinear static mechanics-based methodologies are developed for future applications in the performance-based assessment and preliminary seismic design of Dhajji structural systems.

Quasi-Static Cyclic Tests on Full Scale Dhajji Walls

Dhajji Wall Description and Test Setup

Three full scale Dhajji walls, meeting the current construction practice and the design recommendations of ERRA in the region, were tested recently at the Earthquake Engineering...
Center of the University of Engineering and Technology Peshawar, Pakistan. A comprehensive detail of the testing program and test results were previously published elsewhere [Ali 2010], the present paper will briefly describe the overview of the test and the main findings important within the scope of the present research work. The tested wall specimens and the connection detailing are shown in Figure 2. In this structural scheme, the connectivity of joint is achieved through the use of mortise and tenon connected by mild steel nails which makes the system behave as pin ended truss in which primarily the forces are transferred to the foundation through the tension compression braces and vertical posts: the compression forces are transferred through the tenon-mortise contact while the tensile forces are transferred through shear forces developed in nails.

Two of the three specimens are provided with rubble stone masonry in mud mortar infill, stone to mud ratio of 9:1 for DW1 and 7:3 for DW2. The third wall specimen, DW3, is considered without masonry infill in order to investigate the effect of infill on the mechanical properties of walls. Each wall is subjected to pre-compression level of 2.0 kN to simulate the roof loading on top of walls, respecting the field condition. Lateral cyclic load is applied through displacement-controlled actuators attached to the top horizontal wooden beam of the wall with three complete cycles for a specified target displacement. Displacement transducers are used to measure lateral displacement at the top of the wall, vertical displacement at the extreme end posts, deformation in the diagonal braces and to monitor the out-of-plane movement of the wall which is restrained during loading.

**Observed Response of Dhajji Walls**

The lateral cyclic loading caused the opening and closing of bottom connections at the extreme end vertical posts and at the intermediate connections of beams and braces. 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.
coming from the closing/opening of the connections and deformation of tenon. The deformation contribution from the tenon in compression arise from the tenon-mortise contact and key damage while in tension the deformation is contributed by the widening of tenon holes, provisioned for nails, which 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, and the lateral load carrying capacity of wall get stabilized which is significantly reduced upon the failure of tenon at the bottom of extreme end vertical posts. The damage to the tenon and pullout of the beams and braces was mainly concentrated at the bottom and lower level connections making a soft-storey phenomenon at the ultimate limit state of walls. A typical cyclic lateral force-displacement response, viscous damping, and bilinearized equivalent capacity curves of walls are shown in Figure 3.

![Figure 3. Cyclic Response of Wall With Infill (Left), Viscous Damping (Middle) and Equivalent Capacity Curves (Right).](image-url)

The walls viscous damping is obtained using the area based approach of Jacobsen [1960], normalizing the hysteretic area of the stable complete cyclic envelop over the elastic area and factor $4\pi$. Significant pinching of the hysteretic response caused the viscous damping to remain fairly constant throughout the applied displacement history. The inclusion of masonry infill increased the viscous damping significantly. The shift in viscous damping for DW1 wall is due to the tensile failure of tenon at the bottom end connection upon which the masonry infill is relatively more excited to contribute to the energy dissipation of the system.

The equivalent capacity curves are obtained by normalizing the lateral strength over the seismic mass participation, considered as 80 percent of the total wall mass and additional mass on top, in order to represent the walls as equivalent single degree of freedom (SDOF) systems. The high percentage of seismic mass of the wall is considered due to the reason that the wall behaves with shear type deflected shape with demand mostly concentrated at the bottom level of walls. The inclusion of masonry infill reduces the seismic capacity of walls dramatically which is because the masonry infill does not affect the lateral load carrying capacity of walls but increased the seismic mass participation by larger part which consequently reduces the seismic capacity of the system. Nevertheless, relative to counterpart systems, e.g. brick masonry piers [Javed 2008], the seismic strength of Dhajji walls is higher in the order of around three times.
Displacement-Based Seismic Performance Evaluation of Structures

The calibration of a nonlinear static analytical mechanics-based method is performed for the seismic performance evaluation of Dhajji structural system. Also, a probabilistic framework is developed for the performance evaluation of Dhajji buildings on regional scale. The displacement-based method, developed by Crowley [2004] for reinforced concrete buildings and further developed for brick, stone and adobe masonry buildings of Pakistan by Ahmad [2010a, 2010b, 2011c] and consequently developed for Dhajji structural systems of Pakistan herein.

Nonlinear Static SDOF Systems, Mechanical Models, for Dhajji Wall Buildings

The seismic response of Dhajji wall buildings is mainly governed by the global mechanism and in-plane response of walls. An equivalent SDOF system is used to simulate the nonlinear response of actual building in terms of its displacement capacity at different performance levels, damage states, see Figure 4 for an SDOF idealization of Dhajji wall.

![Figure 4 Nonlinear Static SDOF Idealization of Dhajji Wall Systems.](image)

In this figure, $H_t$ represents the total wall height; $h_i$ represents the $i^{th}$ sublevel of wall, $\Delta_i$ represents the lateral displacement and $m_i$ represents the $i^{th}$ 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 of a building, the static SDOF system is completely defined by secant vibration period, limit state displacement capacity and viscous damping:

$$T_{LS} = T_y \sqrt{\mu}$$  \hspace{1cm} [1]

$$\Delta_{LS} = \theta_{LS} k H_t$$  \hspace{1cm} [2]

$$\xi_{eq} = \xi_{el} + \xi_{hyst}$$  \hspace{1cm} [3]
where $T_{LS}$ represents the limit state secant vibration period; $T_y$ represents the yield period; $\mu = \Delta_{LS}/\Delta_y$ represents the limit state ductility; $\Delta_y$ represents the yield displacement capacity; $\Delta_{LS}$ represents the limit state displacement capacity; $\theta_{LS}$ represents the limit state drift; $H_T$ represents the height of the system; $k$ represents the coefficient to convert structural system 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.

**Individual Building Design:** For the seismic design of buildings to respect the life-safety and heavy damage control criterion, the ultimate limit state of the system may be considered when the lateral load carrying capacity of the system reduced by 20%. For a given site, the seismic demand is represented as elastic 5% damped displacement response spectrum which is overdamped using system viscous damping [3] and overdamped factor [CEN 1999]. The seismic demand at the secant vibration period [1] can be compared with ultimate displacement capacity [2] of the system in order to predict whether a given structural scheme will exceed the ultimate 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. Such design schemes will be verified then through nonlinear time history analysis (NLTHA) of structural models. The present paper will not discuss further the design of individual structural systems and will only focus on the regional scale performance evaluation of Dhajji buildings.

**Earthquake Loss Assessment on Regional Scale:** For earthquake loss assessment intermediate limit states are also important for the economic loss consideration, since retrofit will be required after a given seismic event. Controlled Monte Carlo simulation is used to generate random buildings with different geometrical and material properties representing regional building stock; the variability of each property being defined a priori using a complete probabilistic distribution respecting the uncertainties in the capacity of the buildings. Once the population is generated, the limit state displacement capacities, secant periods and viscous damping are computed using calibrated structure-specific empirical models and pre-defined damage scale. For a given earthquake, each building from the generation is analyzed for the capacity-demand check at secant periods using overdamped displacement spectrum. The number of buildings having capacity less than the demand divided by the total number of generated buildings gives an estimate of the limit states probability of exceedance ($P_f$). The number of buildings in a given damage state i.e. damage probability matrices (DPM), is obtained as follows: undamaged ($D_0$)=1-$P_{f1}$; minor ($D_1$)=$P_{f1}$-$P_{f2}$; moderate ($D_2$)=$P_{f3}$-$P_{f2}$; major ($D_3$)=$P_{f4}$-$P_{f3}$; near-collapse ($D_4$)=$P_{f4}$-$P_{f3}$, which are used to estimate the regional socio-economic losses of earthquakes and develop regional risk maps [Ahmad 2010a].

**Simplified Method for Nonlinear Time History Analysis of Dhajji Structures**

The method developed herein is based on the equivalent frame idealization of Dhajji wall system using multi degree of freedom (MDOF) shear type model with elastic frame elements provided with nonlinear lumped inelasticity hinges, see Figure 5.
The present study considered the methodology developed and employed by Ahmad [2010a, 2010b, 2010c, 2010d], *SD-SAM*, for other building typologies in the region which is practically appealing and computationally efficient. Each of the lumped masses are allowed to move only in the horizontal degree of freedom with all other restrained from movement. The frame elements are significantly stiff and which are provided with lumped inelastic hinges having an appropriate nonlinear force-displacement response that, only, contribute to the displacement capacity of the system. The nonlinear behavior of the hinge is calibrated from the experimental response of the system. A trilinear pinching force-displacement hysteretic rule is considered for which the forces and displacements are obtained as follow. Since, the main contribution was observed to be contributed by the lower levels in a soft-storey ultimate mechanism, the sub-storeys i.e. each frame element’s shear strength can be obtained easily. Also, the inclusion of infill does not affect significantly the sub-storey strength which is mainly coming from the wooden truss system, thus each sub-storey is assigned with the same force-displacement rule. Nevertheless, the shear type modeling hypothesis will make the system to make a soft-storey always, unless openings are found in above levels, since the storey shear demand increases down the height and which is always maximum at the bottom sub-storey.

From the observed response, the cracking force, see Figure 5, is found to be 0.45 of the maximum force developed in the system. A pinching cyclic behavior is observed in the tests in which the reloading stiffness increases at the force level of 0.5F in order to attain the previous peak force, F. The increased pinching at zero displacement was not able to be considered in the model thus the unloading stiffness is selected to obtain similar hysteretic damping as observed in the experiment. The ultimate limit state of the frame elements is considered when the strength of the element drops by 20% which usually corresponds to the tenon damage at the bottom ends. The maximum force in the system is achieved at the pullout of the horizontal beams and braces at the lower sub-storeys.

The structural model of the tested wall is designed in OpenSees [McKenna 2008] using stiff elastic beam-column element for frame elements and hysteretic material with pinching cyclic response to define...
the nonlinear properties of hinge. The model is analyzed using 10 natural accelerograms obtained from the PEER-NGA data base for soft soil with mean spectrum compatible to the 5% damped elastic spectrum of EC8 Type-I-C soil [CEN 1999]. The lumped masses are obtained at each sub-storey which were adjusted proportionally to obtain the 1\textsuperscript{st} modal vibration period as computed for the tested wall. All the accelerograms were scaled to PGA level of 0.30g, the mean elastic 5% damped acceleration spectrum along with the 16\textsuperscript{th} and 84\textsuperscript{th} percentile spectra are shown in Figure 6, in order to exceed the cracking limit state of the structural model. The deformed shapes of the structural model at the cracking limit state is also shown in Figure 6, when the mean top displacement of 5mm is obtained by increasing the starting drift value of frame elements to 0.27%. The nonlinear response of frame element at the bottom sub-storey, at PGA level of 0.25g, is also shown in Figure 6, where the response was linear for frame elements at upper sub-storeys.

Similarly the input excitations were scaled linearly to exceed the post-crack limit states of the structural model in order to calibrate the corresponding drift limits, 4% for maximum force level i.e. pullout of the horizontal beams and braces, and 8% for near collapse state i.e. tenon failure of the extreme ends main vertical posts, for the nonlinear rule of frame elements.

**Figure 6** Acceleration Spectra Selected for NLTHA (Left), Deformed Shape of Dhajji Wall at Cracking (Middle) and Response of Bottom Sub-Storey at PGA Level of 0.25g.

**Derivation of Mechanical Models for Dhajji Buildings**

**Characteristics of the Case Study Structural Models**

Upon the recommendations of ERRA, till now, around 120,000 housing units are constructed with this structural scheme [Schacher 2010]. The construction guidelines recommended the use of Dhajji up to two storeys [Schacher 2010], however most of the current practices adopt single storey housing unit with interstorey height of 3m. The housing units are square and rectangular provided with four/two slopes G.I. sheet roofing. The prevailing building dimensions ranges from 4.60m×4.60m to 13.80m×4.60m. In case of the rectangular configuration, the in-plane walls in the longer direction are provided either with the transverse walls or cross ring beams at 4.60m interval in order to provide in-plane integrity to the building. All the in-plane walls are made of Dhajji configuration with main vertical timber
posts, 100mmx100mm, provided at each 1.50m and main horizontal posts, 100mmx100mm, provided at 2.40m which are provided with vertical and horizontal timber posts, 50mmx100mm, at each 740mm and 570mm respectively. The walls are then braced with timber posts of thickness 25mmx100mm. Each of the walls are provided with rubble stone mud masonry infill in the ratio of 9:1 to 7:3 which is then plastered using mud coating. The in-plane walls are provided with doors, one to two, and windows, one to four, of typical dimensions of 1.10mx2.15m and 1.20mx1.20m respectively. The building rests on shallow strip type footing of width 600mm, 600mm deep with additional 300mm of plinth level, with stepped stone work overlain compacted earth to which the bottom beam of the walls are connected, see Figure 2. The buildings are provided with 300mm deep floor pad with stone masonry work and smooth finishing.

**Dynamic Analysis of Cases Study Structural Models**

The present study designed twelve 2D structural models of single storey with the above characteristics and three cases for frame element strength: maximum positive, maximum negative and mean strength observed in the experiment, thus making up thirty-six cases for which MDOF shear type models were generated. Each of the frame elements of the structural models were provided with a nonlinear trilinear hysteretic pinching rule calibrated in the previous section. The increase in the length of wall i.e. the provision of additional bays, is due considered with the increase in strength, assuming the fact of shear type mechanism and soft-storey phenomenon in which each of the small bay contributed equally to the strength, while keeping the same drift limits. Also, the strength is reduced upon the inclusion of doors and windows, as per number of small bays excluded.

All the case study models were analyzed dynamically using NLTHA with soft soil spectrum compatible natural accelerograms which were linearly scaled in order to observe the post-yield response of all models, to derive static SDOF systems for the considered building typology and retrieve the dynamic characteristics of the considered buildings. The equivalent base shear and equivalent displacement demand at the crack limit state of models were obtained using the proposed SDOF derivation of Ahmad [2010d], which is used to obtain the secant vibration periods of structural models:

\[
T_y = 2\pi \sqrt{\frac{\Delta_{eq}}{V_{B,eq}}}
\]  

where \(\Delta_{eq}\) represents the equivalent displacement and \(V_{B,eq}\) represents the equivalent base shear obtained from the normalization of the sub-storeys displacements and base shear over the deformed shape and seismic mass participation of the model [Ahmad 2010d], see Figure 4. The periods obtained for the cases study buildings are used to develop the empirical period model [5] for future applications:

\[
T_y = a.exp(\pm \varepsilon \beta) H^p
\]
where $H$ represents the total height of the building; $a$ and $b$ are the coefficients obtained through regression analysis; $\beta$ represents the variability in the period computation. The period coefficient with $b$ set to 0.75, which is relatively well correlated than $b$ equal to 1, obtained for all the structural models, 360 cases, is shown in Figure 7 (left). The displacement coefficients, $k_y$: at cracking/yielding limit state and $k_p$: for post cracking limit states; see [2], obtained for all structural models are also shown in Figure 7.

**Figure 7** Mean Period Coefficient (Left) and Displacement Coefficients, $k_y$ and $k_p$, (Middle and Right) for Considered Dhajji Wall Buildings.

### Derivation of Displacement-Based Analytical Fragility Functions

The state-of-the-art, conceptual and explicit, in contrary to the existing conventional procedures e.g. HAZUS [1998] among others, procedure proposed by Ahmad [2010e] for the derivation of analytical fragility functions is used to derive fragility functions for Dhajji buildings of Pakistan. In the first step, Controlled Monte Carlo simulation is used to generate thousands of SDOF systems with different limit states mechanical properties using the developed models for periods and displacement capacity, respecting the regional variability in the geometric and material properties as well as the variability introduced by earthquake loading. Variability in the drift limits and viscous damping, observed in experiment, is also considered using complete lognormal probabilistic distribution. In the second step, linear displacement response spectra were generated which were used to obtain the limit states probability of exceedance for the considered building typologies. Also, each of the generated spectrum was analyzed with the median mechanical properties to obtain the inelastic displacement demand on the system. For a given spectrum, the number of buildings exceeding different limit states is plotted against the displacement demand on that system in order to derive analytical fragility functions. The fragility functions were used to derive damage probability matrices, considering soft soil, for three case study earthquakes with $M_W$ 8.0 and fault rupture distance of 30 (S1), 20 (S2), & 10 (S3) km respectively. Each earthquake was repeated 10,000 times, taking into account the ground motion uncertainties, to simulate the site ground motions using the empirical ground motion prediction equation of NGA [BA08 2008]. The mean DPM derived for the considered earthquakes, see Figure 8, shows that this structural system needs to be retrofitted in case of high ground motions: 3.23% for S1; 6.24% for S2 and 12.62% for S3 which are important from the economic loss stand point.
Conclusions

The paper presents calibration, using experimental and numerical investigations, of nonlinear static analytical displacement-based methods for seismic performance evaluation of Dhajji wall structures. An efficient and simplified method was developed for the nonlinear dynamic seismic analysis of Dhajji structural systems using equivalent frame approach. Three full scale Dhajji walls typical of current construction practice in Pakistan were tested quasi-statically to develop the static and dynamic assessment methodologies. Numerical investigation of thirty-six case study 2D structural models was performed using NLTHA with natural accelerograms in order to derive mechanical models for the considered building typology taking into account the geometric and material uncertainties as well the record-to-record variability in the capacity evaluation. Controlled Monte Carlo simulation was used to generate random populations of regional building stock and to develop analytical fragility functions which can be used for seismic risk map development, regional code development, design of insurance schemes, earthquake preparedness and planning and decision making in the region. The present study shows that this structural system may need to be retrofitted in case of nearby high magnitude earthquakes. Calibration is required with more experimental tests to further improve the mechanical models derived herein. Assessment of the static procedures, developed herein, through NLTHA is required to improve the confidence on the future applications of the methodologies. Comparative vulnerability study is required in the region to investigate the relative performance of different regional structural systems.

References


