International Journal of Civil Structural Environmental And Infrastructure Engineering Research Vol.1, Issue.1 (2011) 1-15
Vol.1, Issue.1 (2011) 42-85

SEISMIC STRUCTURAL DESIGN CODES EVOLUTION IN PAKISTAN AND CRITICAL INVESTIGATION OF MASONRY STRUCTURES FOR SEISMIC DESIGN RECOMMENDATIONS

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ABSTRACT

The paper presents the historical background on the evolution of seismic design codes for structures in Pakistan. The current seismic design code do not provide information and detailing on the design and assessment of ordinary masonry structures in low to moderate seismicity regions, which is found the most in the country. This paper thus presents the investigation on ordinary masonry structures, yet respecting the minimum requirements to ensure significant good performance, representing current field practice through nonlinear time history analysis. The study aim to develop simplified tools and guidelines for the design of masonry structures in low to moderate seismicity regions using static procedures and hand calculations. Also, significant modifications required in the current seismic design code of Pakistan is highlighted for future development. The findings herein can also provide an opportunity to other researchers for investigation of masonry structures in the other parts of the world.

Key words: building codes of Pakistan; masonry structure design; response modification factor; R factor; seismic design.

1. INTRODUCTION

Pakistan has a long history of catastrophic earthquake events due to the obvious reason of the collision of Indian plate with the Eurasian plate (Chandra 1992) that caused the loss of life and complete devastation of historic towns, on average the country can possibly experience a damaging earthquake every ten years (Ahmad 2011), however little effort has been made since in reducing the earthquake disasters in Pakistan. The drastic consequences of all earthquakes in Pakistan are due to the underestimation of ground motions from the expected earthquakes, as experienced in the recent past, due to the high vulnerability of structures and their inhabitants and the lack of well planned preparedness activities in Pakistan (Naseer et al. 2010, Rossetto & Peiris 2009, ADB-WB 2005, Khan 2007). The future disasters from earthquakes and the loss of life can be reduced through well designed/retrofitted structures that can respond to large earthquakes without total collapse though with significant irreparable damage.

Till the recent past no official document was enforced by the Government for the design of structures and infrastructures explicitly considering the seismicity of the region and the design of structural systems against the expected ground shaking. However, even from the early time, structures meeting the minimum requirements to ensure resistance against lateral load and/or designed with modest efforts has performed significantly well and ensured life safety during large earthquakes (Kumar 1933, Jain & Nigan 2000, Jackson 1960, Ali 2007). On the other hand some old traditional structures made of wooden frame or wooden laced masonry structures has performed extremely well (Ambraseys et al. 1975, Mumtaz et al. 2008, Schacher & Ali 2008, Langenbach 2010). The recent seismic provisioned building code of Pakistan didn’t considered any of the above structure type that performed satisfactorily or alternate affordable structures schemes with essential earthquake resistance, since all parts of the country are not subjected to high or extreme hazard level, but rather documented the design and detailing of advanced engineered structures (BCP 2007). Recent experiences has shown that ordinary masonry structures designed to meet the minimum requirements of earthquake resistant structures can escape the total structural collapse and consequently fulfill the objective of life safety during design level earthquakes (Magenes, 2006). Also, affordable retrofitting techniques, for example floor stiffening can make the masonry structure resist ground motions up to peak ground acceleration of 0.70g without collapse (Magenes 2010a, Magenes 2010b). All the above confirm that there is a need for the investigation of ordinary masonry structures, yet confirming the minimum requirements to offer lateral resistance through global in-plane response and avoid the local out-of-plane failure of walls, in order to develop simplified rules for the design and verification of these structures in low to moderate seismicity regions.

This paper thus investigate low-rise brick masonry structures designed with the actual material properties reflecting the field practice in the urban areas of Pakistan. Recent experimental studies carried out on brick masonry at the Earthquake Engineering Center of Peshawar at section level i.e. brick units, mortar and masonry assemblages, and member level i.e. lateral quasi-static cyclic testing of full scale masonry walls, are considered for the design of prototype mathematical models for numerical investigation. Forty-nine case study structures are considered which are analyzed using nonlinear static pushover analysis and nonlinear dynamic time history analysis. Response modification factor R used in static seismic design procedures is estimated and presented for future applications. The computation of R factor through nonlinear time history analysis is performed in order to truly consider the hysteretic energy dissipation of the considered masonry material. Finally, recommendations are made for the minimum requirements of geometrical specification and detailing of masonry structures in different seismic zones of the country which can ensure life safety during design level earthquakes under the considered situations. Further experimental investigations can improve the findings provided herein which can in turn increase the confidence in future applications of the given recommendations.

1.1 Seismic Design Of Structures to Building Codes of Pakistan

1.1.1 Brief historical background of building codes for seismic design in Pakistan

Perhaps the very first initiative towards the development of earthquake resistant design guidelines in Pakistan is set forth soon after the 1931 Mach earthquake M 7.4 in Balochistan, epicenter within 60 km of Quetta city, Pakistan (Kumar 2933). The earthquake ruined adobe structures built with sun dried bricks in mud mortar while severely damaged structures built with fired bricks in mud mortar. Structures built with fired bricks in lime mortar having C. I. sheeting and steel roof trusses received no serious damage while two blocks of menial's quarters built with fired brick in cement mortar withstood the shock successfully34.

A detailed document was pioneered by Kumar, a young railway engineer, which besides presenting a general theory and concept of earthquake resistant design, included the first seismic zoning map for India (including Pakistan) and specification of seismic coefficients for different areas subjected to different level of ground excitations considering two classes of structures, A (including monumental buildings and other taller than 15 m) and B (all other structures), for which the seismic coefficient specified were 0.15g/0.10g in areas of violent earthquakes (High hazard), 0.10g/0.075g in areas of strong earthquakes (Moderate hazard), 0.05g/Nil in areas of weak earthquake (Low Hazard), Nil/Nil in areas having rare earthquake (Negligible hazard), respectively. The base shear demand on the structure is computed by multiplying the specified coefficient with the total mass of the structures. This code was practiced at that time in Quetta for the construction of iron-rail (second-hand) frame structures quarters with brick masonry infill, in cement mortar, and roof trusses for the Railway Department. In 1935 the Quetta city was subjected to a large earthquake of magnitude 7.6 which devastated the historic town of Quetta, destroying almost every building, resulting in 60,000 fatalities (Jackson 1960). The buildings designed to the recommendations of

Kumar were the only structures in the town that remained undamaged (Jain & Nigan 2000).

Following the 1935 Quetta earthquake, a new building code for seismic provision was developed in 1937 (QBC-1937) by the British Government under the guidance and supervision of Taylor[1] during the British Raj in India (QBC 1937), largely influenced by the successful demonstration of Kumar for the earthquake resistant construction. This code comprised of general regulations specifying the appropriate shape, height and spacing, and materials for eight type of structures mainly including two types of steel frame structures with brick masonry infill panels provided with reinforced concrete floors and roof or iron-sheet roofing, three different types of timber structures, two types of brick masonry structures (constructed in Quetta bond), ref. Spence & Cook (1983), with reinforced concrete ring beams and reinforced concrete floors, and structures with rough ballie framework with Pise work (Naseer et al. 2010). Unreinforced masonry without ring beams and vertical reinforcement and reinforced concrete frame structures were not recommended for the reconstruction. The buildings constructed in Quetta city according to the recommendations of QBC-1937 performed again extremely well during 7.1 magnitude earthquake in 1941 (Jain 2008). However, the QBC-1937 was enforced throughout the Quetta municipality only for a time and the control on new construction in the city was lost then following the independence in 1947 and the rapid population growth in the city (Coburn & Spence 2002). Also this code was never extended to other municipalities in Pakistan.

Later in 1986, an official building code for seismic provisions (BCP 1986) is prepared in Pakistan but which is mainly based on the UBC-82 recommendations and guidelines. The code also included seismic zoning map for Pakistan, five seismic zones were considered, specifying design ground motion levels in terms of felt intensity for all the municipalities which was mainly based on the collected instrumental seismicity data and the observed response of ordinary structures (excluding nuclear power plant, dams, and other critical facilities) for the period of 1905 to 1979. The BCP-1986 defines a static lateral force procedure for determining the seismic actions on ordinary structures using the following formula:

$$V = ZIKCSW \text{ where } CS \leq 0.14 \qquad 1$$

where V represents the base shear force; Z represents the zone factor, equal to 0.094 for Zone 0, 0.1875 for Zone 1, 0.375 for Zone 2, 0.75 for Zone 3, 1 for near field condition; I represents the occupancy importance factor which is 1.5 for essential facilities (hospitals, fire and police stations), 1.25 for facilities assembling more than 300 people and 1.0 for all other structures; K represents the horizontal force factor depending on the type of basic structural system which is 0.8 for dual bracing systems, 0.67 for ductile moment resisting frames, and 1.0 for all other frame structures; C represents the base shear coefficient which defines the normalized force at the fundamental period of vibration of structure; S represents a numerical coefficient for site-structure resonance; W represents the total dead load i.e. the weight of the structure including the partition loading and 25 percent of live load in case of storage and warehouse occupancy. The coefficient C in the above equation is determined using the following formula:

$$C = \frac{1}{15\sqrt{T}} \text{ where } T = \frac{0.05h_n}{\sqrt{D}} \text{ or } T = 0.1N \text{ and } C \leq 0.12 \qquad 2$$

where T represents the fundamental period of vibration of the structure; h_n represents the total height of the structure above the ground level; D represent the length of the structure plan in the direction of loading; N represents the number of stories of the structure. The coefficient S in equation 1 is always equal or greater than unity and is computed relating the fundamental period of vibration with that of the site characteristic period.

$$S = 1.0 + \frac{T}{T_S} - 0.5\left(\frac{T}{T_S}\right)^2 \quad \text{for} \quad \frac{T}{T_S} \leq 1.0 \qquad 3$$

$$S = 1.2 + 0.6\frac{T}{T_S} - 0.3\left(\frac{T}{T_S}\right)^2 \quad \text{for} \quad \frac{T}{T_S} > 1.0 \qquad 4$$

where T_S represents the fundamental period of site-soil which need to be properly established for the site. The value of T in the above equation should not be less than 0.30 sec and in case if T exceeds 2.50 sec, the TS value is considered as 2.50sec to compute S. The coefficient S takes the value of 1.5 (maximum) when TS is not properly established for a site.

The base shear obtained then using Eq. (1) is distributed along the height of the structure considering linear deformed shape of the structures using the following formula:

$$V = F_t + \sum_{i=1}^{n} F_i \quad \text{where} \quad F_i = \frac{(V - F_t)W_i h_i}{\sum_{x=1}^{n} W_x h_x} \qquad 5$$

where n represents the total number of floors of the structure; i represents a particular floor level; F_i represents the horizontal load applied at storey i; W_i and h_i represent the effective seismic weight and height of storey i; F_t represents additional load applied at the top floor of structure, 7 percent of the total base shear times T but always less than 25 percent of total base shear, for the case of T greater than 0.70 sec in order to take into account the higher mode effects in flexible structures.

The recent experience of 2005 Kashmir earthquake (ADB-WB 2005) has shown that the zoning map of BCP-1986 underestimated the felt intensity in all the major cities mostly affected in the earthquake (Naseer et al. 2010). This code did not provide additional safety through importance factor to school buildings which suffered the most in Kashmir earthquake. Also, the code did not give a clear details on the seismic resistant design of reinforced concrete and masonry buildings, which is found the most in the country. However this code was never enforced in Pakistan because it was not adopted as Governmental regulations but rather used occasionally and for special structures. Additional details on the early history of seismic building codes in Pakistan and their relative comparison with other regional codes can be found elsewhere (Naseer et al. 2010, Rossetto & Peiris 2009).

1.2 Building Code of Pakistan: Seismic provisions – 2007 (BCP-2007)

1.2.1.Brief on development and adoption of BCP-2007 in Pakistan

Pakistan has a long history of catastrophic earthquake events that caused the loss of life and complete devastation of historic towns, on average the country can possibly experience a damaging earthquake every ten years (Ahmad 2011), however little effort has been made since in reducing the earthquake disasters in Pakistan through well defined seismicity of the country and proper seismic design of structures. The high vulnerability of structures in Pakistan is proved again in the 2005 Kashmir earthquake that devastated many major cities causing 80,000 fatalities and overall economic loss of about 6 billion USD (ADB-WB 2005). In this earthquake the structures damaged were mostly consist of stone masonry, block masonry, reinforced concrete and brick masonry (these structure types are listed according to their observed vulnerability, from maximum to minimum vulnerability) (Naseer et al. 2010, Naeem et al. 2005, Ali 2010, Peiris et al 2008) among others. Few wooden structures were found in the region which performed extremely well (Mumtaz et al. 2008, Schacher & Ali 2008). The Ministry of Housing and Works engaged the National Engineering Services of Pakistan (NESPAK) to develop revised seismic zoning maps for the country and design criterion for reconstruction of structures in the region. NESPAK approached the International Code Council and a core Group of expert individual from across the country to help Pakistan develop earthquake provisions to save lives and reduce property losses (Shabbir & Ilyas 2007).

The BCP-2007 included a new seismic zoning map, five zones are considered, for the design level ground motions in terms of peak ground acceleration corresponding to earthquake motion which has 10 percent probability of exceedance once in every 50 years and 500 years of return period for the design level earthquake. The minimum requirements and detailing specified for all different structural systems (excluding active/passive controlled structures) for the design level earthquake are based on the recommendations of foreign national codes and guidelines mainly the uniform building code UBC-97, American Concrete Institute Code (ACI 318 – 2005), American Institute of Steel Construction ANSI/AISC 341 – 05, and American Society of Civil Engineers, SEI/ASCE 7 – 05 and ANSI/ASCE 7 – 1993. The purpose of the code provisions is to avoid the structural failures and loss of life but not to limit damage or maintain functioning following design level earthquake. The Prime Minister of Pakistan declared that the Pakistan Engineering Council would enforce and publicize the revised builders parameters based on BCP-2007 for the reconstruction and new design of structures in the country. Current detailed studies on seismic hazard analysis has shown that BCP-2007 may underestimate the design level ground motions for some major cities in the country, (PMD 2007, Zaman & Warnitchai 2010 among others). However, it was proposed that the code needs to be revised for possible changes and development every three years which should have a revised edition by this year.

1.2.2 Seismic design of structures to BCP-2007

The BCP-2007 included a static lateral force procedure for determining the seismic actions on structure in a given direction using the following formula:

$$0.11 C_a I W \leq V = \frac{C_v I}{RT} W \leq \frac{2.5 C_a I}{R} W$$

$$\frac{0.8 Z N_v I}{R} W \leq V = \frac{C_v I}{RT} W ; \text{ for Zone 4 only} \qquad 6$$

where V represents the total base shear force; C_v represents the seismic coefficient which depends on the seismic zone and type of site soil and takes the value of 0.18 for Zone 1, 0.32 for Zone 2A, 0.40 for Zone 2B, 0.54 for Zone 3 and $0.64N_v$ for Zone 4 considering type D soil of NEHRP soil classification (soil type D is recommended in case of unavailability of detailed and reliable data on the site soil); C_a and C_v represent seismic response coefficients depending on the seismic zones and site soil condition which are 0.12/0.18 for Zone 1, 0.22/0.32 for Zone 2A, 0.28/0.40 for Zone 2B, 0.36/0.54 for Zone 3 and $0.44N_v/0.64N_v$ for Zone 4 respectively considering type D soil; N_v represents the near source factor depending on the type of seismic source and source-to-site R_{jb} which ranges from 1.0 to 2.0 for R_{jb} less than 2 km, 1.0 to 1.60 for R_{jb} equal to 5 km, 1.0 to 1.20 for R_{jb} equal to 10 km and 1.0 for R_{jb} greater than 15 km for low activity to high active fault respectively; I represents the importance factor which is 1.25 for essential and hazardous facility while 1.0 for all other structures; W represents the total weight of the structure; R represents the response modification factor; T represents the fundamental vibration period of the structure.

The R factor in the code takes into account the inherent over strength and global ductility capacity of lateral force-resisting systems. It is worth to mention that the importance of energy dissipation capacity due to nonlinear behavior of structure is not considered by the code but rather it is referred to the ductility factor computed using the energy balance rule modified by the structural over strength. It has been demonstrated that structural systems with similar stiffness and strength can result in different inelastic deformation demand, and hence different performance state, considering different nonlinear hysteretic response, assigning different level of energy dissipation capacity to the systems (Priestley et al. 2007). The vibration period of structure which is computed using the following equation:

$$T = C_t h_n^{0.75} \tag{7}$$

where h_n represents the total height of structure; C_t represents a numerical coefficient which is 0.0853 for steel moment-resisting frame structures, 0.0731 for reinforced concrete moment-resisting frame structures and eccentrically braced frame structures and 0.0488 for all other structures. Alternatively it can be also computed for concrete and masonry shear wall structures:

$$C_t = \frac{0.0743}{\sqrt{A_c}} \text{ where } A_c = \sum_{i=1}^{NW} A_e \left[0.20 + \left(\frac{D_e}{h_n}\right)\right] \text{ and } \frac{D_e}{h_n} \leq 0.9 \qquad 8$$

where D_e represents the shear wall in the first storey in the direction parallel to the applied forces; A_e represent the minimum cross-sectional area in any horizontal plane in the first storey of a shear wall; NW represents the total number of shear walls in first storey; A_c represent the combined effective area of the shear walls in the first storey of the structure. However recent investigation for the vibration period of existing masonry-infill reinforced concrete structures, considering 50 percent crack stiffness of concrete element, has shown that the above equation underestimate the period by almost 40 percent (Ahmad et al. 2011a) a similar deficiency also pointed out for other seismic codes, e.g. (Goel & Chopra 19998, Crowley & Pinho 2010, Crowley & Pinho 2004) among others. The total base shear thus obtained for a given structural system is essential distributed along the height of structure in a similar fashion of BCP-1986 procedure using linear deformed shape of the structure.

The code specified general recommendations for masonry structures regarding the requirements for minimum compressive strength of masonry units, 8.25 MPa for solid fired-clay bricks and 5.5 MPa for solid/hollow concrete blocks, mix proportions and compressive strength of mortar, 4.10 MPa, and compressive strength of masonry. Limitations on the number and height of stories and minimum thickness of walls have also been specified. Because of the poor performance of stone masonry buildings in the Kashmir earthquake, use of rubble stone masonry is prohibited. In order to increase the integrity and to ensure better performance of unreinforced masonry buildings, use of reinforced concrete bond beams has been made mandatory at the plinth, door, and roof levels. Confined masonry has been introduced and detailed provisions have been specified in the code keeping in view its good performance in other parts of the world.

The code didn't provide guidance on the design and construction of affordable houses which are utmost important considering the scenario of Pakistan where 30 to 50 millions of people live below the poverty line (under 1 USD a day). Despite the fact that the masonry structures that fulfilled the minimum requirements to ensure global response has performed well in the past (Kumar 1933) as well as in the recent (Ali 2007) large earthquakes in Pakistan and met to the code requirements for ensuring life safety. Some parts of the country has low to moderate seismicity where the design of ordinary masonry structures can be promoted but essentially fulfilling the minimum requirements (Magenes 2006) to ensure global in-plane response of structures during earthquakes. On other hand, the floor stiffening of an unreinforced masonry structures, to ensure in-plane response, can resist earthquake with ground acceleration even up to 0.70g without total collapse, 43(Magenes 2010a, 2010b).

1.3 RESPONSE MODIFICATION FACTOR OF STRUCTURES

The response modification factor is the coefficient used to reduce the elastic spectral ordinates for the safety verification of structures against collapse in design level earthquake. The R factor is an approximation to the ratio of the seismic forces that the structure would experience if its response was completely elastic in the design earthquake (typically defined by the 5% damped elastic response spectrum for static equivalent lateral force procedure), to the minimum seismic force that may be used in the design, with a conventional elastic analysis model (elastic dynamic analysis of structure), still ensuring a satisfactory response of the structure i.e. the ultimate deformation capacity is not exceeded which in turn then avoid the collapse of structure during design ground motions (Miranda 1997), formulated as follow.

$$R = \frac{F_e}{F_d} \qquad 9$$

where F_e represents the elastic force demand for the structure deemed to respond elastically to the earthquake, usually obtained using elastic structural model and elastic dynamic analysis or statically, using the acceleration response spectrum to define the spectral acceleration at the fundamental vibration period of the structure; F_d represents the minimum design strength of structures whereby the designed structure can escape the total collapse for the considered design earthquake. Generally, the R factor mainly depends on the ductility capacity of the structure (which relates to the detailing of the structural members), on the strength reserves that normally exist in a structure (depending mainly on its redundancy and on the overstrength of individual members), and on the (effective) damping of the structure; all these factors directly affect the energy dissipation capacity of a structure (Kappos 1999)28, whereby the R factor can be formulated as given below.

$$R = R_{\mu} R_s R_{\xi} \qquad 10$$

where R_{μ} represents the ductility dependent component that considers the hysteretic energy dissipation of the structure; R_s represents the over strength dependent component, it is the ratio of the maximum strength of the system to the minimum design force, also called as the overstrength ratio OSR (Magenes, 2004)40; R_{ξ} represents the damping dependent component in the case of structures with supplemental damping devices and is not thus discussed herein. The ductility dependent component can be obtained best through nonlinear dynamic time history analysis, considering accurate nonlinear hysteretic response of the structural components e.g. the true cyclic response of structural elements (Porto et al. 2009).

Alternatively, the energy balance criterion and the classical analytical model, is used to compute $R\mu$ for masonry structures however this approach does not differentiate the reduction factor for systems having similar strength but distinct hysteretic response; which can significantly affect the seismic demand and the expected performance of structures during earthquakes (Priestley et al. 2007). The system over strength dependent Rs factor can be best obtained carrying out nonlinear static pushover analysis of the structure in order to estimate the maximum lateral strength of the system and the force corresponding to the crisis of the first wall i.e. the force at which the first wall attain its capacity (Magenes & Morandi 2008, Morandi & Magenes 2008). However the estimation of Rs is not very straight forward and is highly influenced by the actual structural configuration, distribution of seismic forces in the plan of structures i.e. to individual walls, in-plane rigidity/flexibility of structure floors and the means of wall coupling (through spandrels, ring beams or both, tie rods, etc) among others. These factors clearly indicate that experimental investigation alone cannot give an estimate of R factor in general and the use of numerical techniques along with experimental findings can be best used thereof.

4. ESTIMATION OF R FACTOR FOR MASONRY STRUCTURES

4.1 Characteristics of the case study structures

The out-of-plane collapse modes of an unreinforced masonry structure can be avoided by the presence of certain simple, yet critical features, given below, as set forth by Magenes based on many experimental and real observations of masonry structures in European countries (Magenes 2006). In theory, the presence of these simple measures in a masonry structure can lower the out-of-plane failure of masonry walls which in turn ensure in-plane global response of masonry structures.

- The quality of the load-bearing walls (masonry unit, mortar, interlocking of units, regularity of courses) to facilitate monolithic behavior right through the wall thickness.
- Restricted slenderness of the walls to ensure out-of-plane stability.

- Presence of efficient connections amongst walls (good interlocking at wall intersections), and between walls and horizontal structure (tie-rods, ring beams at all roof/floors) to ensure box-action.
- Adequately rigid and resistant floor diaphragm providing restraints to out-of-plane vibration of walls, increasing structural redundancy and facilitating internal force redistribution.
- Presence of suitable elements such as ties, floor diaphragm, etc., or availability of in-plane resistance of strong walls or buttresses to counteract horizontal thrusts (from roofs, arches, or vaults) to form a closed self equilibrium system.

Unreinforced brick masonry structures are practiced abundantly in the northern urban areas of Pakistan by people with high income (Ali 2007). Brick masonry are used mostly in the form of low-rise structures with number of storeys varying from one to three and provided with rc floors and roof. The structures are also provided with ring beams and lintel/plinth level bands, which restrict the wall slenderness, in order to ensure in-plane integrity of structures during earthquake excitations. The most prevailing building dimensions ranges from 9.15 m × 18.30 m (5 Marla) and 18.30 m × 27.40 m (10 Marla) constructed in cement sand Khaka mortar with typical wall density, the ratio of the cross sectional area of the in-plane walls to the total floor area, ranging from 1.5% to 7%. These structures are largely practiced with 230mm thick load bearing walls of solid fired bricks, with dimensions of 230mm×115mm×65 mm, arranged in orthogonal planes having proper connections at the corners and wall junctions through brick toothing of English bond type arrangement, provided with 150mm thick rc rigid floors having clear storey height and ground floor height of 3.0m and 3.5m respectively. The structures are provided with shallow strip type footing, with stepped brick work overlain compacted earth (sub grade). The load bearing walls, 70% of the total wall length, are perforated by doors, with typical dimensions of 1.2 to 1.8 square meters and 2.2 to 2.5 square meters respectively. The primary seismic resistance mechanism for this configuration, as observed in the dynamic test (Ali & Naeem 2007) and earthquake observation (Naseer et al. 2010, Peiris et al. 2008) is in-plane global mechanism with shear damage of masonry walls. The present study is thus limited to the above mentioned residential unreinforced and unconfined masonry structures which is equally valid for very lightly reinforced masonry for which there is no significant difference in seismic behavior from its unreinforced counterpart.

4.2 Prototype cases study structural models

4.2.1 Design of prototype structures

The present study considered forty-nine case study two storey structures designed to meet the geometric and material characteristics of brick masonry structures in Pakistan. Due to unavailability of detailed and reliable data on the structural configuration, prototype of structures are designed essentially compatible to field practice with varying wall density and floor area. The mechanical characteristics of brick masonry material i.e. unit and mortar, and masonry walls is obtained from the recent investigation on these structure type at the Earthquake Engineering Center of Peshawar (Ali & Naeem 2007, Javed 2008). The material properties mainly compressive strength, tensile strength, elastic moduli, etc are considered as the characteristic estimate from the experimental values. Table 1 shows the characteristics of cases study structures analyzed in the present study. A total of forty-nine combinations are made for the considered wall density and floor areas. Other material properties considered are compressive strength 3500 (kN/m^2), masonry tensile strength 105 (kN/m^2), modulus of elasticity E 1225 Mpa, shear moduli 490 Mpa and specific weight of masonry γ 18 (kN/m^3)

Table 1. Characteristics of case study structures analyzed for R factor computations

Wall Density (%)	1.98	2.55	3.27	4.2	5.39	6.11	6.92
Floor Area (m2)	68.91	85.87	107	133.33	166.14	185.46	207.02

4.2.2 Mathematical modelling of masonry structures

The method used for the mathematical modelling and nonlinear dynamic time history analysis (NLTHA) of masonry structures in the present study is based on the equivalent frame idealization of masonry walls as proposed earlier by other researchers and experts (Magenes & Fontana 1998) for the global performance evaluation of masonry structures. The present study formulated the method with simplified constitutive laws, developed through quasi-static cyclic tests on masonry walls, for frame elements representing piers and spandrels which can be applied to any type of masonry structures i.e. unreinforced, reinforced and confined, with regular openings and rigid or flexible floors. The method requires an appropriate strength evaluation models for a given masonry typology.

The method uses the idea of modelling masonry spandrels and piers as one dimensional beam-column elements with bending and shear deformation with infinitely stiff joint element offsets at the ends of the pier and spandrel elements,

Figure 1. The method has been developed and employed for many different types of masonry structures within the context of seismic vulnerability and risk assessment essentially considering global mechanisms. A similar type of modelling approach, constitutive law being defined at the section level, is also used by Kappos (Kappos et al. 2002, Kappos 2007) for the global vulnerability assessment of unreinforced masonry structures in European countries. Recently, the method is also employed for the global vulnerability assessment of masonry structures in Euro-Mediterranean regions and South-Asia countries for the derivation of seismic capacity curves and fragility functions within the context of global earthquake analysis for prompt response (Ahmad et al. 2011b, 2011c, 2010).

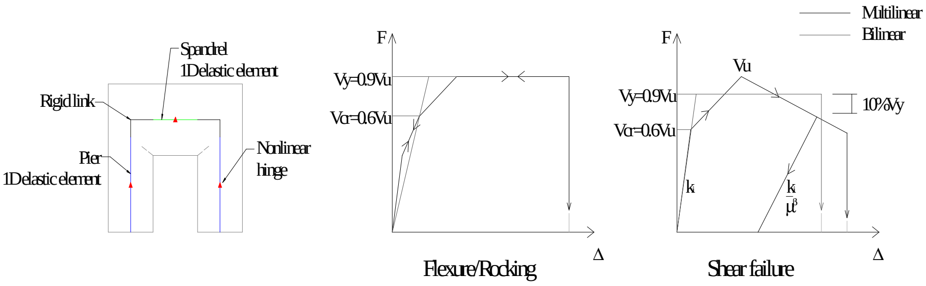


Figure 1. Equivalent frame method. *From left to right*: equivalent frame idealization of masonry structural wall and nonlinear force-displacement response of frame element, considering either multilinear or bilinear behavior (Ahmad et al. 2010).

Following the comparison of available shear strength models for masonry walls against the quasi-static cyclic tests on full scale walls (Ahmad et al. 2011b), the following strength models is found to provide best estimate of masonry shear for the case study masonry structures, due to the prevailing mechanism of considered structures. It is worth to mention that other recommended masonry shear strength models for rocking mechanism and shear sliding mechanisms are also used, however the following model always resulted in lower shear capacity which is considered conservatively.

$$V_d = \frac{f_{tu} Dt}{b} \sqrt{1 + \frac{p}{f_{tu}}} \qquad 11$$

where f_{tu} represents the principal tensile strength, also called diagonal tensile strength; p represents the mean normal stress on wall; D represents the wall length; t represents the thickness of the wall; b represents the coefficient used to approximate the complex shear stress distribution in masonry wall; V_d represents the lateral load carrying capacity of the wall for the considered mechanism.

The coefficient b varies with the aspect ratio of the pier. A simple criterion is proposed (Benedetti & Tomazevic 1984) to evaluate b: b=1.5 for $H/D \geq 1.5$, b=1.0 for $H/D \leq 1.0$ and b=H/D for $1< H/D <1.5$. The diagonal tensile strength of masonry is a global strength parameter and can be obtained performing diagonal shear test on masonry panel by knowing the maximum vertical load at the onset of diagonal cracks in the panel. The diagonal tensile strength is then computed using Eq. (12):

$$f_{tu} = \frac{0.5N}{t(l_1 + l_2)} \qquad 12$$

where N represents the ultimate vertical load at the onset of diagonal cracking in the panel; t represents the thickness of the panel; l1 and l2 are the length of the two respective sides of the panel.

The above equation is an approximation of the model proposed by frocht (1931) and which is also recommended by RILEM (1994), where the constant value of 0.52 (in the original proposal) is replaced conservatively by 0.50. Alternatively, f_{tu} can be obtained from the maximum shear strength of masonry wall, when the wall fails due to the formation of diagonal inclined cracks in the wall using Eq. (13) and/or from the compressive strength of masonry using Eq. (14), empirical rule proposed on the bases of large sets of experimental results by Tomazevic (1999).

$$f_{tu} = b\,\tau_{max} \qquad 13$$

$$0.03f_m \leq f_{tu} \leq 0.09f_m \qquad 14$$

where τmax represents the shear stress obtained when the principal stress at the center of the pier reaches the tensile strength of masonry; fm represents the compressive strength of masonry.

For all the frame elements the yield force is considered to be 90% of the value estimated using the above strength models based on the experts recommendations (Magenes & Calvi 1997) for the possible bi-linearization of non-linear capacity curves for masonry walls. The analysis of masonry structures is performed in OpenSees (McKenna et al. 2008). The beam-column element used for masonry idealization is completely defined by masonry Young modulus, shear modulus, wall sectional area and the wall moment of inertia. The hinge, defined through zerolength element and hysteretic material assigned with nonlinear force-displacement constitutive law to simulate the lateral capacity of the masonry wall.

The nonlinear behavior, force-displacement response of the frame element is idealized as elastic-perfectly-plastic for the masonry piers as per recommendation by other researchers and used also elsewhere (Magenes 2000) for nonlinear pushover analysis and NLTHA (Menon & Magenes 2011) with a limited deformation capacity at which the strength of the element goes to zero (defining the failure of element). It is worth to mention that due to the provision of rc slab, ring beam and lintel bands the horizontal coupling elements (spandrels) do not yield for the case study structures and is thus considered as elastic. The present study considered the force-displacement constitutive law for frame elements (walls) with Takeda-Type rule of Otani (1974) having Emori (1978) type of unloading, see Figure 2.

where F_y represents the yield strength obtained using the above strength model for masonry walls, reduced by 10 percent; k_i represents the initial 50% crack stiffness, (k_s + k_f); k_s represents shear stiffness, k_f represents flexure stiffness; Δ_y represents the idealized yield displacement, (Δ_{yh}+Δ_{yf}); Δ_{yh} represents the shear deformation, simulated through shear hinge; Δ_{yf} represents the flexure deformation, simulated through flexure element; Δ_u represents the ultimate displacement at element failure.

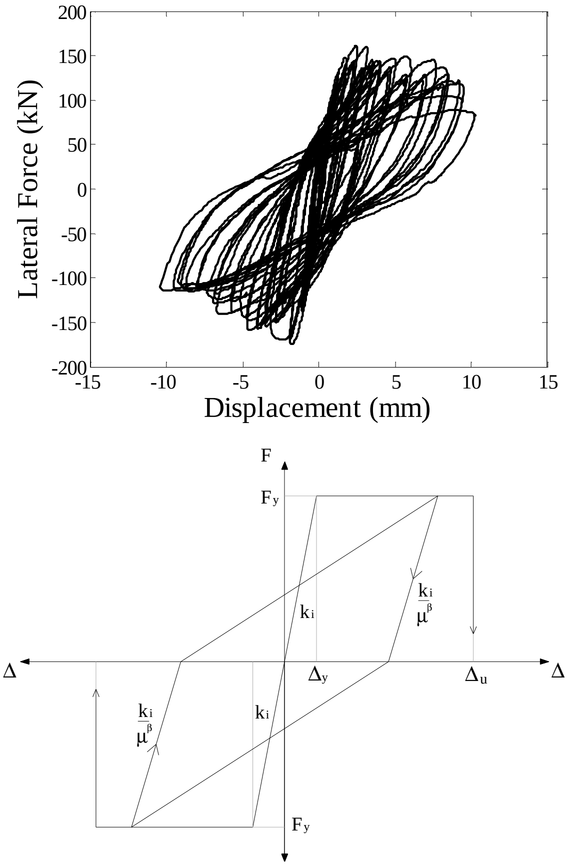


Figure 2. Lateral force-displacement response of case study masonry walls. From left to right: experimentally obtained response through quasi-static cyclic test on full scale wall (Javed 2008) and simplified used in the present study.

The yield displacement of nonlinear hinge is computed by dividing lateral strength over the shear stiffness of masonry element.

$$\Delta_{yh} = \frac{F_y}{K_S}; K_S = \frac{GA_S}{H_P} \qquad 15$$

where Δ_{yh} represents the yield displacement of shear hinge; F_y represents the lateral strength of masonry element; K_s represents the shear stiffness of masonry element; G represents shear modulus of masonry; A_s represents effective shear area, common as 80% of gross area (Magenes et al. 2000); H_P represents height of pier. The considered constitutive law has been investigated to give consistent result with that of static predictions for inelastic displacement demand on low-rise masonry structures (Ahmad et al. 2011d).

4.2.3 Acceleration time history used in the present study

The case study structural designed according to the considered characteristics are analyzed dynamically using NLTHA with ten natural accelerograms extracted from the PEER NGA data base for stiff soil condition with the mean spectrum compatible to EC8 Type I C-soil spectrum, see Figure 3 for the spectral shape of each time history. The accelerograms are previously selected and used also by other researchers for masonry structures (Menon and Magenes 2011), however in the present study all the accelerograms are anchored to a common PGA level thus resulting in different scaling factors than the previously used.

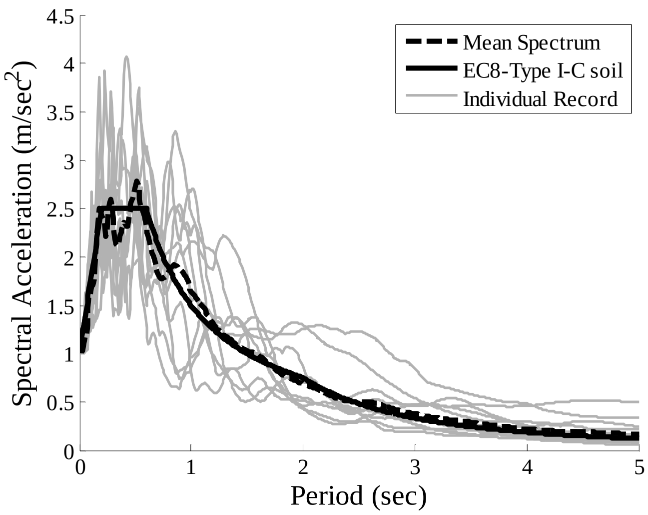


Figure 3. Mean spectrum of the selected accelerograms and comparison with the EC8 Type I-C soil spectrum

The structures are analyzed through incremental dynamic analysis (IDA) to different target PGA in order to deform the structures significantly beyond the yield capacity and which are used then to estimate the R factor for case study masonry structures. The computation of R factor herein is limited to the basic response modification factor due to ductility capacity and hysteretic response of masonry structures while the contribution from overstrength is not considered due to the reason mentioned earlier. The current study used the concept of Kappos to estimate the basic R factor through IDA and which is recently applied to other structures as well (Kappos et al. 2011, Ali et al. 2011, Zafar & Andrew 2011).

$$R = \frac{PGA_{ultimate}}{PGA_{yield}} \qquad 16$$

where $PGA_{ultimate}$ corresponds to the ground motions at the ultimate ductility capacity i.e. the PGA that trigger the collapse of the structure by exceeding the ultimate inter-storey drift capacity; PGA_{yield} corresponds to the ground motions that causes the structures to yield. The above concept is derived from the early proposal of Kappos (1991) and which is proposed and employed by other researchers as well (Elnashai & Broderick 1996, Mwafy & Elnashai 2002,).

The idea of IDA of structures is to develop seismic demand chart correlated with input excitations and which can be interpolated to identify the ground motions capable of exceeding yield and ultimate capacity limit states of the structures and which can in turn be used to estimate R factor using Eq. (16). All the accelerograms are anchored to different target PGA considering linear scaling, which are used to analyze the case study structural models through IDA. The linear scaling of accelerograms can reasonably provide estimate of seismic response parameters, however with relatively higher dispersion (Hancock et al. 2008), which is nevertheless conservative.

Figure 4 reports such an exemplificative charts for a case study masonry structure while charts for all other structures are provided in the Appendix A-1 & -2. The analysis shows significant effects of target ductility, wall density, floor area and energy dissipation on the R factor of masonry structures. It is worth to mention that except the ductility capacity all other parameters are not considered in the current building code of Pakistan which is generally found less than the code specified. The derived charts for R factor can be readily used given the wall density, floor area and target ductility for the assessment and preliminary design of masonry structure using hand calculations.

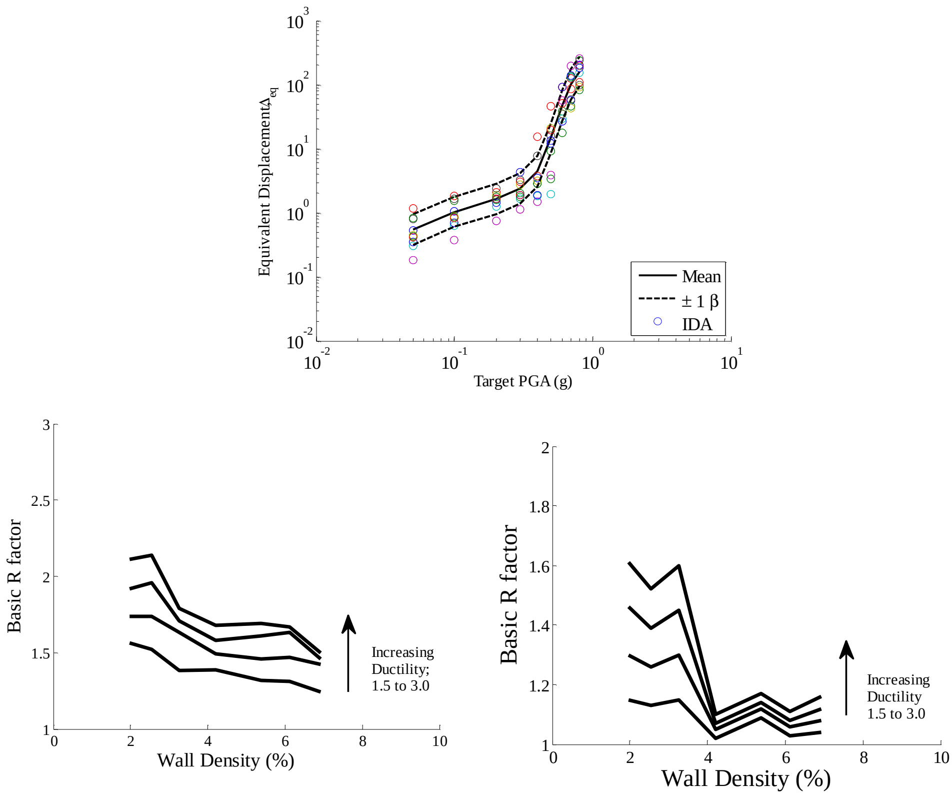


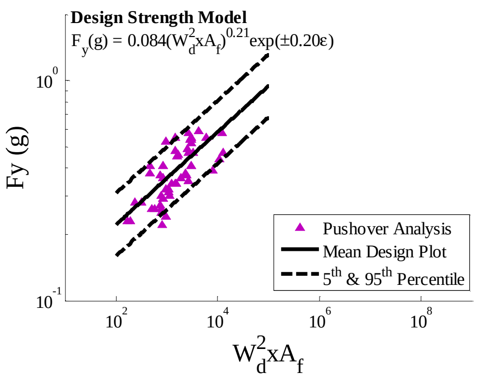
Figure 4. IDA analysis of a masonry structure for different target PGA and the development of chart to estimate R factor. From top to bottom and left to right: Displacement demand chart for case study structure with floor area of 133m^2 and wall density of 6.92% and the computation of R factor (16th percentile which has 84 percent chances of

being exceeded considering the record-to-record variability) for different target inter-storey ductility using the proposed hysteretic rule and the origin centered rule (with no hysteretic energy dissipation).

4.3 Simplified design charts for masonry structures

The present study also included a simplified hypothesis for the design and assessment of brick masonry structures in Pakistan using readily available design charts. For this purpose additional fifty random structural models are prepared considering the material and geometric properties as random variable which are analyzed through nonlinear static pushover analysis in order to quantify the lateral strength of masonry structure correlated with the structure geometric features for the considered region.

Figure 5 shows the strength of all case study structures and the analytical design strength model developed for structures. Similarly, the R factor is also correlated with the geometrical features in order to develop design model for R. The results for yield strength and R factor show a reasonably good correlation with wall density and floor area of the structures. The two plots thus provide an easy mean to design and/or assess brick masonry structures given the wall density and floor area of considered structure or select an appropriate wall density for a given seismic demand given the floor area (it will be known at the start of the design).



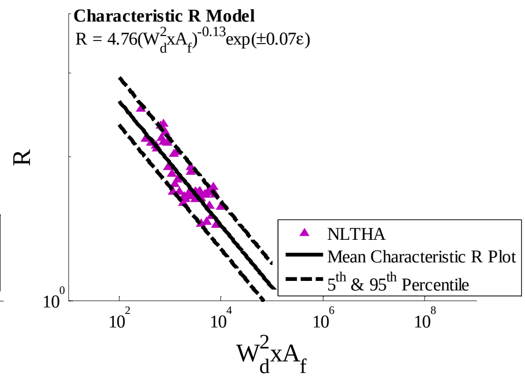


Figure 5 Simplified design charts for masonry structures. ***from left to right:***

lateral strength of masonry structures and R factor for target interstorey ductility of 3.

Similarly using the derived model for lateral strength of masonry structures and the R factor can be correlated using the Kappos model Eq. (16) to obtain the ground motions corresponding to the collapse of the system which can be used with a certain factor of safety to estimate the wall density of masonry structures given the floor area. The following strength model is derived considering the mean lateral strength model and the characteristic value of R factor.

$$PGA_{ult}(g) = 0.16\left(W_d^2 x A_f\right)^{0.08} \quad 17$$

Where PGA_{ult} represents the peak ground motions, in terms PGA(g) of code spectra, the structure can survive without total collapse; W_d represent the wall density of the structure at the weaker storey (ground floor in the present case due to soft-storey mechanism); A_f represents the total covered area in square meters. The above derived model is thus in turn used to develop a simplified chart for the design of masonry structures given the design ground motions PGA and floor area of the structure, see Figure 6, which can be used for the design and/or assessment of case study brick masonry structures in Pakistan.

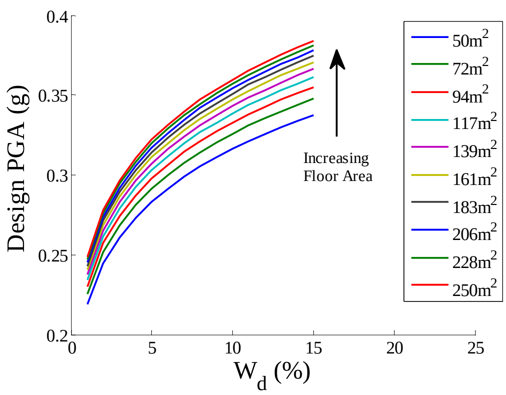


Figure 6 Simplified design charts for masonry structures for a specified ground motion given the structural floor area.

5. CONCLUSIONS AND RECOMMENDATIONS

The present paper presents the numerical investigation of low-rise (two storey) brick masonry structures in Pakistan through nonlinear static and dynamic time history analysis, in light of the regional material and geometric properties besides the prevailing mechanism, in order to develop tools and guidelines for the design and assessment of masonry structures through the use of readily available charts. The findings here in is applicable to low-rise unreinforced brick masonry structures in Pakistan that can ensure in-plane global seismic response by any means of achieving the minimum requirements discussed earlier and are governed by shear failure of masonry walls, most prevailing in the field. The present study can provide opportunity of learning and future research investigation for design code development for masonry structures. The following conclusion are drawn from the present study.

- The basic R factor specified by the building code of Pakistan for masonry structures is higher for the case study brick masonry structures and thus is severely unconservtaive for the considered structures.
- The building code of Pakistan in general ignores the effect of hysteretic energy dissipation in the specification of basic R factor which is only dependent on the ductility capacity, as given in the code. However, the present study shows that the hysteretic energy dissipation affects the R factor tremendously.
- The basic R factor is obtained using incremental dynamic analysis and the Kappos model which is reasonably very well correlated with the structural geometric features i.e. wall density and floor area, for the case study masonry structures.

- The following model is developed for R factor given the floor area A_f (m^2) and wall density W_d (%) of structures:

$$R = 4.76\left(W_d^2 x A_f\right)^{-0.13}$$

- The yield strength of case study masonry structures are reasonably well correlated with wall density and floor area. the following strength model is developed, considering the characteristic material properties, for the lateral strength evaluation of case study masonry structures.

$$F_y(g) = 0.084\left(W_d^2 x A_f\right)^{0.21}$$

- Nonlinear static pushover analysis is performed on randomly generated structures, designed according to the regional geometric and material properties, in order to develop model for the minimum ground motions capable of causing the collapse of case study masonry structure. The following model is developed herein.

$$PGA_{ult}(g) = 0.16\left(W_d^2 x A_f\right)^{0.08}$$

- Simplified design charts are provided to estimate the required wall density for a given seismic zones in Pakistan, given the floor area of structure. The derived charts show that the case study structures can survive ground motions well above 0.21g and up to 0.36g given that the minimum requirements for floor area and wall density are achieved.

The authors acknowledge the accuracy of the currents findings which are made in light of the experimental investigation of masonry material and full scale masonry walls and numerical investigation of prototype of structural models and thus recommend their onward use in the field for assessment and/or design of case study brick masonry structures. Nevertheless, additional experimental and numerical investigations can further validate the findings provided herein and in turn then can increase the confidence in future applications.

ACKNOWLEDGEMENT

The first author sincerely thank Prof. Guido Magenes and Prof. Tim Sullivan of the University of Pavia and Dr. Ihsan Bal (Structural Engineer at Fyfe Europe) for their early generous discussion which was very helpful in the nonlinear static and dynamic seismic analysis of case study masonry structures. Dr. Qaisar Ali of the Earthquake Engineering Center (EEC), Peshawar is highly thanked for hosting the first author and for his kind hospitality during his one week visit to UET Peshawar.

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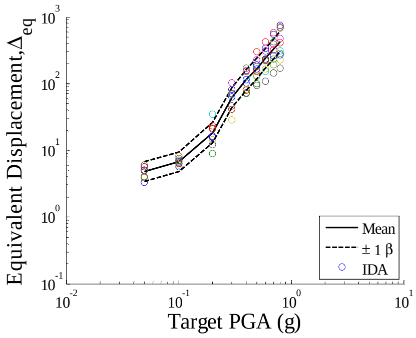
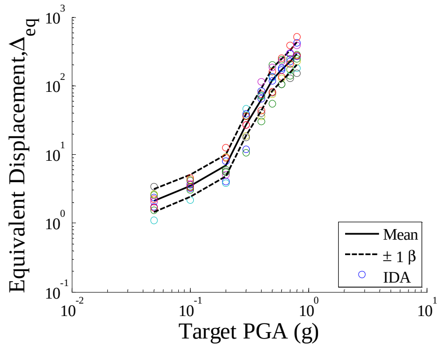
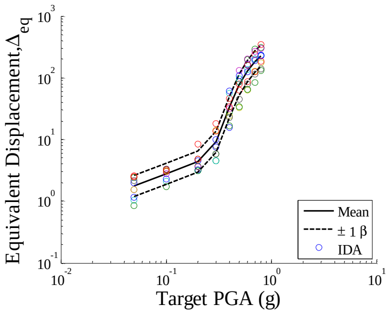
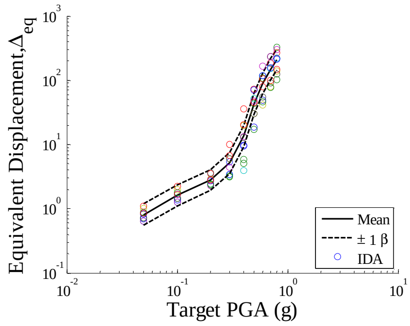
APPENDIX A

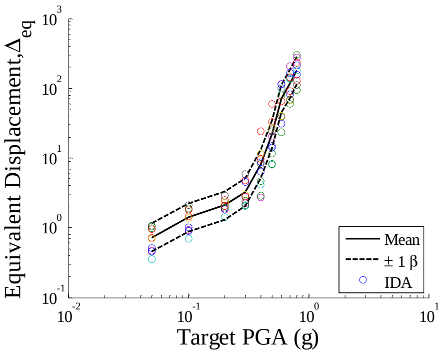
Derivation of displacement demand chart through incremental dynamic analysis for the computation of basic R factor

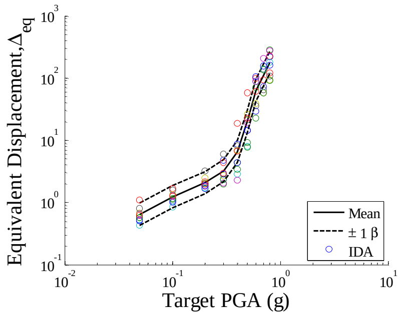












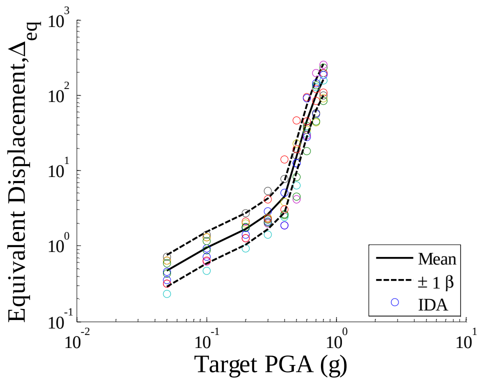
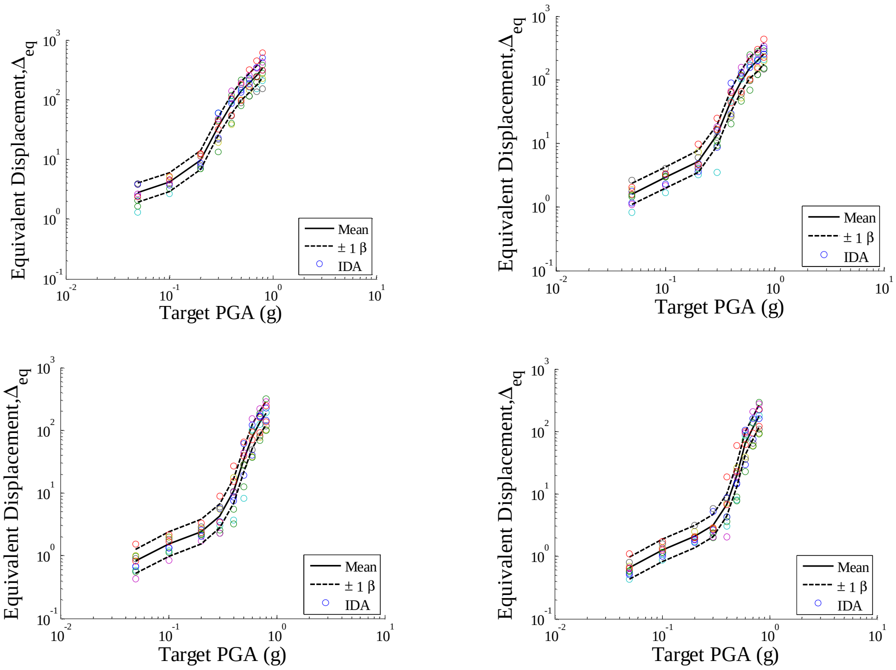
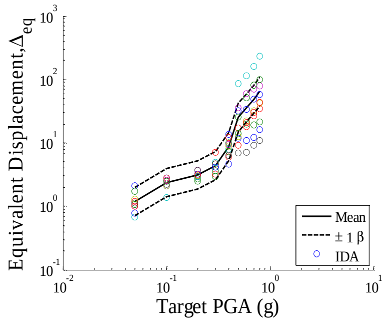


Figure A. 1 Displacement demand chart for case study structures with A_f = 69 m^2. ***From left to right and top to bottom:*** **Wd = 1.98 %, Wd = 2.55 %, Wd = 3.27 %, Wd = 4.20 %, Wd = 5.39 %, Wd = 6.11 %, Wd = 6.92 %**





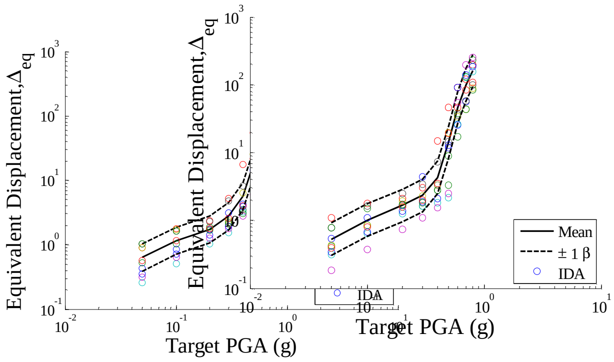
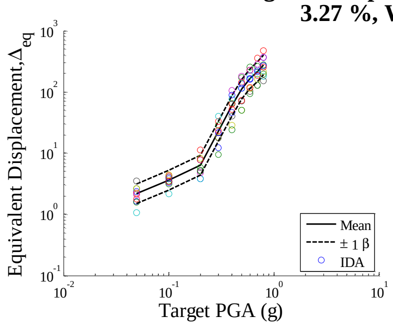
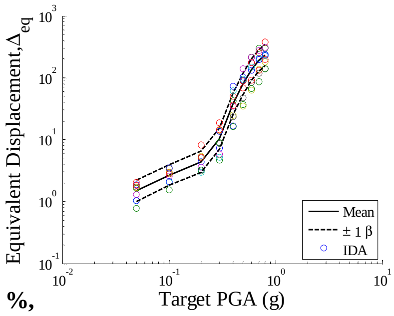


Figure A. 2 Displacement demand chart for case study structures with Af = 86 m2. From left to right and top to bottom: Wd = 1.98 %, Wd = 2.55 %, Wd = 3.27 %, Wd =





4.20 %, Wd = 5.39 %, Wd = 6.11 %, Wd = 6.92 %

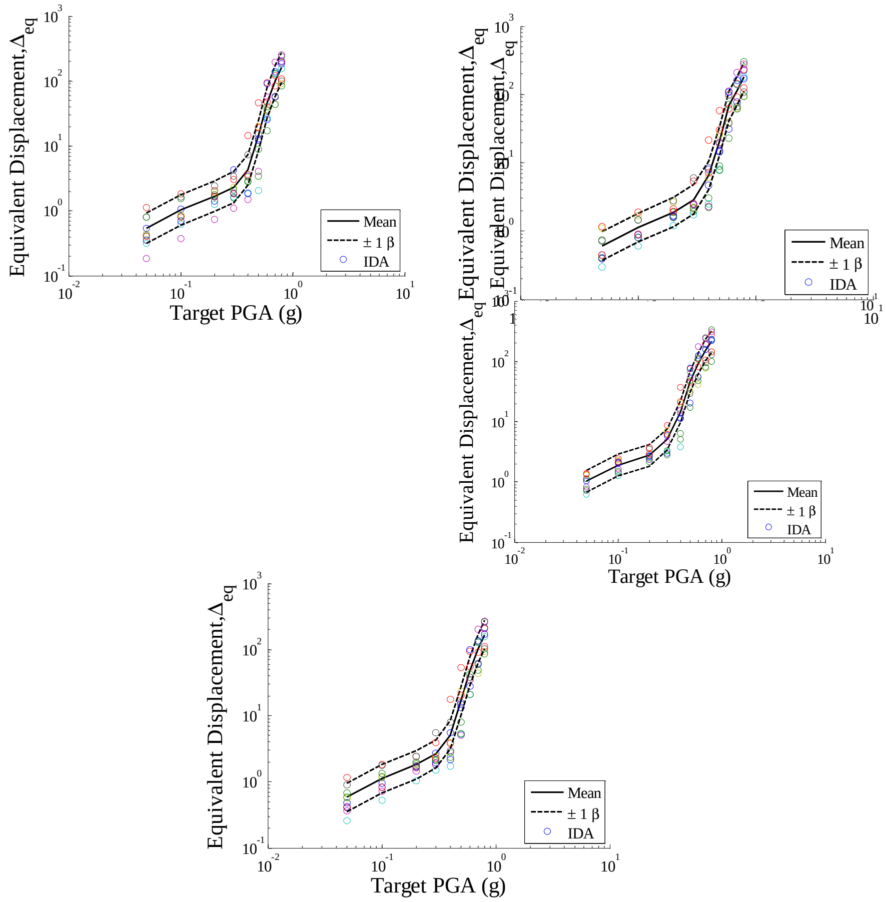
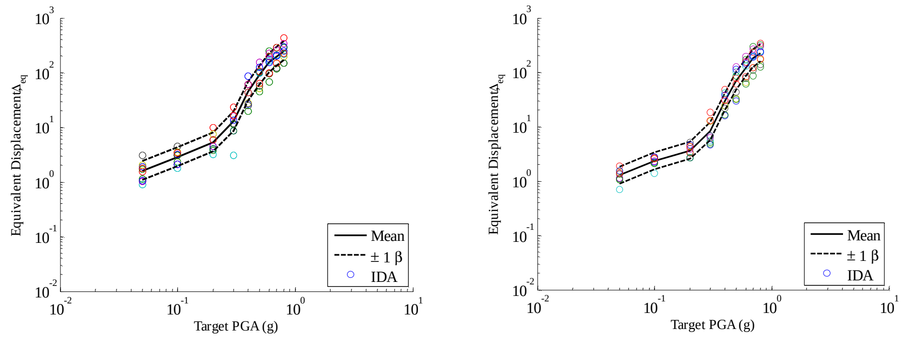


Figure A. 3 Displacement demand chart for case study structures with Af = 107 m2. From left to right and top to bottom: Wd = 1.98 %, Wd = 2.55 %, Wd = 3.27 %, Wd = 4.20 %, Wd = 5.39 %, Wd = 6.11 %, Wd = 6.92 %



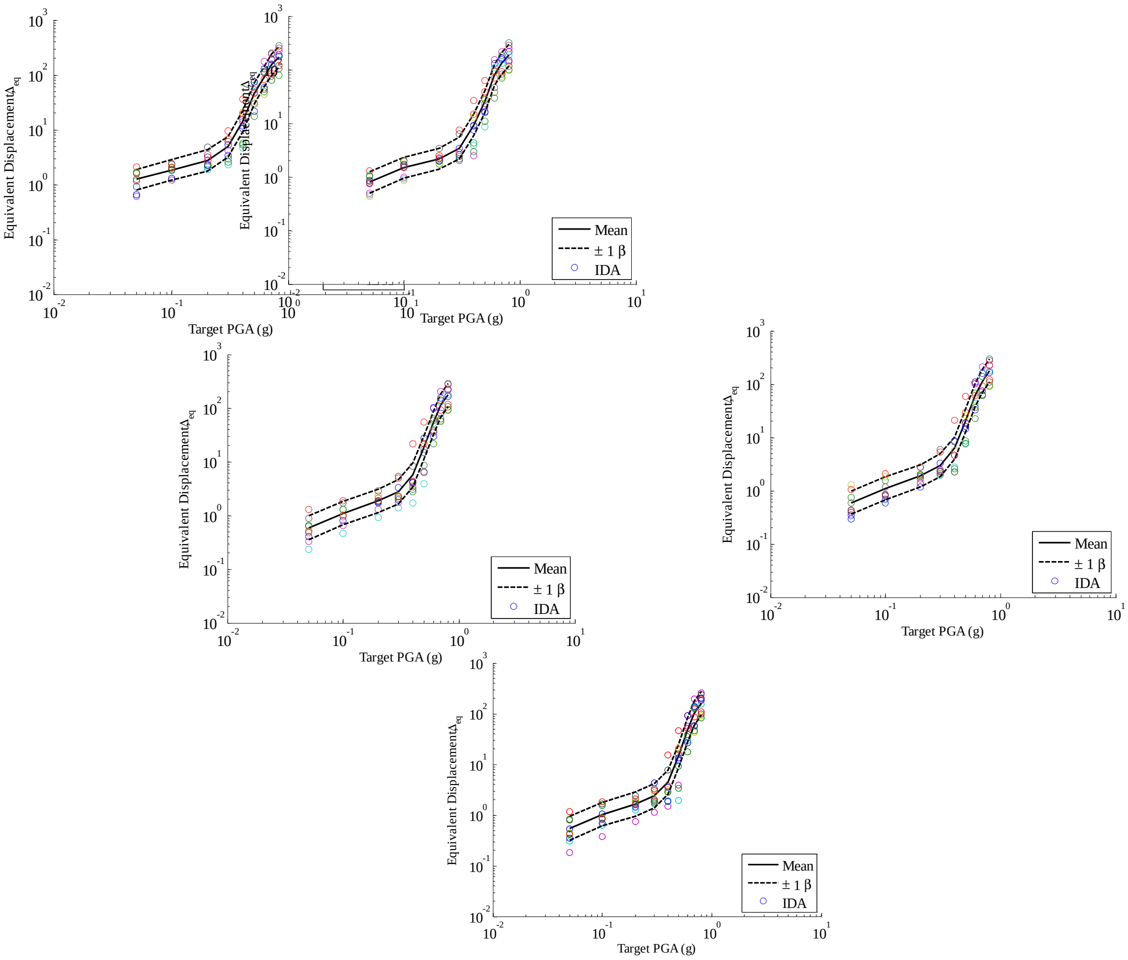
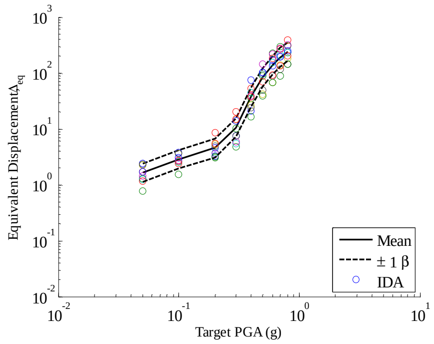
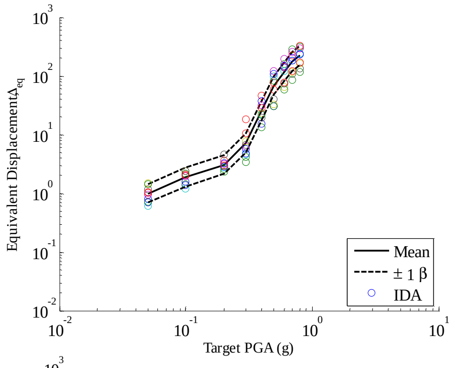
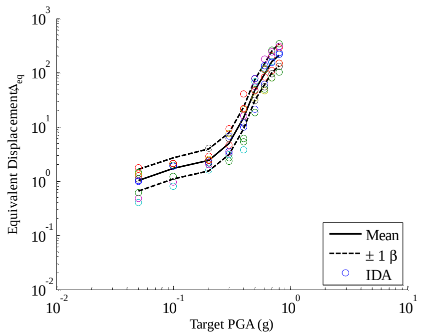
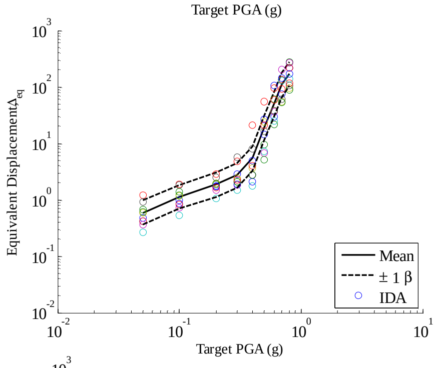


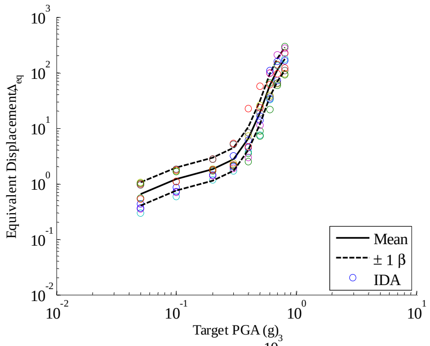
Figure A. 4 Displacement demand chart for case study structures with Af = 133 m2. From left to right and top to bottom: Wd = 1.98 %, Wd = 2.55 %, Wd = 3.27 %, Wd = 4.20 %, Wd = 5.39 %, Wd = 6.11 %, Wd = 6.92 %

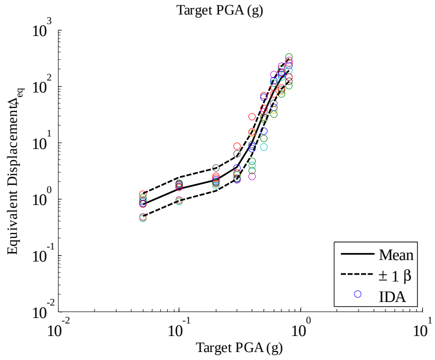

Equivalent DisplacementΔ_{eq}
Target PGA (g)
Mean
± 1 β
IDA


Equivalent DisplacementΔ_{eq}
Target PGA (g)
Mean
± 1 β
IDA


Equivalent DisplacementΔ_{eq}
Target PGA (g)
Mean
± 1 β
IDA


Equivalent DisplacementΔ_{eq}
Target PGA (g)
Mean
± 1 β
IDA


Equivalent DisplacementΔ_{eq}
Target PGA (g)
Mean
± 1 β
IDA


Equivalent DisplacementΔ_{eq}
Target PGA (g)
Mean
± 1 β
IDA

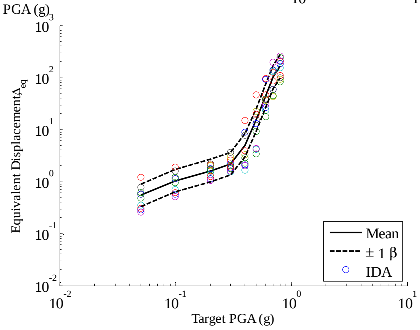
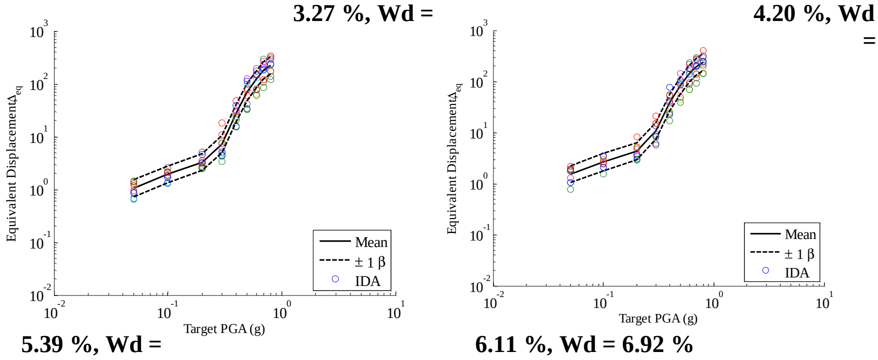

Equivalent DisplacementΔ_{eq}
Target PGA (g)
Mean
± 1 β
IDA

Figure A. 5 Displacement demand chart for case study structures with Af = 166 m2. From left to right and top to bottom: Wd = 1.98 %, Wd = 2.55 %, Wd = 3.27 %, Wd = 4.20 %, Wd =



5.39 %, Wd = 6.11 %, Wd = 6.92 %

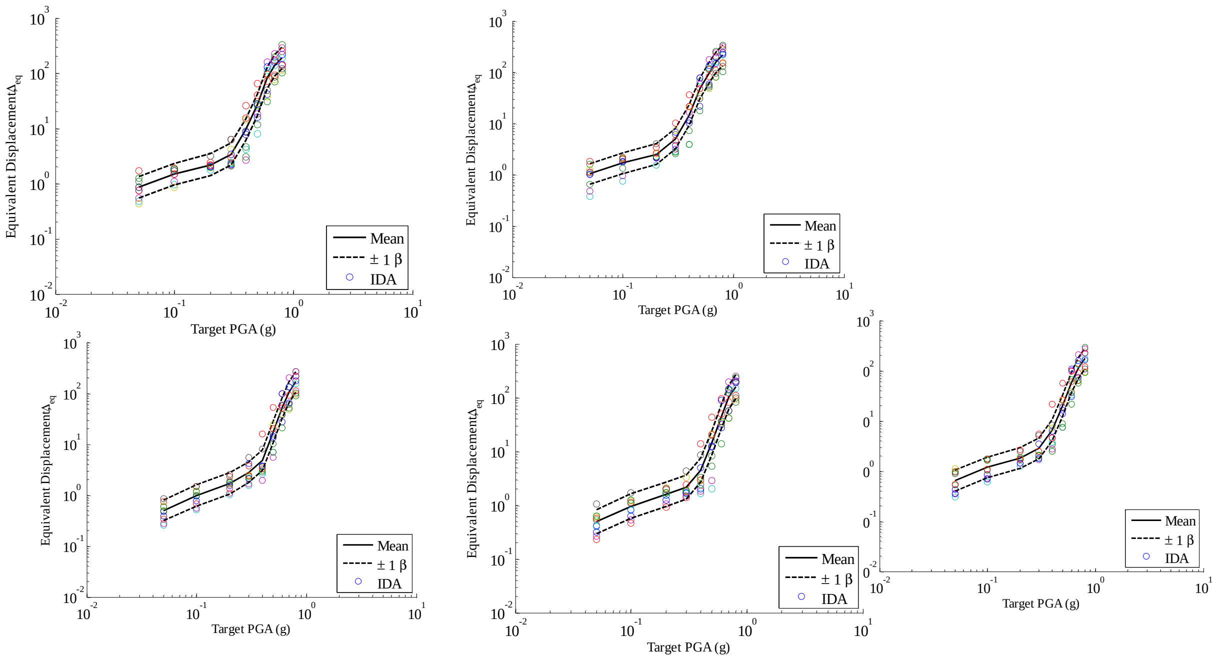


Figure A. 6 Displacement demand chart for case study structures with Af = 185 m2. From left to right and top to bottom: Wd = 1.98 %, Wd = 2.55 %, Wd = 3.27 %, Wd = 4.20 %, Wd = 5.39 %, Wd = 6.11 %, Wd = 6.92 %

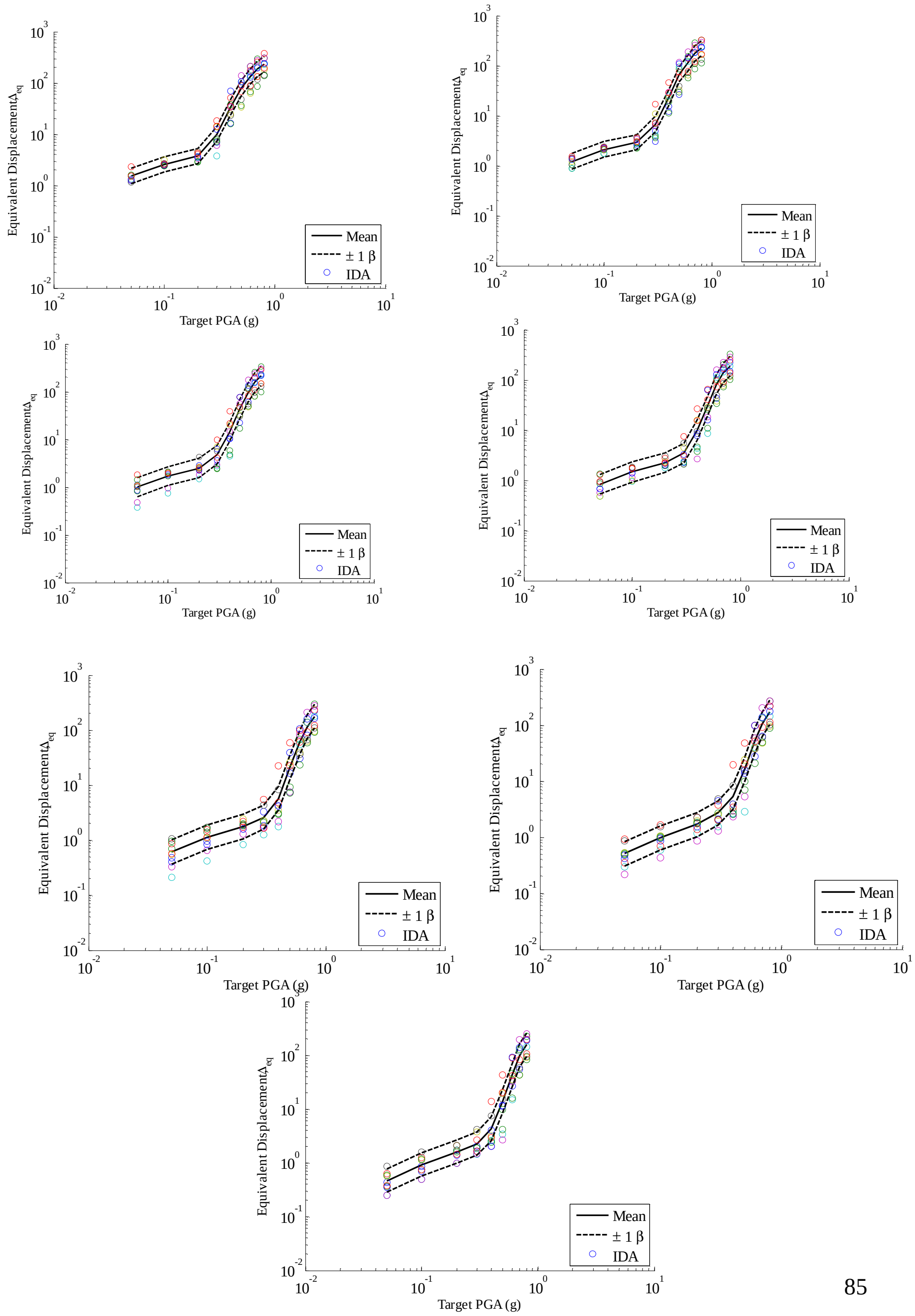
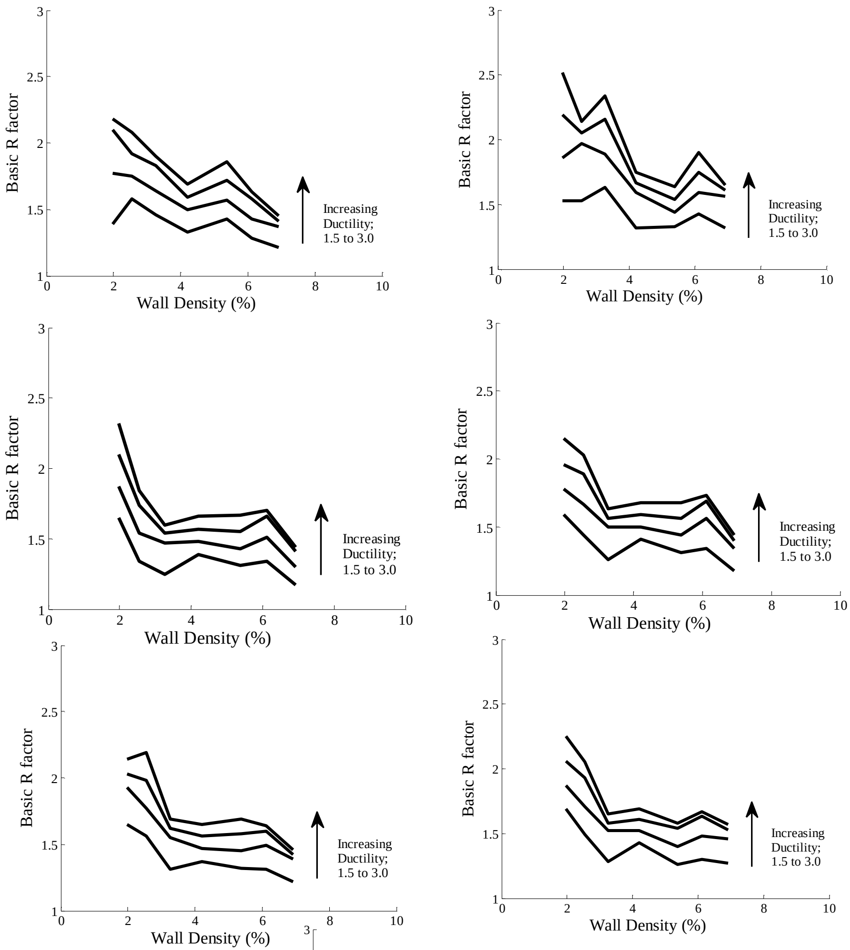


Figure A. 7 Displacement demand chart for case study structures with Af = 207 m2. From left to right and top to bottom: Wd = 1.98 %, Wd = 2.55 %, Wd = 3.27 %, Wd = 4.20 %, Wd = 5.39 %, Wd = 6.11 %, Wd = 6.92 %

Appendix A-2

Computation of Basic R factor for different target ductility



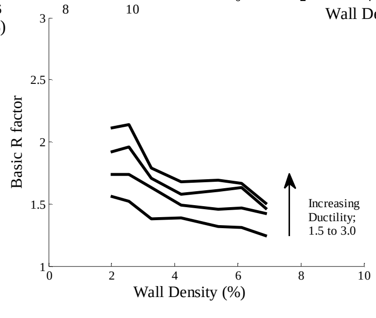


Figure A. 8 Basic response modification factor of masonry structure for target ductility. From left to right and top to bottom: Af = 69 m2, Af = 86 m2, Af = 107 m2, Af = 133 m2, Af = 166 m2, Af = 185