Confined Masonry for Seismically Resilient Low-Cost Housing in India:
A Design and Analysis Method

by

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B.S. Civil and Environmental Engineering
University of Illinois at Urbana-Champaign, 2012

Submitted to the Department of Civil and Environmental Engineering
in Partial Fulfillment of the Requirements for the Degree of

Master of Science in Civil and Environmental Engineering
at the
Massachusetts Institute of Technology
June 2015

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Abstract

Confined masonry (CM) is a viable housing typology that is resilient and economical for developing countries in seismic regions. Given its suitability for low-tech environments, multiple authors have published instructions on CM construction that do not require engineering knowledge. As a result these guidelines impose constrictive design requirements. Analysis methods exist for calculating the stress demand on shear walls of a CM building under earthquake loads which may be applied to any design, but they require technical expertise to perform. A procedure for designing confined masonry buildings is presented that employs a combination of seismic analysis techniques to take into account torsional effects and allow for complex designs while requiring low computational effort. Parametric studies are performed on this procedure which show reliable, conservative structural design outputs.

Confined masonry is a structural wall system, therefore its seismic resilience depends on the wall shear strength, which is related to the compressive strength of the masonry. In India bricks used to build homes are often of poor compressive strength, even lower than the minimum allowed by the Indian masonry code, which is lower than that prescribed in other international standards. Experimentation was conducted on the strength of masonry in Gujarat, India to investigate the effect of varying mortar qualities when low strength bricks are used. With average brick strengths below 2.5 MPa a mud mortar with no cement and a 1:8 cement:sand ratio mortar resulted in approximately 41% and 21% higher prism strengths, respectively, than a 1:6 cement:sand ratio mortar. This shows that a mortar with less cement would save cost and result in a more resilient structure when building with bricks of this strength. Observations and hypotheses are presented for this behavior, but larger scale testing is recommended to better understand this outcome and inform better building practices that can save lives and money.

Thesis Supervisor: John Ochsendorf

Title: Professor of Architecture and Civil and Environmental Engineering

Acknowledgements

This project would not have been possible without the generous financial and administrative support of the Sir Darabji Tata Trust and the Tata Center for Technology and Design at MIT and its members.

I could not have done this research without the help and support of my advisor and friend Professor John Ochsendorf. His guidance and unwavering confidence in me were encouraging in times of uncertainty and they provided solid ground on which I could always stand. His influence means great deal to me and I will forever be grateful to him.

Throughout my research Dr. Svetlana Brzev, faculty member at the British Columbia Institute of Technology, has provided invaluable guidance. From problem identification onward she has provided support in the most selfless way I have ever seen. None of this research would be possible without her and I am forever thankful for her contributions.

Vivek Rawal of People in Centre and all of the firm's members were valuable partners in this research. They provided enthusiastic support for my work, especially in the design and execution of my experiments. On my trips to India they were a familiar group that made me feel welcome in an unfamiliar country. These projects would not have been completed without them and their help and friendship is greatly appreciated.

P.B. Prajapati of the Government Polytechnic for Girls in Ahmedabad was extremely generous to allow the researchers to use his department's facilities for the experimental work on masonry prisms. Mr. Prajapati provided assistance during experimentation and welcoming and enthusiastic hospitality. The researchers are grateful to Mr. Prajapati and the school.

Vineet Kothari and Shija Sam, students at the LD College of Engineering, helped with prism testing and tested a number of prisms after I had left the country. Their assistance and hard work is greatly appreciated.

I would like to thank the KBM Engineering Research Laboratory for their excellent work in testing materials for this research. Their diligence and competence were beyond what I could have hoped for, and I am thankful to them.

Dr. Thomas Poinot provided excellent guidance as my post-doc supervisor in the Tata Center. He provided practical advice that was always relevant, useful, and reasonable amidst expectations that sometimes felt unreasonable. He was also a good friend and I thank him.

I would like to thank Mike Laracy. As my officemate at MIT he kept life from getting too stressful, as my teammate he was a trustworthy and fun comrade, as my roommate he put up with my annoying lifestyle, and as my friend he continues to provide good memories that will last forever. I would also like to wish him and his new fiancé Maggie a long and happy marriage.

Lastly, I would like to thank my family. The love and support of my mom, dad, brother, and sister continue to positively influence my life in ways that cannot be measured, and in return I give them my love and endless gratitude.

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1 Introduction

1.1 Context and Problem Statement

Much of India is prone to substantial earthquakes (Fig. 1.1) and housing for families in the economically weaker sector (EWS) and lower income group (LIG) is highly vulnerable to such events. This fact was exposed in the 2001 Bhuj earthquake that killed over 13,800 people and destroyed or damaged more than one million structures (Fig. 1.2), with building damage as the primary cause of human casualties (Jain et al. 2002; Saito et al. 2004; Murty et al. 2005). In poorer sectors the materials and workmanship for housing are often of low quality, and builders are unfamiliar with modern structural techniques (Murty et al. 2006). There is also a lack of motivation to invest in safe homes because many Indians have not been exposed to a devastating earthquake and their financial priorities lie in daily life functions (Arya et al. 2005). To make things worse, the Ministry of Housing and Urban Poverty Alleviation estimates the rural and urban housing shortages in India at 40 and 19 million homes, respectively, and these numbers continue to grow due to the increasing population and rapid urbanization (Kurup 2014; Jain 2015). There is a substantial need for housing solutions that cater to the needs and resources of the lower economic classes. The current research focuses on this issue within the context of India, however it applies to all earthquake prone developing countries.

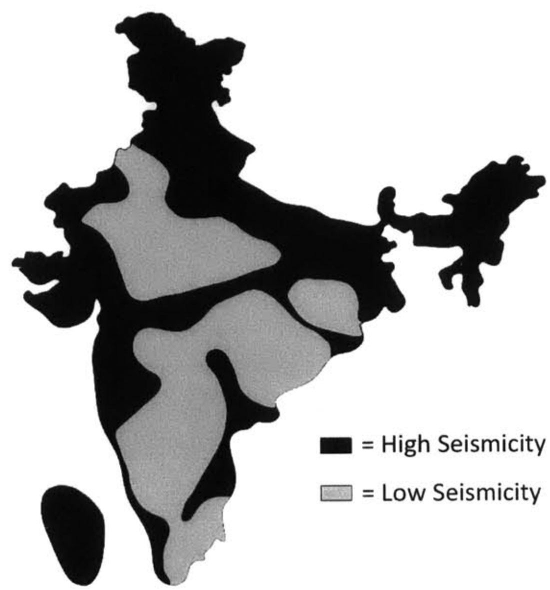
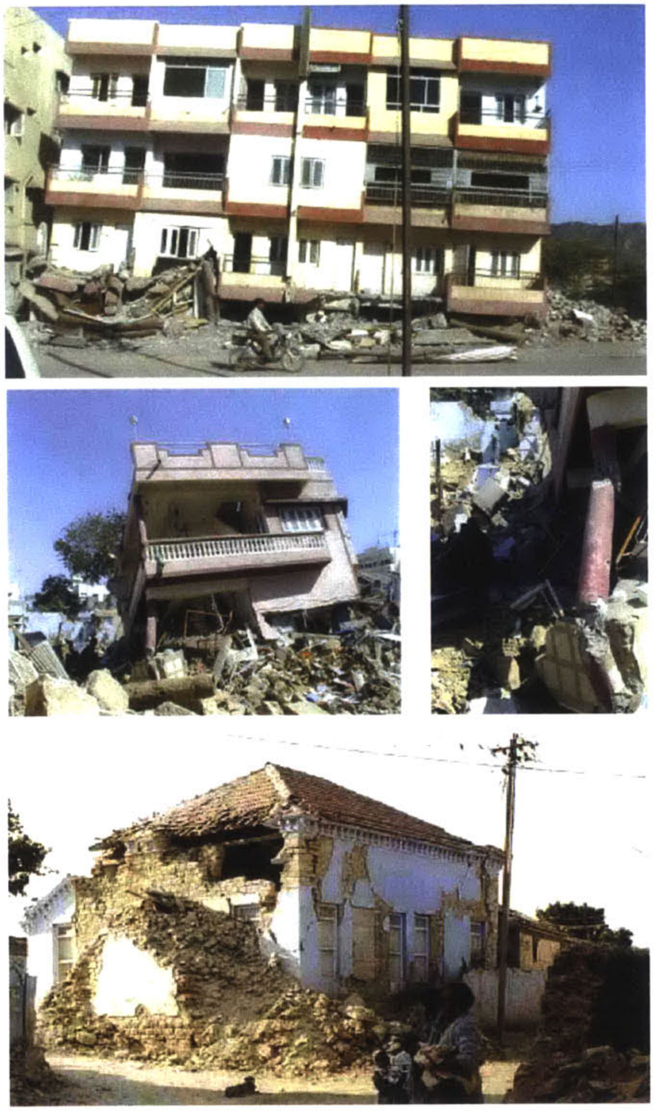


Fig. 1.1. Map of India designating zones of high seismicity (Zones III-IV) and low seismicity (Zones I-II), derived from IS 1893.



(a)

(b)

(c)

Fig. 1.2. Various common failures observed after the 2001 Bhuj earthquake (M7.7), soft story failures were extremely prevalent (a), columns often failed due to the short column effect (b) and inadequate rebar detailing, and masonry wall failures were common as well (c) (Madabhushi et al. 2005).

Approximately 45% of houses in India are made of unreinforced burnt clay brick masonry, and reinforced concrete frames with masonry infill have been very common in urban India for the last 35 years and more recently it has spread to rural areas (Jaiswal et al. 2002; Murty et al. 2005; Murty et al. 2006; Iyer et al. 2012). As stated earlier, these houses are not often built safely: in the 2001 earthquake over 230,000 masonry houses and several hundred RC frame buildings collapsed (Jain et al. 2002). Given the vulnerability and prevalent use of these materials, it is apparent that a better way of building with bricks and concrete must be found.

1.2 Introduction to Confined Masonry

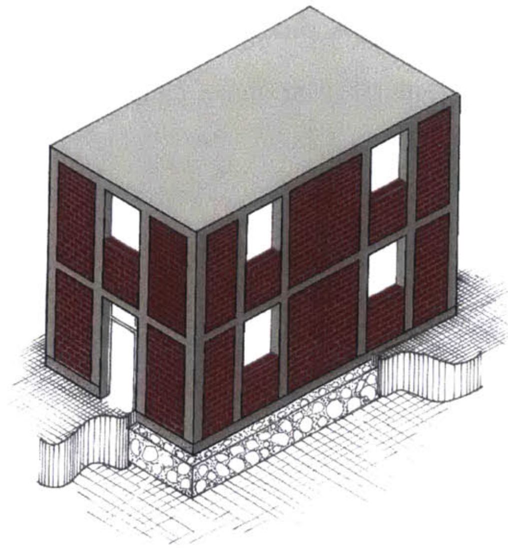


Fig. 1.3. Sketch of a two-story confined masonry building from Brzev (2007).

Confined masonry is a proposed solution to the need for seismically resilient housing in India. CM is a structural wall system comprised of load bearing masonry walls with surrounding reinforced cement concrete (RCC) elements, see Fig. 1.3. CM is attractive for its desirability, efficient use of materials, and satisfactory seismic performance. It is used in many countries, however there are challenges to its widespread use in India and other developing countries: there is no confined masonry code in India, the construction of CM is less mechanized than RCC frame, and although proper CM has performed sufficiently in earthquakes, premature failures have been reported as well due to insufficient design and construction (Basu et al. 2014, Jain 2015). These challenges will be elaborated on later. One way proponents of CM address concerns with under-designing and construction deficiencies is by publishing guidelines to help ensure its proper use in seismic zones.

1.3 CM Construction and Analysis Guidelines

Construction guidelines for confined masonry are powerful tools for architects and builders without an engineering background to build with proper construction detailing. Multiple guidelines exist for different countries, many of which are derived from Blondet et al. (2005), originally made for Peru. For engineers with technical expertise, multiple methods for analysis and design of CM structures also exist which will be discussed, such as those from Meli et al. (2011) and by the Earthquake Engineering Research Institute (EERI). Many building codes also have requirements pertaining to confined masonry including the Eurocode 6 (EN 1996), the Mexican building code (NTC-M 2004), Chilean (2003) and Peruvian (2006) codes (Meli et al. 2011).

Existing guidelines are limited in that they either do not perform any seismic analysis and heavily constrict the design, or they require technical expertise to perform. Herein lies an opportunity for an architectural guideline for confined masonry which employs seismic analyses while still being accessible to architects and builders without an engineering background. A design procedure is presented for confined masonry homes that is intended to empower architects with fair architectural freedom while guiding them to a structurally resilient solution. Two existing methods for the design and analysis of CM are described in detail and are combined with a few simplifying adjustments to create the current method.

1.4 Material Properties and Experimental Research

Experimental results are also presented which investigate the effect of varying mortar qualities on masonry compressive strength when low strength bricks are used. Material properties are a crucial consideration in any structural design, especially with in low-tech environments where the quality of the materials is highly variable. In rural areas of India and other developing countries it is common to use bricks with strength as low as 2 MPa (Sarangapani et al. 2002; Sarangapani et al. 2005; GSDMA 2005; Gumaste et al. 2007; San Bartolomé and Quiun 2008), and compressive strength of masonry is one of the main parameters influencing the vertical and lateral load capacity of a confined masonry wall. Despite the widespread use of low quality bricks in these areas, limited research exists that investigate their effect on masonry strength. This presents an enormous research opportunity that can lead to better informed construction in low-tech seismically active countries.

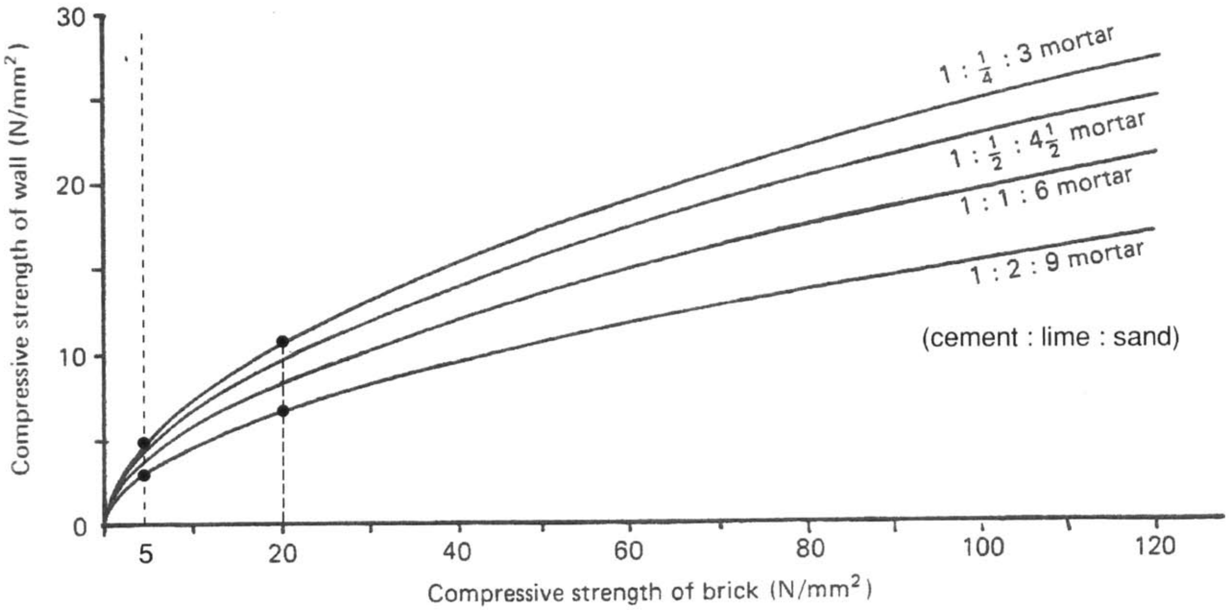


Fig. 1.4. Plot of masonry versus brick compressive strength for four mortar mixes (derived from Hendry 1990).

Masonry and mortar strengths are assumed to have a positive relationship in building codes (IS 1905, EN 1996, IBC 2012), but studies have found that this relationship is less pronounced with lower strength bricks. In Fig. 1.4 one may see that as brick compressive strength decreases, the difference in masonry strength observed between mortar mixes (represented by the vertical distance between the plot lines) becomes less pronounced, and for bricks weaker than 5 MPa the difference shrinks to zero (Hendry 1990; Drysdale et al. 1994). For example, with 20 MPa strength bricks, the difference in masonry strength achieved with the strongest ($1{:}\frac{1}{4}{:}\,3$) and weakest ($1{:}\,2{:}\,9$) mortars is approximately 4 MPa, whereas with 5 MPa bricks the difference drops to approximately 2 MPa. Some researchers have concluded that with low quality bricks in certain circumstances the opposite trend is true, that is, stronger mortar leads to weaker masonry (Samarasinghe and Lawrence 1992; Sarangapani et al. 2005; Gumaste et al. 2007). Although some studies exist that address these issues, the research is very limited. The current experiments investigate whether material cost can be saved, and masonry strength gained, by using a mortar mix with a small cement proportion when low strength bricks are used.

1.5 Thesis Outline

In Chapter 2 of this thesis confined masonry as a structural system is introduced and it is assessed whether CM is suitable for seismically active, low-tech construction environments such as India. In Chapter 3 existing approaches to the seismic design and analysis of masonry and confined masonry are discussed and research gaps are identified. Chapter 4 presents an innovative method for the design and analysis of confined masonry structures that combines and simplifies two unrelated CM procedures. The author seeks to create a method that is usable by architects and builders without engineering expertise that still allows fair architectural freedom. Chapter 5 contains a brief discussion on the material properties of masonry as they apply to the proposed design method and on masonry materials in India. This chapter also summarizes experimentation that was conducted in India on local masonry properties and discusses the effect of cement quantity of mortars when low-strength bricks are used. Finally, Chapter 6 summarizes the main conclusions of this thesis and presents opportunities for future research.

Appendices A – F are also included which contain references, supplementary materials for the proposed design method, detailed experimentation reports, and a full list of notations used in this thesis.

2 Confined Masonry

2.1 Performance in Past Earthquakes

Confined masonry is a proposed solution for India based on its satisfactory earthquake performance in Latin America. Peru, for example, is a country with economic standing comparable to that of India where confined masonry has contributed significantly to mitigation of earthquake losses, as was confirmed by the 2007 Pisco earthquake. Confined masonry has been the most common housing construction practice in Peru for over 40 years (Loaiza and Blondet 2002). Prior to that, adobe construction was predominant for single family houses, and these were the most damaged buildings in the 1970 Chimbote and 2007 Pisco, Peru earthquakes (Cluff 1971; San Bartolomé and Quiun 2008). According to Cluff (1971) and Blondet (2007), most adobe buildings in affected cities for both earthquakes were destroyed or seriously damaged. These events present a revealing case of two earthquakes with similar magnitudes (7.9 and 8.0, respectively) in the same country, separated by 37 years and very contrasting outcomes. The death toll of the 1970 earthquake was approximately 70,000; for the 2007 Pisco earthquake it was 519 (Cluff 1971; San Bartolomé and Quiun 2008; Romero 2010). The difference between the two events is due to the shift in construction practice from adobe towards confined masonry after the 1970 earthquake and its satisfactory performance in the 2007 earthquake, see Fig. 2.1. Some CM structures did suffer severe damage or collapse in the 2007 earthquake, but these were due to construction and design deficiencies attributed mainly to informal construction, an issue that is also very common in India. Otherwise proper CM construction suffered little to no damage in 2007 (Blondet 2007; Brzev 2007; San Bartolomé and Quiun 2008; Meli et al. 2011). It should be noted that a large portion of deaths in the 1970 earthquake were due to landslides that destroyed the villages of Yungay and Ranrahirca, the likes of which did not occur in the more recent event, however the effects discussed here pertain to those cities not affected by the landslides (Cluff 1971).

Fig. 2.1. Photograph taken in the aftermath of the 2007 Pisco, Peru earthquake (M8.0) by Blondet (2007). A six story confined masonry structure has survived with no damage whereas the building next to it completely collapsed.

CM has shown good performance both in laboratory testing and in multiple other earthquakes as reported by Moroni et al. (2002), Moroni et al. (2004), Tena-Colunga et al. (2009), Brzev et al. (2010), Meli et al. (2011), and others (http://www.confinedmasonry.org/category/around-the-world/). Many open source publications on the use and research of confined masonry across the globe and its earthquake performance may be found at confinedmasonry.org.

2.2 Seismic Behavior of CM Walls

So what gives CM its seismic resiliency? CM is a composite wall system, that is, a combination of masonry and surrounding reinforced cement concrete elements, called tie-columns and tie-beams. The masonry is the main lateral and vertical load bearing component, see Fig. 2.2. The RCC elements exist solely to grip the masonry to engage it under lateral loading, provide extra ductility, and help prevent out-of-plane failure. The concrete carries a limited vertical

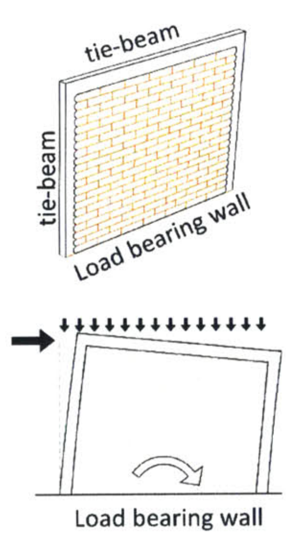
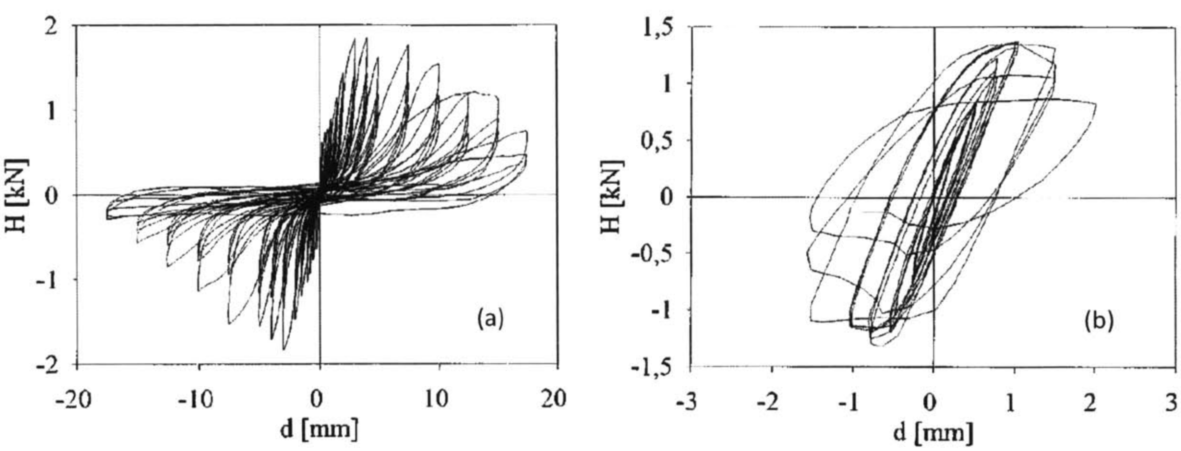


Fig. 2.2. Sketch of the lateral load transferring behavior of confined masonry (Meli et al. 2011).

load, hence the RCC tie elements are smaller in cross section than a load bearing concrete frame (Brzev 2007; Meli et al. 2011). Because steel and cement are the most expensive building materials in India, this factor makes CM less expensive than RCC frame construction. Each component of the walls plays an important structural role. The masonry panel develops a diagonal compression strut when resisting lateral loads, initially not relying at all on the RCC columns (Brzev 2007). Once the masonry has cracked, tie-columns play an important role in providing ductility prior to collapse. Experimental research by Tomazevic and Klemenc (1997) shows the large increase in ductility of confined versus unconfined masonry, see Fig. 2.3. Added ductility prior to failure saves lives during an earthquake by giving building occupants extra time to evacuate.



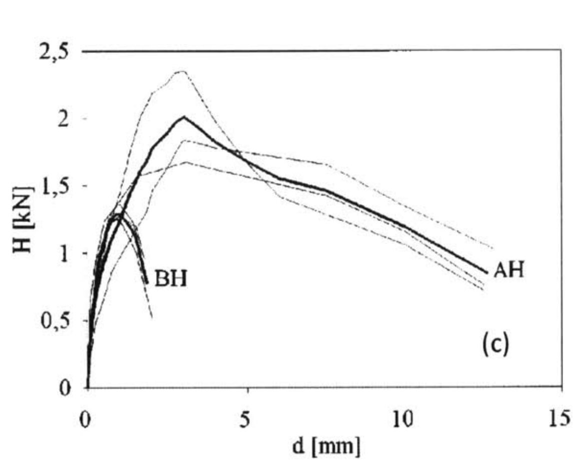


Fig. 2.3. Lateral load, H, versus deflection, d, hysteresis loops for a confined masonry wall (a) and a plain (unreinforced) masonry wall (b), note the deflection of the confined wall reaches nearly ten times that of the plain wall. Load deflection curve for a plain masonry wall, BH, and a confined masonry wall, AH (c). (Tomazevic and Klemenc 1997).

Wall density, d, or the ratio in plan of the structural wall area in the direction of applied seismic force to the total plan (footprint) area of the floor, as illustrated in Fig. 2.4, is a key factor affecting the overall seismic performance of CM. Reports of building performance after the 2007 Pisco, Peru and 1985 Llolleo, Chile earthquakes showed that many of the collapsed and severely damaged buildings had inadequate wall densities (San Bartolomé and Quiun 2008). A relationship between the level of damage in a building and the wall density per unit floor based on a survey of buildings following the 1985 Llolleo earthquake is shown in Table 1, adapted from Moroni et al. (2000). Wall density directly relates to the shear force capacity of a floor, and the Simplified Method for Seismic Analysis (SMSA), described later, uses this as the main design and analysis criteria for a CM building. Furthermore, confined masonry with adequate wall density is forgiving of minor construction defects (Brzev 2007).

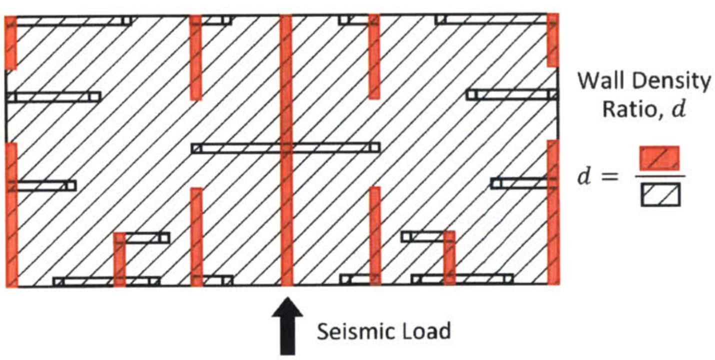


Fig. 2.4. Wall density illustrated in the floor plan view above (derived from Meli et al. 2011).

Level of Damage	Wall Density d/N (%)
light	1.15
moderate	0.85-1.15
severe	0.5-0.85
heavy	< 0.5

Table 1. Relationship between wall density and damage sustained in CM buildings after 1985 Llolleo, Chile earthquake (Moroni et al. 2000).

2.3 Suitability for India: Advantages and Drawbacks

CM is attractive because it is less expensive than an RCC frame but holds the same aesthetic, which has strong aspirational qualities (Murty et al. 2005; Murty et al. 2006; Iyer et al. 2012). Furthermore, as discussed, its seismic performance has been proven in past earthquakes and verified by laboratory experiments. However there are challenges to its use in India and other developing nations. Without confined masonry codes engineers in India are reluctant to approve of such small concrete elements, and there is no code approved system for designing it (Basu et al. 2014). Secondly, the construction of CM is less mechanized than RCC frame, therefore the labor cost can be greater and it is more prone to construction flaws, especially initially when the process is unfamiliar (Jain 2015). The materials cost savings are expected to far outweigh the increase in labor cost in developing countries such as India where labor is inexpensive (Meli et al. 2011). Although proper confined masonry has performed well in earthquakes, premature failures and collapses have been reported as well, which occurred when the design or construction was inadequate (Brzev 2007; San Bartolomé and Quiun 2008; Brzev et al. 2010; Meli et al. 2011). CM researchers have published guidelines to address concerns with under-designing and construction deficiencies in developing countries (Meli et al. 2011).

The proposed method combines the methods of Guzmán and Escobar (2010), Tena-Colunga and Cano-Licona (2010), and Brzev et al. (2015) to create an integrated analysis and design tool that is useful for architects without the means to perform analysis themselves. This thesis aims to bridge the gap between rigorous structural analyses that involve complex computation and low-tech construction guidelines for CM that don't perform any seismic analysis and are restrictive on the architectural design. An integrated design and analysis approach that is simple enough to be conducted by a builder without engineering knowledge yet thorough enough to perform seismic analysis given a wide range of architectural forms is the ultimate objective.

3 Current Seismic Analysis Approaches for CM Buildings

3.1 Introduction

Since confined masonry is attractive for low-tech construction in seismic regions, existing guidelines pertain mostly to construction and not to seismic analysis (Ali 2005; Blondet 2005; Brzev 2007; Totten 2010; Schacher 2011; Iyer et al. 2012). This research seeks to expand on existing CM guidelines by incorporating seismic analysis into the design methodology while still maintaining a practical perspective for architects and technicians without an engineering background.

During an earthquake, masonry structures resist the effects of induced lateral loads through shear stresses in the walls parallel to the direction of the applied load. Seismic analysis of masonry buildings is a complex task due to the non-homogeneous nature of masonry and non-linear behavior in the post-cracking stage. This is further complicated in the case of composite masonry systems such as CM, where the masonry and RCC components work in unison. Numerous studies on seismic analysis approaches for masonry buildings have been performed, and an overview and comparison of the Equivalent Frame Model (EFM) and Finite Element Method (FEM) was presented by Kappos et al. (2002).

3.2 Equivalent Frame and Finite Element Models

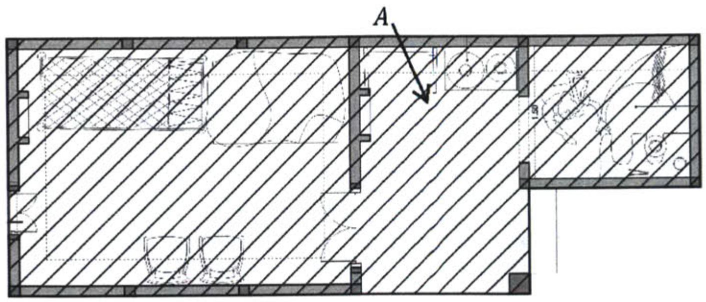
Analysis methods for confined masonry buildings vary in their complexity from simple procedures that can be performed by hand to determine the shear demand on each wall, to more rigorous methods that involve micro- or macro-modelling of structural components and require computer-based analysis. An example of a complex analysis approach uses the EFM, also known as the Wide Column Model. In this procedure one models a CM wall as an equivalent column located at the wall geometric center with lateral stiffness based on the material properties and geometry of the masonry panel and transformed RCC tie-column sections (tie-columns are transformed into equivalent masonry sections based on the relative moduli of elasticity of the materials) (Guzmán and Escobar 2010). EFM can be used to perform both elastic and non-linear analyses of confined masonry buildings (Terán-Gilmore et al. 2009). EFM can take into account flexural effects and is suitable for analysis of medium-rise buildings, but it requires advanced analysis skills, especially related to modelling of walls with openings. Micro-modelling approaches such as FEM have also been used for seismic analysis of confined masonry buildings. FEM uses computer software to analyse a structure and is computationally expensive and also requires technical knowledge to accurately model structures and their loading, hence for masonry it is mainly only used in research or for high profile projects where the extra cost is acceptable (Kappos et al. 2002).

3.3 Simplified Method for Seismic Analysis of Masonry Buildings

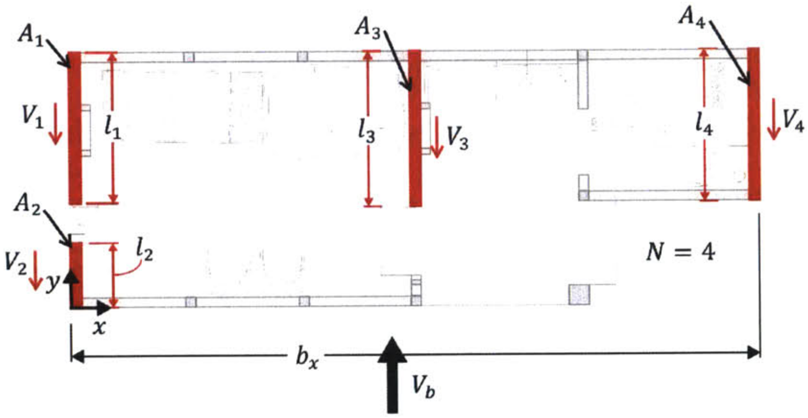
A less complex method, known as the Simplified Method for Seismic Analysis (SMSA), has been used for seismic design of confined masonry buildings since the 1970s and was allowed by the Mexican masonry code (NTC-M 2004). This procedure is based on Mexican practice by Meli (1994) and Meli et al. (2011). It was expanded upon for application in India by Brzev et al. (2015) and is employed in this thesis.

The SMSA determines the required wall density for a building; it assumes rigid floor diaphragms (e.g. RC floor slabs) and ignores torsional effects (see Fig. 3.1), therefore it applies only to buildings with regular plan shapes. It also assumes that shear stresses govern, and flexural effects are disregarded in the wall design. Due to its simplicity and modest computational requirements, the SMSA is suitable for seismic design of low-rise regular buildings only, such as single-family housing. It should be noted that NTC-M (2004) restricts the SMSA applicability to buildings with height less than 13 meters, while Meli et al. (2011) recommend a 6 meter height limit. An advantage of the SMSA over alternative analysis methods such as EFM and FEM is that it is an integrated analysis and design approach, which is one reason it is used in the current method. However, the current research seeks to expand on this method to make it applicable to more complex, irregular buildings.

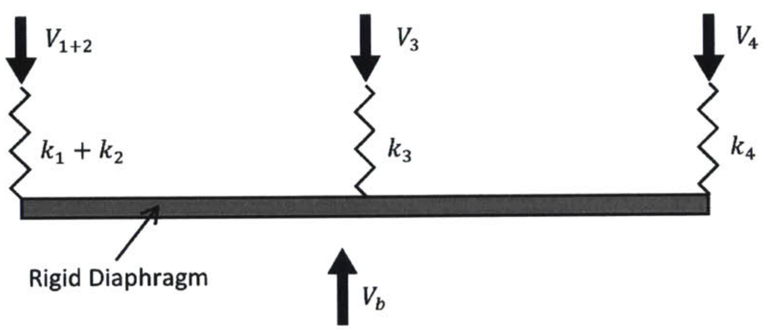
Fig. 3.1. Visualization of the concept behind the SMSA. Walls oriented in the direction of the applied seismic load resist the load in proportion with their relative stiffness to one another.



(a) Plan of an Actual Confined Masonry Building



(b) Shear Walls Resisting Seismic Load in y-direction



(c) Idealized Seismic Analysis Model

3.4 Seismic Analysis of Irregular Masonry Buildings

Walls in buildings with irregular plan shapes, or buildings where the plan is regular but the wall layout is irregular, experience an increase in internal lateral forces and deformations due to torsional effects caused by building eccentricity. Eccentricity is the distance between a structure's Center of Mass (C_M), where the seismic load is applied, and Center of Rigidity (C_R), where lateral loads are resisted, see Fig. 3.2. A higher level of irregularity usually leads to larger eccentricity and hence an amplified torsional effect. The seismic design of irregular buildings is covered in several references, such as Naeim (2000). The goal is to calculate the increase in internal forces in the structural members such as walls due to torsional effects. This analysis is usually complex and requires advanced technical skills and computational tools. However, it is recognized that most buildings are irregular to an extent and torsional effects must be considered. A procedure for seismic analysis of irregular buildings developed by Escobar et al. (2004), Escobar et al. (2008) and presented by Guzmán and Escobar (2010) is considered in this study because it gives a simple factor for each wall which captures the increase in the shear demand due to torsion. This method alone performs analysis but not design from a base structural requirement standpoint. It is therefore proposed to use this method in conjunction with the SMSA, which will provide the base structural design, and allow for a wide range of architectural forms.

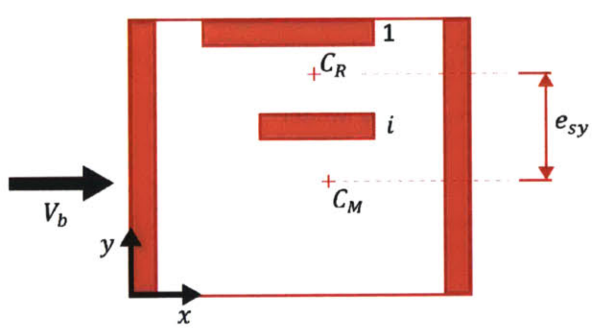


Fig. 3.2. Illustration of eccentricity as the distance between the C_M and the C_R. Note the base shear is applied in line with the C_M and the C_R is located closer to the structural walls resisting the base shear in the x-direction.

4 Seismic Design Procedure for Confined Masonry Buildings

4.1 Introduction

The proposed method can be used to check whether the wall layout and dimensions (length, thickness) are adequate for a given confined masonry building with a regular or irregular plan shape. First, the SMSA is used to determine a **preliminary** value for the required wall density. This assumes a square plan with the footprint area, masonry strength, number of stories, and seismic zone as inputs. Next, this procedure adjusts for the aspect ratio of the actual design building plan (still using the SMSA). The aspect ratio is taken as the ratio of the shorter building dimension, W, to the longer building dimension in the orthogonal direction, L ($W\!:L$, where $W \leq L$). A parametric study was conducted using the SMSA and it was determined that if the aspect ratio is less than 1:1 but greater than 2:3, the wall density for the shorter dimension can be conservatively increased by 2% (that is, multiplied by 1.02), and if it is less than 2:3 but greater than or equal to 1:3 can be increased by 8% (multiplied by 1.08), see Equation (1). If the building is regular (rectangular) in plan, the wall density determined from the SM and multiplied by the appropriate aspect ratio factor (for the shorter dimension only) is the final design requirement.

$$(1) \quad f_{AR} = \begin{cases} 1.0 \; if \; W\!:L = 1 \\ 1.02 \; if \; 1 > W\!:L \geq 2/3 \\ 1.08 \; if \; \frac{2}{3} > W\!:L \geq 1/3 \end{cases}$$

Where f_{AR} is the amplification factor for the building plan aspect ratio.

For buildings with irregular plan shapes the procedure then involves a torsional analysis given the actual building geometry, assuming only perimeter walls. The method by Guzmán and Escobar (2010) is used to determine a torsional amplification factor (FAT factor) for each wall. The maximum FAT factor in each direction is applied as a governing factor to determine the design wall density for the direction of the seismic force applied. A flow chart of the full method is found in section B.1 of Appendix B.

For this method to apply, the following assumptions must be followed:

- The procedure applies only to buildings up to and including three stories tall.
- It is assumed that the aspect ratio is greater than or equal to 1:3 ($W\!:L \geq 1\!:3$).
- It is assumed that the structural walls are continuous throughout the building height.
- It is assumed that there are at least 2 lines of structural walls in each direction.
- Floors and roofs are assumed to act as rigid diaphragms (there is uniform inter-story displacement).

4.2 SMSA Design Procedure

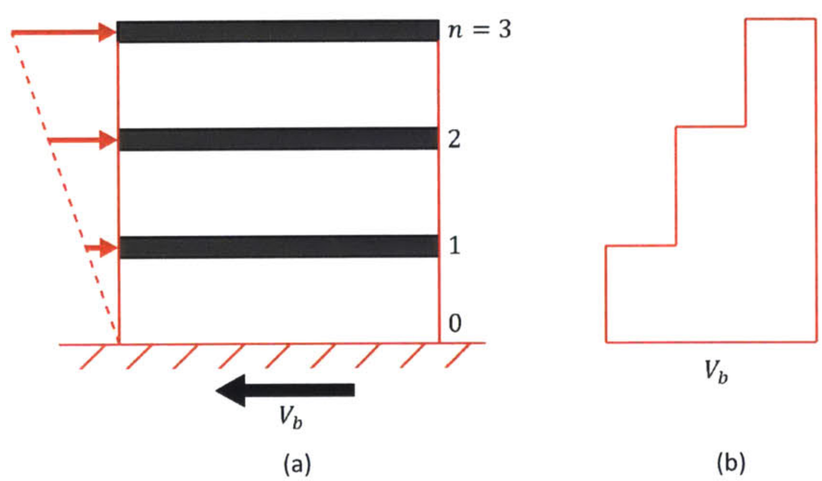


Fig. 4.1. Seismic force distribution along the building height (a) and the total shear force resisted by each story (b).

Consider the confined masonry building shown in Fig. 3.1. The SMSA estimates the required amount of walls, expressed in terms of the wall density ratio, d (%), in the specified direction of the building plan for given seismic hazard and soil conditions.

(2) $$d = \sum_{i=1}^{N} A_i \Big/ A$$

Where A_i is the cross sectional area in plan of wall i, A is the footprint area of the building, and N is the number of structural walls at the floor level in the direction of analysis.

This is accomplished by comparing seismic demand, V_b, that is, shear force acting at a story level and the corresponding shear capacity of the story (V_R), as shown in Equation (3).

(3) $LF \times V_b \leq V_R$

Since the seismic demand in wall buildings increases from the top towards the base, the analysis is usually performed at the base level of the building where seismic demand is equal to the seismic base shear force (V_b), see Fig. 4.1. It should be noted that SMSA can be used according to both the Allowable Stress Design (ASD) method and the Load and Resistance Factor Design (LRFD) method; the latter will be followed in this study. As a result, a load factor (LF) is applied to V_b, and a material resistance factor (ϕ) is applied to masonry shear resistance in the V_R equation.

In the current method the simplified method for seismic analysis, as described by Brzev et al. (2015), is used to determine a preliminary value for the required wall density in each direction by assuming a square plan shape. The design wall density is the final output of this tool. If the required length of confined masonry walls is the desired output, then one must also input wall thickness into this calculation, see Equation (10).

Consider a confined masonry building with a regular plan shape and wall layout. The seismic base shear force (V_b) can be expressed as a product of the seismic coefficient (A_h) and the seismic weight (W):

(4) $V_b = A_h \times W$

Where A_h depends on the seismic hazard, the type of soil, the building importance, fundamental period, etc. The seismic weight (W) can be expressed as a product of the average weight per unit floor area w, the actual floor area A, and the number of stories n.

(5) $W = n \times (w \times A)$

In a building with rigid diaphragms the shear force V_i resisted by wall i at a specific floor level is proportional to its stiffness k_i, see Fig. 3.1 (c). However, since the SMSA assumes that the wall behavior is shear-dominant, the stiffness k is proportional to the wall area A_i based on the fundamental principles of mechanics of solids, that is,

$$(6) \quad k_i = \frac{G \times F_i \times A_i}{H_i}$$

Where $G = 0.4E_m$, the shape factor $F = 1.2$ for rectangular sections, and H = wall height.

The shear capacity of a regular building at a particular floor level (V_R) (see Equation (3)) can be determined based on the sum of shear resistances for individual walls at that level. It is assumed that shear resistance of a wall is equal to the product of masonry shear resistance (v_m) and the wall cross-sectional area A_i (note that material resistance factor ϕ is applied to v_m). However, the sum of the cross-sectional areas can be expressed in terms of the wall density d, as follows:

$$(7) \quad V_R = \phi \times (v_m \sum_{i=1}^{N} A_i) = \phi \times v_m \times d \times A$$

It should be noted that the SMSA assumes that all walls have equal shear strength v_m for the shear capacity calculation. In this study masonry shear strength, v_m, is determined as function of the compressive strength f'_m without considering other factors such as the effects of axial precompression or the shear span ratio (see Chapter 5 for more details):

$$(8) \quad v_m = 0.18 \times \sqrt{f'_m}$$

The required wall density index, d, can be determined from Equation (3) as follows:

$$(9) \quad d = \frac{LF \times A_h \times w \times n}{\phi \times v_m}$$

The above equation can be further simplified when the wall thickness, t, is constant, thus the total required wall length, l_r, for a specific floor level and direction can be found:

(10) $l_r = \dfrac{A \times d}{t}$

Where $l_r \geq l = \sum l_i$ (see Fig. 3.1 (b))

For regular buildings the wall density determined in Equation (9), multiplied by the appropriate aspect ratio factor (f_{AR}) for the shorter dimension is sufficient and conservative for design. However for buildings with more complex plan shapes, torsional effects must be taken into account.

4.3 Simplified Method for Irregular Buildings

4.3.1 Introduction – Original Method by Escobar

The method proposed by Escobar et al. (2004, 2008) is used to account for torsional effects in irregular buildings. Some simplifying techniques are applied to reduce the calculation effort and make the design method applicable to a wider range of designs. The method uses an Amplification Factor for Torsion (FAT) to account for an increase in the shear demand in each wall due to torsional effects. The FAT accounts for the building eccentricity associated with irregularity in plan shape, or the effect of a non-symmetric wall layout with respect to the geometric center of the floor plan, or both. The design method presented in this study seeks to find a critical FAT value for each orthogonal direction of the building in order to determine the required wall density for that direction. The underlying concepts of the method are explained in the following section.

4.3.2 Procedure

Consider a building with an irregular wall layout of masonry shear walls shown Fig. 4.2 (a). In an irregular building with torsional effects the total shear force in each wall, V_{tot_i}, is equal to the sum of the direct shear force, V_{d_i} (without considering torsional effects), and torsional shear force, V_{t_i}:

(11) $V_{tot_i} = V_{d_i} + V_{t_i}$

It is assumed that the building has rigid diaphragms, thus a direct seismic force, V_{d_i}, in wall i is distributed in proportion to its stiffness relative to the sum of stiffnesses of all walls aligned in that direction. The seismic story force is equal to V_b since the analysis is performed at the base of the building where the seismic forces are largest.

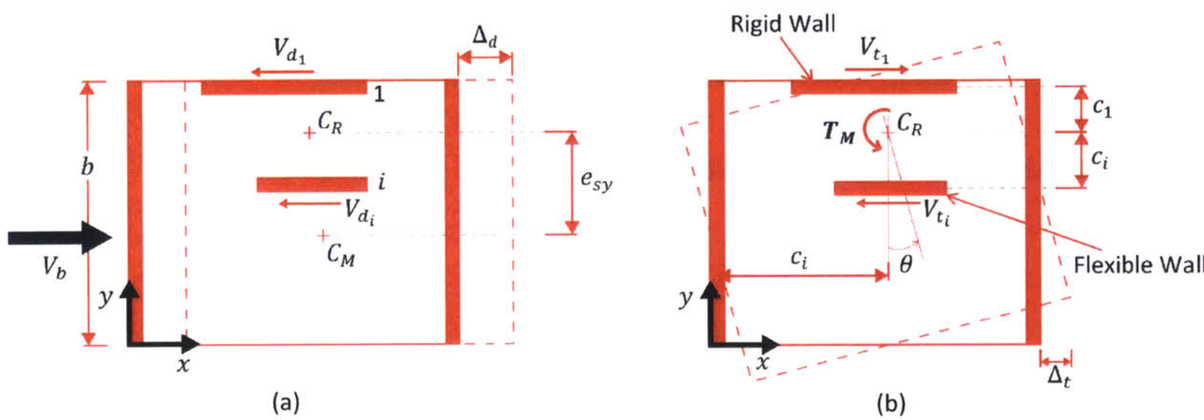


Fig. 4.2. Wall direct shear forces, V_d, (a) and torsional shear forces, V_t, (b) and the corresponding characteristic story displacements.

(12) $$V_{d_i} = \left(\frac{k_i}{\sum k_i}\right) \times V_b$$

Where V_b is the design base shear determined from the SMSA and k_i is the shear stiffness of wall i, see Equation (6).

If torsional effects are ignored, the inter-story displacement, Δ, due to force V_b at the base level of the building can be determined from the seismic shear force and the total story stiffness (equal to the sum of the individual wall stiffnesses):

(13) $$\Delta = \frac{V_b}{\sum k_i}$$

The torsional component of the shear force, V_t, is induced by the torsional moment T_M, which is equal to the product of the applied seismic force V_b and static eccentricity, e_s, see Fig. 4.2 (b). The eccentricity occurs when the center of mass (C_M) does not coincide with the center of rigidity (C_R) at a specific level in a building, and likewise is the distance between these two locations. Fig. 4.2 (a) shows static eccentricity for direction y, e_{sy}, in the building. See section B.3 of Appendix B for instructions on how to calculate the center of rigidity.

It should be noted that seismic codes in most countries consider the design eccentricity, e_d, as the sum of the static eccentricity, e_s, and the accidental eccentricity which is expressed as a fraction of b, therefore

$$(14) \quad \begin{cases} e_d = \alpha e_s + \beta b \\ \quad\quad or \\ e_d = \delta e_s - \beta b \end{cases}$$

Where α = multiplier for static eccentricity usually taken as 1.5 (and $\alpha \geq 1.0$) when accidental eccentricity is positive, and δ = multiplier for static eccentricity when the accidental eccentricity is negative, usually equal to 1.0.

The FAT factor represents the ratio of the total seismic shear force to the direct shear force, that is:

$$(15) \quad FAT_i = \frac{V_{tot_i}}{V_{d_i}}$$

According to this method, a wall is considered as *flexible* (*f*) if it is located on the same side as the center of mass (C_M) with respect to the center of rigidity (C_R), and as *rigid* (*R*) otherwise, see Fig. 4.3. For elements classified as *flexible*, Equation (16) is used, and for *rigid* elements Equation (17) is used to determine the FAT value.

(16) $FATf_i = 1 + \frac{\zeta_i}{\rho^2}(\beta + \alpha e)$

(17) $\begin{cases} FATr_i = 1 + \frac{\zeta_i}{\rho^2}(\beta - \delta e) & \delta e < \beta \\ FATr_i = 1 & \delta e \geq \beta \end{cases}$

Where e is the normalized eccentricity perpendicular to the direction of the applied load:

(18) $e = {|e_s|}/{b}$

Where e_s is the static eccentricity perpendicular to the direction of the applied load.

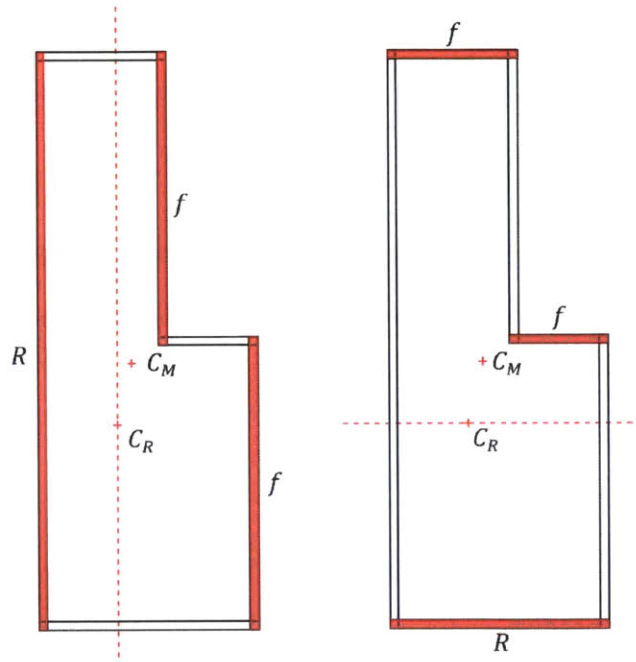


Fig. 4.3. Walls are considered *flexible, f,* if they are on the same side of the building plan as the center of mass, C_M, from the reference line through the center of rigidity, C_R, and as *rigid, R*, otherwise, as shown above.

The ζ_i factor depends on the position of wall i relative to the center of rigidity and building plan dimension b perpendicular to the direction of applied seismic force, see Fig. 4.2 (a). and ρ is the normalized radius of gyration:

(19) $\zeta_i = {c_i}/{b}$

Where c_i is the distance of wall i to the center of rigidity, (C_R), see Fig. 4.2 (b).

The factor β accounts for accidental eccentricity, which is typically expressed as a fraction of b. In most countries the seismic code prescribes a β value in the range of 0.05 to 0.1.

Given the rigid diaphragm assumption made in the SMSA which constitutes that the direct shear force and deflection withstood by each wall is directly proportional to its stiffness, the equation for the normalized radius of gyration can be presented as:

(20) $\rho = \frac{1}{b}\sqrt{\frac{k_\theta}{\sum k_i}}$

Where k_θ is the torsional stiffness:

$$(21) \quad k_\theta = \sum k_i \times c_i$$

Under circumstances similar to those of the current study where a shear-dominant behavior is assumed, that is, the stiffness is directly proportional to the wall cross sectional area, this can be written as:

$$(22) \quad \rho = \frac{1}{b}\sqrt{\frac{\sum A_i \cdot c_i^2}{\sum A_{xi}}}$$

Where A_{xi} and A_{yi} = cross sectional areas of walls in the x- and y-directions

Furthermore, if the thickness of all walls is constant, this can be further simplified to:

$$(23) \quad \rho = \frac{1}{b}\sqrt{\frac{\sum l_i \cdot c_i^2}{\sum l_{xi}}}$$

$$(24) \quad A_i = l_i \cdot t_i$$

4.3.3 Simplifications

The original design analysis approach by Escobar et al. (2004, 2008) has been modified to determine the required wall density or length for buildings with irregular plan shapes. After the FAT value is found for each wall, the maximum (critical) value in each direction, FAT, is applied to the wall density in that direction. This value is used to determine the total wall required length l_d or density d_d:

$$(25) \quad \begin{cases} l_d = FAT \times l_r \\ \quad or \\ d_d = FAT \times d \end{cases}$$

The following simplifying assumptions are made to reduce the calculation effort and make the design method useful for a wider range of field applications: i) the method is limited to buildings with plan geometries of a finite variety of shapes, defined as the four shapes shown in Fig. 4.4, and ii) the interior layout of the building is unknown, therefore the eccentricity calculations are performed assuming all of the walls are aligned along the perimeter of the building and the entire perimeter consists of structural walls, see Fig. 4.4. It is expected that the latter assumption will end up being false, that is, in the actual design there will be interior walls and the full perimeter of the building will not consist entirely of structural walls (due to openings). However, since the perimeter walls are the most critical in a building for torsional considerations this assumption is considered conservative.

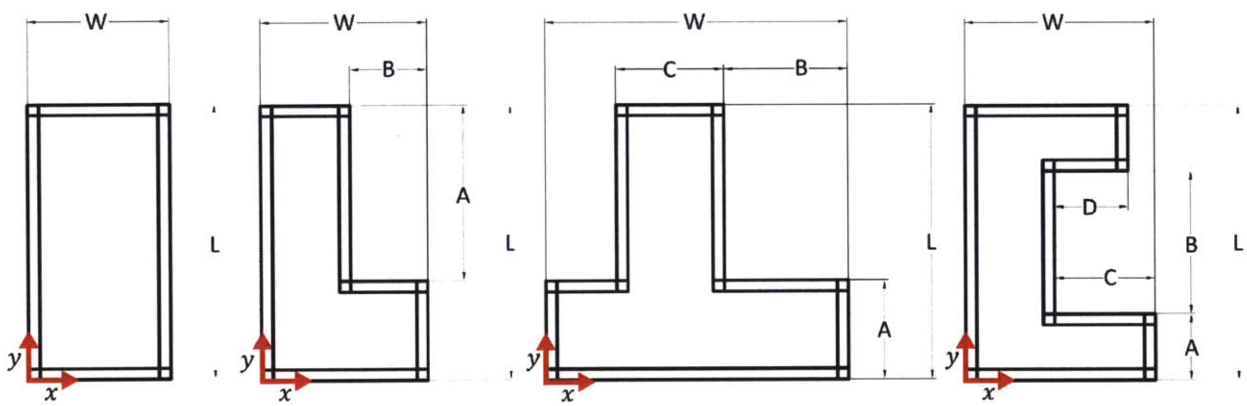


Fig. 4.4. The four basic plan geometries that the proposed design method is tailored to. It is possible to find the eccentricity and *FAT* values for any combination of the dimensions shown.

The design procedure can be summarized as follows:

1) A unique *FAT* factor is calculated for each of the perimeter walls. Since the goal is to come up with one factor for the entire building in each direction and it is uncertain where the architect will place additional walls, the critical factors are chosen to be applied to the entire building. That is, the largest *FAT* factor in each direction is chosen. Although the *FAT* factor represents an increase in the shear demand, the SMSA directly relates shear demand to the required wall density, therefore the *FAT* can be directly applied directly to that value (d).

2) Once the architect has estimated the preliminary design requirement (wall length or wall density) from Equation (9), they will then input the actual design dimensions corresponding to those in Fig. 4.4 into a spreadsheet. The FAT factors for each direction are calculated in an Excel spreadsheet which is described in section B.2 of Appendix B. Separate spreadsheets that find the torsional factors are given for "L" shaped, "C" shaped, and "T" shaped plans. A guideline in the form of a physical booklet is also in development which will contain a finite number of predetermined FAT values for designs considered to encompass a sufficiently representative range of dwelling layouts commonly found in India. With this guideline the architect must match their actual design with the given one that is most similar. If they are unsure as to which one most closely matches their design, they are advised to select the more conservative one.

3) The architect applies the critical FAT value to the required wall density or length, see Equations (25).

4.4 Parametric Study

A parametric study was performed for 45 arbitrary building plan geometries (15 each of "L", "C", and "T" shaped geometries) and the results are presented in Fig. 4.5. The values for $\beta, \alpha,$ and δ were taken as prescribed in IS 1893 ($\beta = 0.05, \alpha = 1.5$, and $\delta = 1.0$). The main output is the FAT factor tested against two parameters: the normalized eccentricity, e/b, and the normalized radius of gyration squared, ρ^2, to investigate their relationships and verify the design method.

The first parametric study of the FAT against the normalized eccentricity, the results of which are shown in Fig. 4.5 (a), reveals a strong positive relationship between the parameter and the output. This chart of maximum FAT value versus e/b can be used to predict the increase in shear stresses in a building due to torsion if the normalized eccentricity is known. Furthermore, the even distribution of results on the FAT versus e/b graphs shows that the output relationship to eccentricity is not erratic in spite of the assumptions being made (e.g. perimeter walls only) and there are no strong outliers, therefore the design method provides reliable results for a wide range of eccentricities.

The second parametric study investigates the maximum FAT value versus the normalized radius of gyration squared, ρ^2, as shown Fig. 4.5 (b). The clear display of an inverse quadratic relationship again verifies the function of the design method. Another observation is that as the squared normalized radius of gyration increases past a certain value, the FAT factor plateaus at 1.1. The value for ρ^2 at which this happens is different for each of the three plan geometries. For "L" shaped building plans, the change in FAT occurs mostly between values of 0.2 and 0.4 for ρ^2, above which it plateaus. For "T" shaped buildings the FAT plateaus after ρ^2 values of approximately 0.5 and 0.3 for the x and y-directions, respectively. This occurs for "C" shaped plans at ρ^2 of approximately 0.35 and 0.6 for the x and y-directions, respectively. The value of 1.1 is therefore a viable FAT estimate for values of the squared normalized radius of gyration above those just stated and for the respective geometries and axis directions. Note that these values pertain specifically to the plan shapes and orientations shown on the graphs of Fig. 4.5 (b).

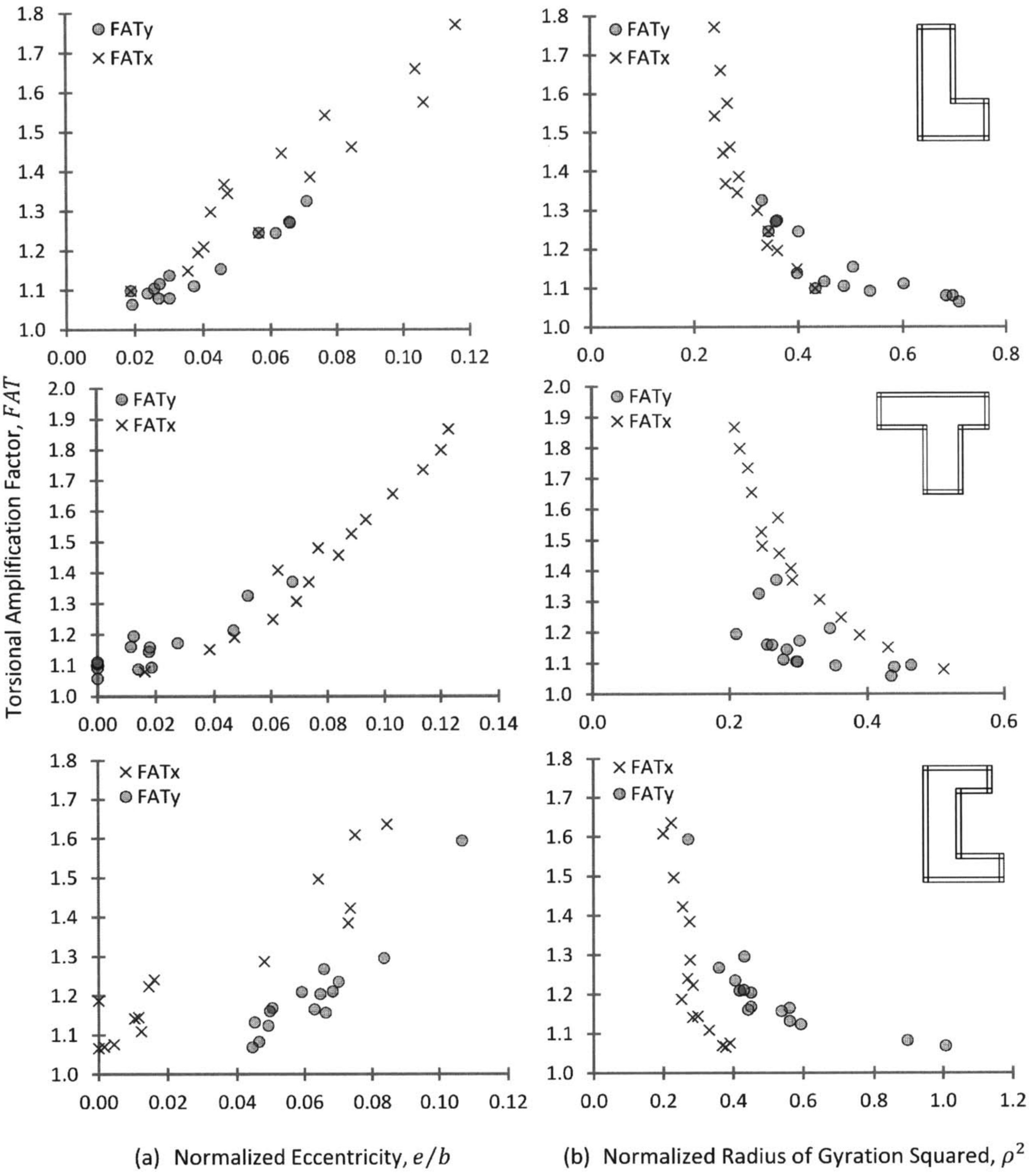


Fig. 4.5. Results from parametric studies of the current design method for 15 arbitrary "L", "T", and "C" shaped building plans.

4.5 Design Example – Rural Building in Gujarat, India

4.5.1 Introduction

The following example is intended to illustrate the effectiveness of the proposed method. The design used for this example is a confined masonry home for rural areas of Gujarat designed by the Ahmedabad based architecture firm People in Centre (PiC), see Fig. 4.6, the same one as shown in Fig. 3.1 (PiC 2014, http://www.peopleincentre.org). This home was designed for implementation in the Indira Awaas Yojana (IAY) federal housing project. The IAY project will provide affordable houses, subsidized by the government, to qualifying families across the country. PiC was tasked with designing homes to be built in the State of Gujarat so they came up with multiple designs which may be selected by the beneficiaries. As with any such program, cost is a key factor for the design and selection of homes, and the confined masonry design shown here is estimated at nearly 20% less expensive than their next cheapest option.

This is a real world example designed by an Indian architecture firm for widespread implementation. An in depth seismic analysis of the design was performed, with a few variables open to alteration to determine the required masonry compressive strength. The increase in shear demand for the walls due to torsional effects was determined using the conventional analysis method (Naeim 2000), and the thickness of the structural walls was the main variable considered. The researcher recommended a specific design which minimized the required masonry strength while maximizing the cost efficiency.

4.5.2 Example

The method proposed in the current study is used on the design (see Fig. 4.6) and the output requirement is compared to the recommended design determined from the detailed torsional analysis (Naeim 2000). The recommendation was based on the assumption that a second story would be added to the home in the future. The seismic design parameters were obtained from the Indian seismic design code IS 1893. The assumed site location is Bhuj, Gujarat (the area most affected by the 2001 Bhuj earthquake).

Design assumptions:

- Masonry compressive strength $f_m' = 2\ MPa$
- Response reduction factor $R = 3.0$ from Brzev et al. (2015) (according to IS 1893 $R = 1.5$ for unreinforced masonry)
- Type II Medium soil type
- Load factor $LF = 1.5$ and strength reduction factor $\phi = 0.5$ (Brzev et al. 2015)
- Per IS 1893 torsional considerations:

$$\beta = 0.05, \alpha = 1.5, \delta = 1.0 \rightarrow e_d = \begin{cases} 1.5e_s + 0.5b \\ e_s - 0.5b \end{cases}$$

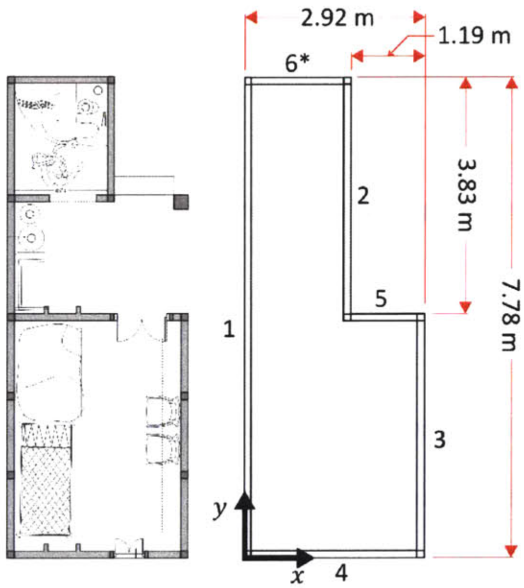


Fig. 4.6. Floor plan of the CM home designed by the Ahmedabad architecture firm People in Centre used in this example for the current design method.

*Critical member for design of walls in the x-direction. Calculation shown for this wall.

$$Area = 18.2\ m^2$$

$$H = 2.75\ m$$

$$A.R. = \frac{W}{L} = \frac{2.92\ m}{7.78\ m} = 0.38 \geq 0.33 \quad OK$$

$$n = 2\ stories$$

$$v_m = 0.18\sqrt{2.0\ MPa} = 0.25\ MPa$$

$w = 8.0\ {}^{kN}/_{m^2}$, A conservative value, see Brzev et al. (2015)

The seismic coefficient A_h can be determined from the Indian building code IS 1893 as follows:

$$(26) \quad A_h = \frac{Z \times I \times S_a}{2 \times R \times g}$$

Where Z is the seismic zone factor, $Z/2$ represents the design basis earthquake, I is the importance factor, R is the response reduction factor, and S_a/g is the average response acceleration coefficient:

$Seismic\ Zone\ V \rightarrow Z = 0.36$, IS 1893 Table 2

$I = 1.0$, IS 1893 Table 6

$$(27) \quad S_a/g = \begin{cases} 1.0 + 15T & if\ 0.0 \leq T < 0.1 \\ 2.50 & if\ 0.1 \leq T \leq 0.55 \\ \frac{1.36}{T} & if\ 0.55 < T \leq 4.0 \end{cases}$$

Equation (27) was taken from IS 1893 Cl 6.4.5, and T is the natural period of the structure:

$$(28) \quad T = \frac{0.09 \times H \times n}{\sqrt{D}}$$

Equation (28) was taken from IS 1893 Appendix, Amendment to section 7.6.2, H is the story height, and D is the base dimension length along the direction of the applied force

$$Base\ dimension\ (square\ plan)\ D = \sqrt{18.2\ m^2} = 4.27\ m$$

$$T = \frac{0.09(2.75\ m)(2)}{\sqrt{4.27\ m}} = 0.24\ s$$

$$T = 0.24\ s, 0.1 \leq T \leq 0.55 \rightarrow \frac{S_a}{g} = 2.50$$

$$A_h = \frac{(0.36)(1.0)}{(2)(3.0)}(2.50) = 0.15$$

$$d = \frac{1.5(0.15)(8.0\ ^{kN}/_{m^2})(2)}{0.5(0.25\ ^{N}/_{mm^2})(1000)} = 2.9\%$$

The following shows the calculation of the torsional amplification factor for the critical wall in the shorter direction (*Wall* 6).

$$\zeta_6 = \frac{|4.67\ m|}{(7.78\ m)} = 0.60, \text{see } \mathbf{Fig. 4.7}.$$

$$e = \frac{|-0.49\ m|}{(7.78\ m)} = 0.063, \text{see } \mathbf{Fig. 4.7}.$$

$$\rho_x = \frac{1}{(7.78\ m)}\sqrt{\frac{(66.81\ m^3) + (23.95\ m^3)}{(5.84\ m)}} = 0.51$$

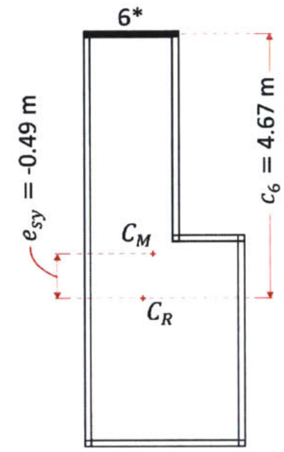


Fig. 4.7. Dist. of *Wall 6* to the center of rigidity, c_6, and static eccentricity in the y-direction, e_{sy}, for use in Equations (19) and (18), respectively.

Wall 6 is on the same side as the center of mass with respect to the center of rigidity, therefore it is *flexible* and Equation (16) will be used to determine the FAT value for torsion for *Wall 6*:

$$FATf_6 = 1 + \frac{0.60}{(0.51)^2}(0.05 + 1.5(0.063)) = 1.34$$

$\frac{1}{3} \leq \frac{W}{L} \leq \frac{2}{3} \rightarrow\ f_{AR} = 1.08$ (8% increase applied to account for $A.R.$ not applied to the longer direction):

$$FAT_{xMAX} = (1.08)(1.34) = 1.45$$

$$FAT_{yMAX} = 1.08 \text{ (calculation not shown here)}$$

$$d_x = (1.45)(2.9\%) = 4.2\%$$

$$d_y = (1.08)(2.9\%) = 3.1\%$$

Table 2. Wall Density Comparison

	SMSA	Proposed Method	Conventional Analysis (% Difference)	Architectural Design (% Difference)
d_x	2.9	4.2	3.1 (-26%)	4.0 (-5%)
d_y	2.9	3.1	2.9 (-6%)	8.7* (181%)

*The wall density of the actual design for the y-direction is overdesigned due to the long dimension of the building, providing excess support.

It should be noted that the FAT values can be determined from the respective chart in Fig. 4.5 for "L" shaped plans. The corresponding chart from Fig. 4.5 is shown in Fig. 4.8 with a trend line for the FAT_x values; it can be seen that for an e/b value of 0.063, the FAT_x according to the trend line is near the 1.34 value found from the calculation.

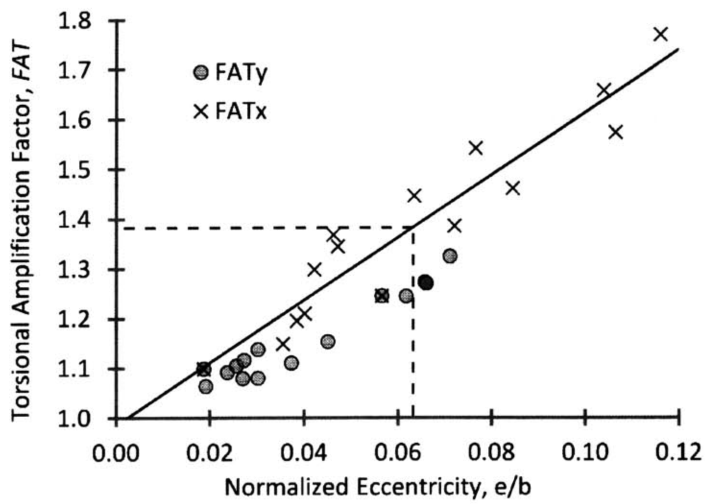


Fig. 4.8. FAT versus e/b chart for "L" shaped buildings, also shown in Fig. 4.5 (a).

The architectural design was based on the geometry of the home and a conventional torsional analysis performed according to the method from Naeim (2000) provided the corresponding results. It can be seen from Table 2 that the required wall density according to the proposed method for each direction is more conservative than that determined from conventional analysis. The actual design wall density for the y-direction was naturally overdesigned because of the building geometry which allowed for a plethora of confined masonry wall length in that direction. The wall density for the y-direction found using the proposed method is closer to that for the conventional analysis than in the x-direction because the walls in the y-direction are less affected by torsion. This case study shows that the proposed method provides conservative design requirements compared to other, more rigorous analyses which is expected considering that the proposed method is simplified.

4.6 Summary

The proposed design procedure developed for the seismic design of low-rise confined masonry buildings was discussed in detail. The procedure first uses the SMSA to determine a preliminary required wall density. Subsequently, a method originally developed by Escobar et al. (2004, 2008) is used with simplifying assumptions to determine the torsional amplification factor (FAT) which is used as a multiplier for the wall density in each orthogonal direction of the building plan. The proposed method was used to perform a parametric study on a number of arbitrary designs. A well dispersed, positive relationship is seen between the building plan eccentricity and the torsional amplification factors. It is also observed that as the normalized radius of gyration, ρ^2, increases, the FAT converges at 1.1. The method was also applied to evaluate a design which was previously analyzed using a conventional torsional analysis. The design requirements found from the proposed method are conservative, as expected.

The design and analysis approach developed in this research can be used to rapidly determine the structural design requirements for a large variety of low-rise CM buildings. The two assumptions made are: i) the variety of plan shape options are limited, and ii) the eccentricity calculation assumes perimeter walls only. These assumptions allow for a simplified design and analysis method (SMSA) to be combined with a more complex torsional analysis (Escobar 2004, 2008) in a streamlined, simplified method that is rapidly repeatable. The power of this streamlined method is captured in an Excel spreadsheet (see Fig. B.1Fig. B.2) which will be available for download on the National Information Centre for Earthquake Engineering (NICEE) and Confined Masonry Network (CMN) websites and distributed by these organizations wherever possible. For applications where Excel cannot be used, the method will be used to create design tables and charts for a physical guideline for the design of CM buildings in low-tech construction environments. Both of which are usable by builders without technical expertise.

5 Material Properties and Experimentation

As with any structural analysis it is critical to assess the material strengths that apply, and in the context of India and other developing countries material properties are even more important because they are highly variable and often low quality. Many studies exist that investigate the behavior of bricks and mortar in masonry under compression and relate the effects of their properties to one another and to the overall behavior of the masonry. However, there is limited experimental data related to low strength bricks, despite the fact that in some regions of India it is common to use clay bricks with compressive strength lower than 3 MPa. This is partially due to the fact that such data is difficult to obtain because there is a lack of experimental equipment in places where low strength bricks are used. The experiments presented in this thesis seek to partially fill this research gap to inform more context conscious building practices in developing countries and promote future studies on this topic.

In India bricks are often hand molded and produced using traditional firing techniques in one of the country's 100,000 kilns (Maithel et al. 2012). Poor quality clay and variability and lack of quality control in the manufacturing process are the causes of low-strength bricks. Since masonry is the most common material for housing construction in the country (Iyer et al. 2012), this is a very important topic and the reason for this study.

(a) (b) (c)

Fig. 5.1. Local masons constructing prisms (a), curing masonry prism specimens (b), and a prism under compression testing (c) more details and experimentation photos can be found in Appendix C.

Masonry compressive strength is important for the current analysis because it is critical for determining shear strength, which governs seismic resistance of low-rise masonry walls. In the proposed design method, Equation (8) relating shear capacity to compressive strength was chosen based on a comparison of equations used by different building codes to model this relationship, Fig. B.4 in Appendix B shows this comparison. Equation (8) relies solely upon masonry compressive strength and does not take into account the effect of compressive stress as some do. This is because the proposed method applies only to low-rise buildings subjected to low compressive stress (Meli et al. 2011). The equation was also used by Brzev et al. (2015).

Experimentation was conducted to test the effects of varying mortar types on masonry compressive strength when low strength bricks are used (see Fig. 5.1). The materials were locally acquired in Gujarat, India and local masons were hired to construct the prism specimens. The bricks were common "high strength" and "low strength" bricks as defined by the dealer, but in reality both were lower than 2.5 MPa strength on average. The minimum brick compressive strength prescribed by the Indian masonry code IS 1905: Code of Practice for Structural Use of Unreinforced Masonry is 3.5 MPa, which is lower than the minimum value from other building codes, even in developing countries such as Peru (min. 6.9 MPa) and Indonesia (min. 4.6 MPa) (Meli et al. 2011). The following three mortar mixes were used: i) 1:6 cement:sand ratio mortar, ii) 1:8 mortar, and iii) a mud mortar with no cement. These mortar mixes are commonly used in India, mud mortar being more common in rural areas.

The results of the testing reveal an inverse relationship between the cement content in the mortar and the compressive strength of the masonry, which is the opposite from the trend any building code defines. The mud mortar resulted in the strongest average prism strength, 41% stronger than the 1:6 mortar and 17% stronger than the 1:8 mortar prisms as shown in Fig. 5.2. Likewise the 1:8 mortar prisms were 21% stronger on average than the mortar with the highest cement content (1:6). Building codes do not address masonry with brick units of such low strength despite the fact that they are so commonly used in certain countries. Research studies have shown that masonry material behavior with such bricks is different from the common "stiff brick - soft mortar" assumption (Drysdale et al. 1994). Codes exclusively assume that the mortar is less stiff than the bricks and therefore under compression the mortar is in tri-axial compression as it tries to expand and the bricks are in bi-axial tension and axial compression (see Fig. C.1 in Appendix C). However, with low-strength bricks the mortar may have equal or greater stiffness than the bricks which leads to different structural behavior of these constituent materials.

It was hypothesized in this study that the softer mortars with lower cement content were able to deform more compatibly with the low strength bricks, whereas the stronger 1:6 mortar with higher stiffness caused stress concentrations in the brick, leading to earlier failure. It is further hypothesized that the lower cement quantity mortars were able to develop better bond with the bricks, increasing the compressive strength. This hypothesis is in line with the findings of previous studies by Samarasinghe and Lawrence (1992), Sarangapani (1992) and Gumaste et al. (2007). A detailed report of the experimentation, results, and conclusions behind the observed behavior can be found in Appendix C.

It is recommended based on these observations that with low quality bricks, a low cement mortar that is more compatible with the bricks be used in construction. This will lead to more resilient housing in low-tech construction environments that is less expensive and takes into account the reality of building materials that are being used instead of specifying mortar mix based on brick strengths specified by the building codes.

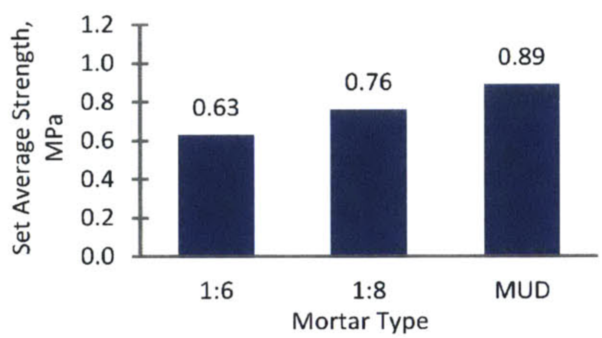


Fig. 5.2. Prism strengths by mortar type. Complete results can be found in Appendix C.

6 Summary and Conclusions

A seismic design procedure was developed that can rapidly generate structural design requirements for a large range of low-rise confined masonry buildings with regular and irregular plan configurations. This method combines two approaches for the design and analysis of CM buildings as an integrated approach and simplifies the calculation procedure to a minimum. Results from the proposed method show that it is conservative and applicable to buildings with a wide range of irregularities, with no severe outliers.

The assumptions made allow for a simpler and more rapidly reproducible calculation, however it is possible that the eccentricity calculated with the current method be less than that for the actual building design. These assumptions are: i) the method application is limited to a finite variety of plan shapes, and ii) only perimeter walls are considered for the eccentricity estimation. Since the method considers only perimeter walls, it is unlikely that the addition of interior walls and the removal of exterior walls for openings would result in a greater eccentricity, and if this did occur the difference would be minimal. Therefore conservative factors implemented in the procedure make such an occurrence irrelevant.

Experimentation shows higher masonry strength associated with lower mortar strength when bricks of lower strength than the 3.5 MPa minimum requirements of the Indian masonry code IS 1905 are used. A 41% higher average compressive strength was observed for prisms with mud mortar than prisms with a 1:6 cement:sand ratio mortar, and 21% higher strength was observed in prisms with 1:8 mortar than those with the higher strength 1:6 mortar. It is hypothesized that the higher strength mortars, which were stronger than the bricks, were also more stiff, leading to stress concentrations in the bricks which caused the earlier failure, whereas the lower strength mortar, with more comparable mechanical properties to the bricks, allowed for more deformation prior to failure. It is also hypothesized that the lower strength mortars developed a better bond with the bricks, also contributing to better masonry strength.

These observations suggest that when low strength bricks that are common in developing nations are used, mortars with low cement content are more appropriate and will result in masonry that is as strong, if not stronger, than a higher strength mortar would. Such a suggestion, if followed, would reduce the cost of building construction and lead to more resilient communities. It is the opposite of what any building code would recommend, however, the bricks being used do not follow the code and this is a reality that is not going to change in the near future.

6.1 Future Research Needs

6.1.1 Continuing Partner Collaboration

The partnerships created during this research are valuable and can lead to opportunities for future research. The author of this thesis is currently working with partners on a guideline for the design, analysis, and construction of low-rise confined masonry homes for low-tech environments that will incorporate the design method developed herein. This guideline will be published by the National Information Center of Earthquake Engineering (NICEE) at IIT Kanpur.

Future students at MIT can continue to collaborate with PiC to provide consultation on their confined masonry home designs to help ensure their approval by the governing board of housing projects. Continued collaboration with partners such as PiC, EERI and its Confined Masonry Network, IIT Kanpur, IIT Gandhinagar, NICEE, and experts on confined masonry and Indian construction, can lead to guidelines that better fit this context and to conduits for implementation of CM in India.

6.1.2 Confined Masonry Formwork

Construction challenges pose a hindrance for widespread implementation of CM in India. One piece of technology that could help solve this issue is formwork engineered specifically for confined masonry. In RCC frame construction, four pieces of wood are held together that support each other as the formwork for a concrete column. In CM, where the masonry is laid first, the masonry provides two of the formwork surfaces and the construction team must find a way to support the other two pieces as flush against the masonry as possible so as to prevent leakage of the concrete without damaging or disturbing the masonry.

This has shown to be a challenge, and the confined masonry construction of the new IIT Gandhinagar campus reveals this, where issues occurred with concrete seapage and inadequate distribution of the concrete in some cases see Fig. 6.1 (Basu et al. 2014, Jain 2015). Developing formwork technology that is self-supporting and can be rapidly removed and reused without damaging the masonry would be a valuable contribution to efforts to promote confined masonry construction which could also be a marketable product.

(a) (b) (c)

Fig. 6.1. CM construction at the new IIT Gandhinagar campus: concrete seapage results in unsightly conditions (a), contrastingly, inadequate filling of tie-columns results in less bonded area between concrete and masonry (b), and separation of cement and aggregate weakens beam-column joints (c).

6.1.3 Continued Experimentation

The author of this study was unable to perform deformation analyses on the bricks or mortar, therefore no data on the relative moduli of elasticity was collected. Furthermore, experimental data on masonry with such low quality bricks is very limited. There is enormous potential for future research in this area. Testing prisms on a larger scale with more precise technology and measuring capabilities would shed light on the behavior of masonry with low strength bricks, and develop firmer conclusions as to the reasons behind this behavior. Such research could help inform better construction practices in developing countries across the globe that would save lives and money. These implications will be powerful because they take into account the reality of informal building practices instead of hinging on the assumption of code adherence.

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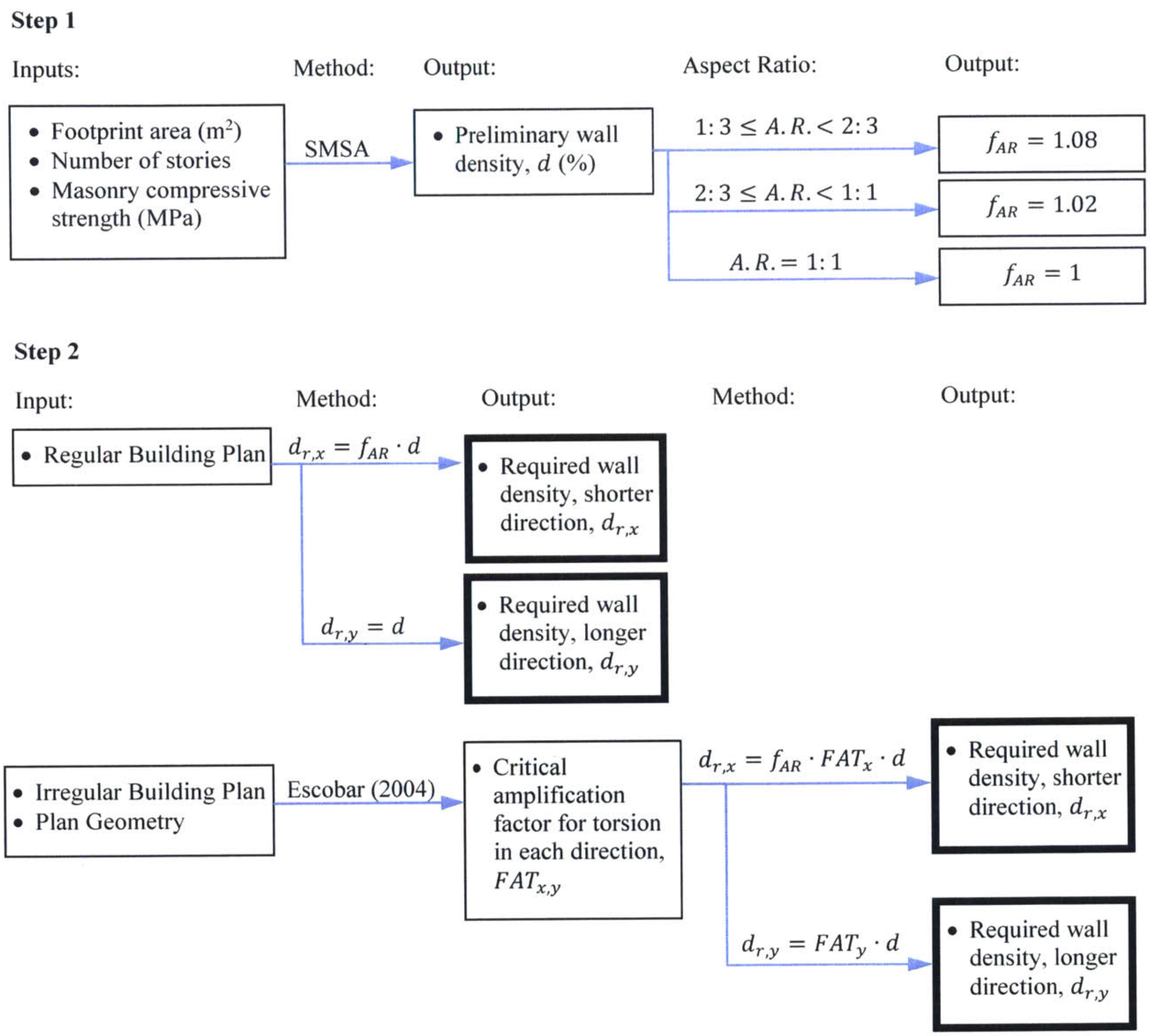
Appendix B – Supplementary Material for Design Method

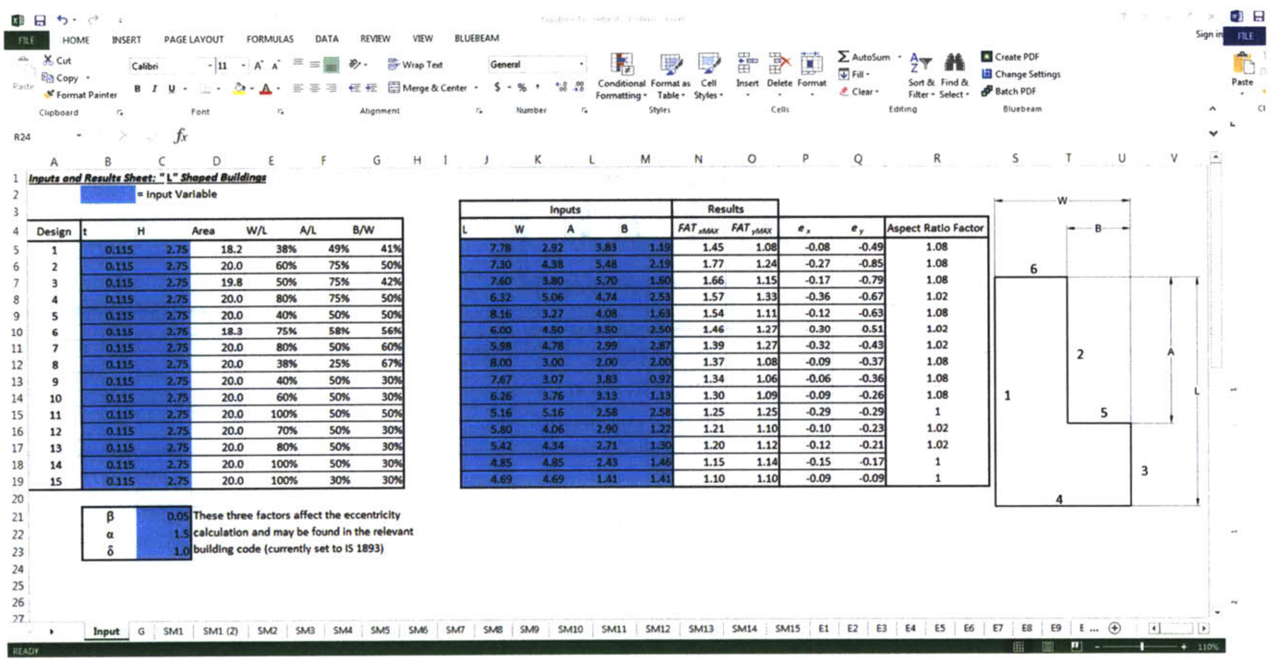
B.1 Method Flow Chart



Note: In this flow chart, x signifies the axis of the building plan with the shorter dimension and y signifies the axis with the longer dimension. Therefore the aspect ratio is the ratio of the longest dimension of the building's x-axis to the longest building dimension on the y-axis.

B.2 Design Method Spreadsheet

The following Fig. B.1Fig. B.2 are screenshots of a spreadsheet that has the full calculation procedure for the design method for "L" shaped buildings embedded in it. Similar spreadsheets exist for "T" and "C" shaped buildings as well. In the first sheet, labeled "Input", the user need only input the required dimensions as shown previously in Fig. 4.4 and the spreadsheet labeled "E[#]" will calculate the design structural requirement for design number "#", using the current method. The "Input" sheet has cells for 15 designs, which is what the spreadsheet is built to calculate simultaneously. The final output will also be displayed in the "Input" sheet next to the cells with the input values for the respective design. These spreadsheets will be available for download online at sites such as the NICEE website and will be distributed whenever possible to those who may find it useful.



Inputs and Results Sheet: "L" Shaped Buildings

= Input Variable

Design	t	H	Area	W/L	A/L	B/W
1	0.115	2.75	18.2	38%	49%	41%
2	0.115	2.75	20.0	60%	75%	50%
3	0.115	2.75	19.8	50%	75%	42%
4	0.115	2.75	20.0	80%	75%	50%
5	0.115	2.75	20.0	40%	50%	50%
6	0.115	2.75	18.3	75%	58%	56%
7	0.115	2.75	20.0	80%	50%	60%
8	0.115	2.75	20.0	38%	25%	67%
9	0.115	2.75	20.0	40%	50%	30%
10	0.115	2.75	20.0	60%	50%	30%
11	0.115	2.75	20.0	100%	50%	50%
12	0.115	2.75	20.0	70%	50%	30%
13	0.115	2.75	20.0	80%	50%	30%
14	0.115	2.75	20.0	100%	50%	30%
15	0.115	2.75	20.0	100%	30%	30%

Inputs				Results				
L	W	A	B	FAT_{xMAX}	FAT_{yMAX}	e_x	e_y	Aspect Ratio Factor
7.78	2.92	3.83	1.19	1.45	1.08	-0.08	-0.49	1.08
7.30	4.38	5.48	2.19	1.77	1.24	-0.27	-0.85	1.08
7.60	3.80	5.70	1.60	1.66	1.15	-0.17	-0.79	1.08
6.32	5.06	4.74	2.53	1.57	1.33	-0.36	-0.67	1.02
8.16	3.27	4.08	1.63	1.54	1.11	-0.12	-0.63	1.08
6.00	4.50	3.50	2.50	1.46	1.27	0.30	0.51	1.02
5.98	4.78	2.99	2.87	1.39	1.27	-0.32	-0.43	1.02
8.00	3.00	2.00	2.00	1.37	1.08	-0.09	-0.37	1.08
7.67	3.07	3.83	0.92	1.34	1.06	-0.06	-0.36	1.08
6.26	3.76	3.13	1.13	1.30	1.09	-0.09	-0.26	1.08
5.16	5.16	2.58	2.58	1.25	1.25	-0.29	-0.29	1
5.80	4.06	2.90	1.22	1.21	1.10	-0.10	-0.23	1.02
5.42	4.34	2.71	1.30	1.20	1.12	-0.12	-0.21	1.02
4.85	4.85	2.43	1.46	1.15	1.14	-0.15	-0.17	1
4.69	4.69	1.41	1.41	1.10	1.10	-0.09	-0.09	1

β	0.05
α	1.5
δ	1.0

These three factors affect the eccentricity calculation and may be found in the relevant building code (currently set to IS 1893)

(a)

Inputs				Results	
L	W	A	B	FAT_{xMAX}	FAT_{yMAX}
7.78	2.92	3.83	1.19	1.45	1.08
7.30	4.38	5.48	2.19	1.77	1.24
7.60	3.80	5.70	1.60	1.66	1.15
6.32	5.06	4.74	2.53	1.57	1.33
8.16	3.27	4.08	1.63	1.54	1.11
6.00	4.50	3.50	2.50	1.46	1.27
5.98	4.78	2.99	2.87	1.39	1.27
8.00	3.00	2.00	2.00	1.37	1.08
7.67	3.07	3.83	0.92	1.34	1.06
6.26	3.76	3.13	1.13	1.30	1.09
5.16	5.16	2.58	2.58	1.25	1.25
5.80	4.06	2.90	1.22	1.21	1.10
5.42	4.34	2.71	1.30	1.20	1.12
4.85	4.85	2.43	1.46	1.15	1.14
4.69	4.69	1.41	1.41	1.10	1.10

(b)

Fig. B.1. "Input" sheet of design method spreadsheet for "L" shaped buildings (a), and a close up of the inputs and design outputs side by side in the "Inputs" sheet for 15 separate designs (b). Shaded cells indicate inputs. This is the only part of the spreadsheet where a user is required to input and interpret values.

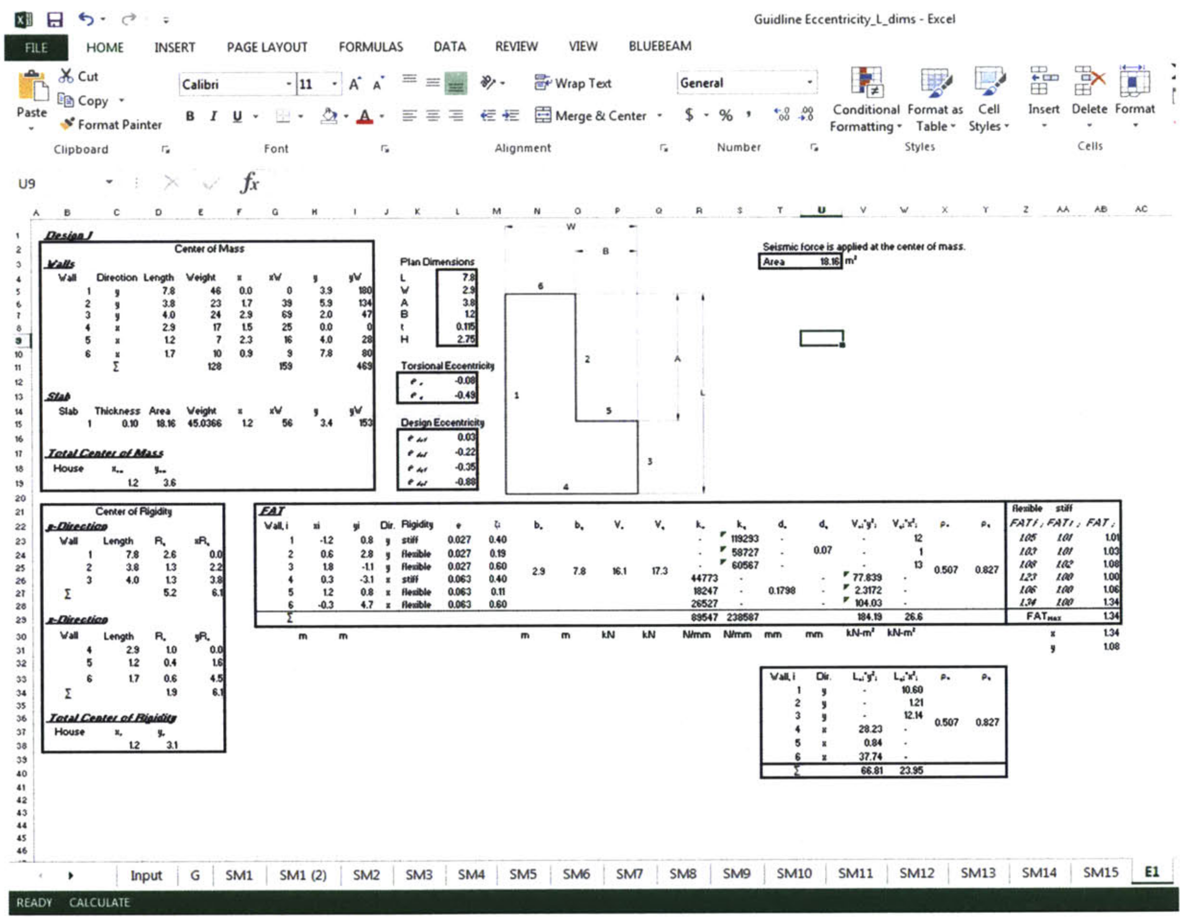


Fig. B.2. Sheet "E[#]" (sheet shown is "E1") that performs the design eccentricity and torsional amplification factor calculation for design number "#". No inputs are required in this sheet.

B.3 Finding the Center of Rigidity ($\boldsymbol{C_R}$)

In order to calculate the x and y coordinates of the center of rigidity of a story one may employ the following equations using the dimensions illustrated in Fig. B.3:

Wall	k	A
1	k_{y1}	A_{y1}
2	k_{y2}	A_{y2}
3	k_{y3}	A_{y3}

Subscript y indicates wall in the y-direction

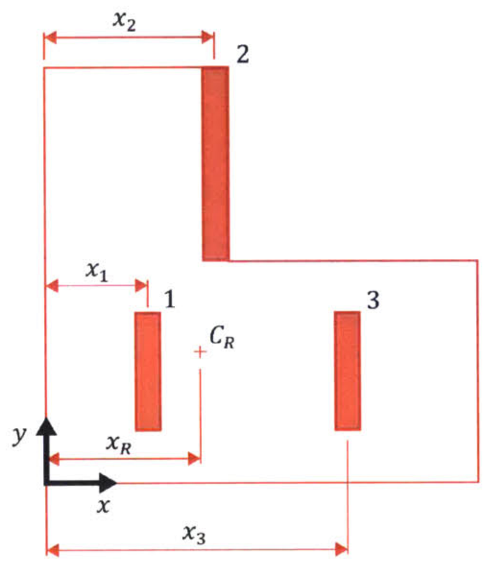


Fig. B.3. Example floor plan for center of rigidity calculation.

$$x_R = \frac{\sum k_{yi} \cdot x_i}{\sum k_{yi}}$$

Or, if the wall stiffness is considered to be shear dominated:

$$x_R = \frac{\sum A_{yi} \cdot x_i}{\sum A_{yi}}$$

Or, if the stiffness is shear dominated and the wall thickness is constant throughout the floor:

$$x_R = \frac{\sum l_{yi} \cdot x_i}{\sum l_{yi}}$$

Find y_R in a similar manner

$$y_R = \frac{\sum k_{xi} \cdot y_i}{\sum k_{xi}}$$

B.4 Comparison of Models of Masonry Shear Strength

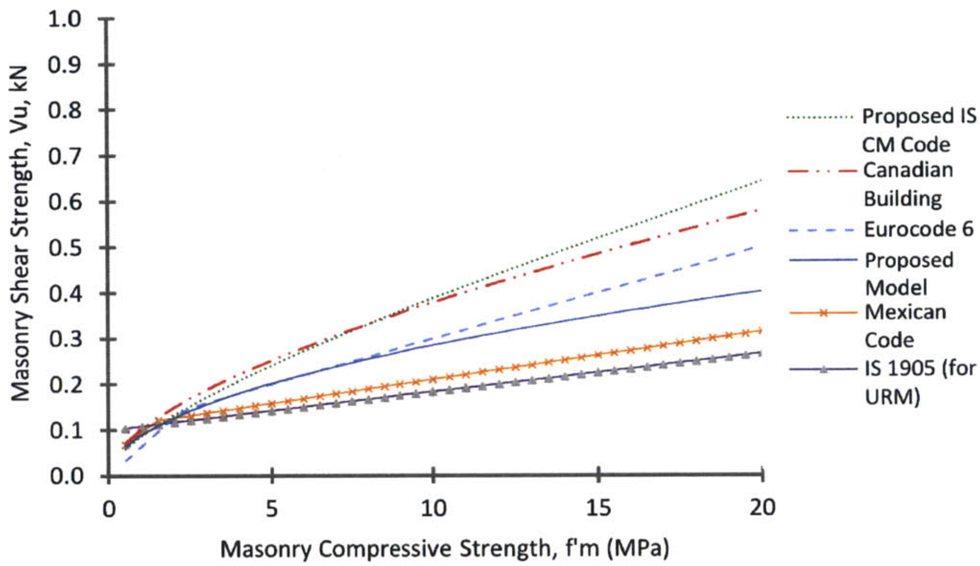


Fig. B.4. Various models for predicting confined masonry shear strength given masonry compressive strength. Note that a compressive stress of $0.05f'_m$ is considered for models that incorporate it.

Appendix C – Experimental Study on Masonry Compressive Strength

C.1 Introduction

Several studies have investigated the behavior of bricks and mortar in masonry under compression and the effects of their properties to one another and to the overall behavior of the masonry. Most studies concluded that as mortar strength increases, prism strength increases, and building codes follow this assumption exclusively (IS 1905, EN 1996, IBC 2012). However, this trend has been found to be more pronounced with high strength units (Hendry 1990; Drysdale et al. 1994), and multiple researchers have concluded that in some cases the trend is not true at all (Sarangapani 1992; Gumaste et al. 2007). The findings of these research studies also suggest that the bond between the mortar and brick units has a more significant effect on masonry compressive strength than mortar strength (Sarangapani et al. 2005). A study by Samarasinghe and Lawrence (1992) found that with dry laid bricks lean mortars achieved higher bond strength than richer cement and lime mortars.

Table 3. Compressive Strength Ranges for Common Bricks

Location	Compressive Strength (MPa) (Sarangapani et al. 2002)	Compressive Strength (MPa) (GSDMA 2005)
Bangalore	3 – 12	-
Mysore	4 – 6.5	-
Kerala	7.6 – 14.2	-
Kanpur	14.6 – 23.7	-
Uttar Pradesh	22.2	10 – 25
Pondicherry	5 – 11	-
Mumbai	4 – 7.5	-
Chennai	2 – 15	-
Delhi and Punjab	7 – 10	7 – 15
Gujarat	3 – 10	3 – 10
Madhya Pradesh	3.5 – 10	3.5 – 5
Maharashtra	-	5
Rajasthan	-	3
West Bengal	-	10 – 25
Andhra Pradesh	-	3 – 7
Assam	-	3.5

Few studies have investigated these effects when low strength bricks are used despite the fact that it is common to use clay bricks with compressive strength lower than 3 MPa in India, especially in rural areas. The variability of brick strengths throughout India is shown in Table 3, derived from results of experimental work presented by Sarangapani et al. (2002) and a guideline for the structural use of masonry by the Indian Institute of Technology Kanpur (IITK) and the Gujarat State Disaster Management Authority (GSDMA) (2005). India is estimated to have over 100,000 brick kilns comprising a sector dominated by small-scale kilns with limited financial, technical, and managerial capacity (Maithel et al. 2012). Unfortunately no change in the quality of bricks used across the country can be expected in the near future. The use of these bricks cannot be ignored simply because they don't adhere to the Indian masonry code IS 1905, which prescribes a minimum brick compressive strength of 3.5 MPa. Hence, since masonry is the most common material for housing construction in the country (Iyer et al. 2012), this is a very important topic and the motivation for this research.

The primary purpose of this experiment was to test the effect of varying mortar qualities on masonry compression strength when low strength bricks are used. A total of 30 red clay brick masonry prisms were tested in India with varying mortar type, curing period, and brick strength. Testing was performed with a 2000 kN capacity compression testing machine at the Government Polytechnic Institute for Girls in Ahmedabad, Gujarat, India. Local materials and labor were used to construct the prisms so that the results would reflect what exists in the field.

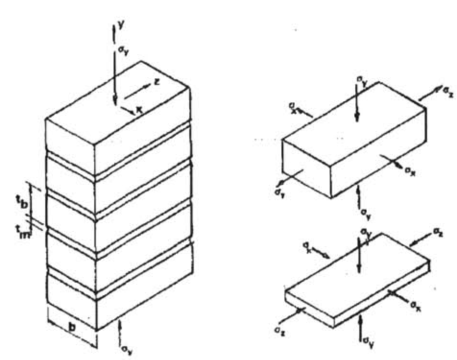


Fig. C.1. Illustration of multi-axial stress theory for bricks and mortar in a compressed prism (McNary and Abrams 1985)

C.2 Current Hypothesis

It is hypothesized that if low quality bricks are used, a weaker, less expensive mortar can provide the same compressive strength in the masonry as a higher quality mortar. This is intended to inform more context conscious designs that can save costs. Common failure theory for masonry in compression considers a multi-axial stress state of bricks and mortar shown in Fig. C.1, where the mortar is in tri-axial compression and the bricks are in axial compression in the direction of the force vector and bi-axial tension in the two orthogonal directions (McNary and Abrams 1985). This failure theory is only valid when the bricks are stronger than the mortar (Hendry 1981). Masonry samples were tested in the current study in which the mortar compressive strength was higher than that of the bricks. In that case the opposite behavior is expected, that is, the bricks are in tri-axial compression and the mortar is in axial compression and bi-axial tension. It is hypothesized that the onset of vertical tensile cracks in low strength bricks will not be delayed by a high quality mortar because under this behavior higher strength mortar will not engage the full strength of the bricks. It should be noted that these behavior theories assume a perfect bond between the bricks and mortar which is not always the case, especially with low-tech construction. More failure theories that match the hypothesis and the experimental results and don't assume a perfect bond are referenced in section C.8 of this report.

The hypothesis was tested in these experiments. More specifically it was examined whether typical low-strength bricks used in Indian non-engineered masonry construction should be used with higher quality mortars to increase wall capacity, or whether doing so would be a waste of funds and actually may even reduce the masonry strength. The results of this study may be used to inform masonry design in a manner that is conscious of both cost and safety. This project was also taken with an objective to provide technical input for a rural housing project being facilitated by PiC for the Commissionerate of Rural Development Gujarat State.

C.3 Results Summary

With brick strengths less than 3 MPa the weaker mortars resulted in no reduction in masonry compressive strength and on the contrary showed higher average strength. The prisms constructed with the marginally higher strength bricks also resulted in higher average masonry strengths. The masonry prisms made using mud mortar without cement showed higher strengths than those with cement mortars on average.

C.4 Materials

C.4.1 Bricks

Two types of red clay bricks were used to construct the prisms. The bricks were obtained from a local construction materials distributor who provided us with commonly used "low quality" and "high quality" bricks per his expertise. Half of the prisms were constructed out of the low quality bricks (L) and half out of the high quality bricks (H). Fig. C.2 shows one of each brick type.

Fig. C.2. A high strength brick (left) and a low strength brick (right).

The brick dimensions were measured for each individual brick tested. The three dimensions were each taken as an average of the four measured lengths at the mid spans of all four faces of the brick as per the American Society for Testing and Materials C67 (ASTM C67) standard procedures.

The strengths of these bricks are presented in Fig. C.7 and Table 4. Although the H bricks were marginally stronger than the L bricks, both were of very poor quality compared to what is prescribed in both Indian and international building codes and standards. When tested, the frogs were filled with hand-compacted sand to provide an even testing surface. The bricks were dry when tested and when laid during construction; this is poor construction practice, however it was done to simulate the reality of field construction.

C.4.2 Cement

A 50 kg bag of 53 Grade Ambuja Cement was purchased for making the cement-based mortars from the same materials dealer who sold the bricks.

C.4.3 Sand

Sand was purchased from the same distributor as the bricks and cement. The sand was stored outside at the store and during testing, hence it was moist due to the monsoon season. Moisture testing of the sand was not conducted due to limited time and resources. The water added to each mortar mix was not controlled and was left to the mason as he deemed appropriate.

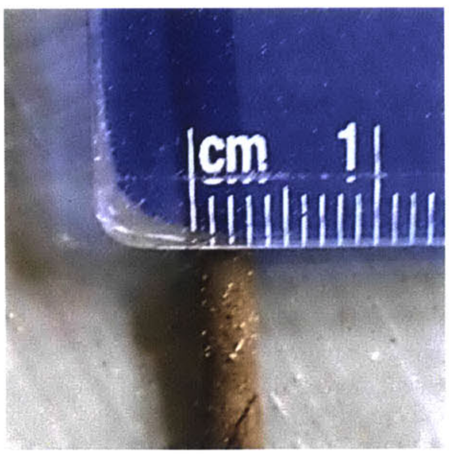


Fig. C.3. Mud mortar thread retained cohesiveness to approximately 3 mm diameter.

C.4.4 Mud

Mud with an appropriate clay content and cohesiveness was sourced from a nearby construction site and was selected by hand and visual inspection by Vivek Rawal, head architect at People in Centre (PiC), and P.B Prajapati, Engineering Department Head at the institute where the testing was performed. The mud was crushed and sieved and then mixed with water multiple times until it was deemed a proper consistency and then was left overnight in an airtight container. The mud was able to be rolled to a 3 mm diameter thread without breaking, see Fig. C.3. The mud mortar prisms were pointed with a 1:6 cement mortar.

The mud was analyzed by KBM Engineering Research Laboratory in Ahmedabad, Gujarat. It was determined through the analysis that the mud was comprised of 7% Medium Sand (2 mm to 0.425 mm), 38% Fine Sand (0.425 mm to 0.075 mm), 40% Silt Sized Particles (0.075 mm to 0.002 mm), and 15% Clay Sized Particles (<0.002 mm). The water content of the mud, after water had been added to it for workability and consistency purposes, was 28.64%.

C.4.5 Water

Water from the restroom tap at the Government Polytechnic for Girls Ahmedabad was used for mortar mixing. It was left to the mason to decide how much water to add to each mix for proper workability.

C.5 Construction

C.5.1 Mortar Mixing

The cement mortar contained only cement, sand, and water; no lime was used. The cement-to-sand ratios were chosen by People in Centre based on prevalent practice of common mixes in low-income, rural housing construction. Cement and sand quantities were measured out for each cement mortar batch by the author of this thesis, dry mixed by the masons, and water was added and wet mixed to proper workability by masons. The cement-to-mortar ratios were measured by volume by the author using graduated cylinders which were purchased at a local supply store. Each batch was mixed in a small bowl typically used by Indian masons, see Fig. C.4, and provided enough mortar to build approximately 1 ½ prisms.

Fig. C.4. Mason mixing mud mortar. Cement mortars were mixed in the same manner.

For comparison with the Indian Standard, the 1:6 cement:sand mortar can be classified as type M2 mortar, and the 1:8 mortar can be classified as type L1 mortar as per IS 1905 Table 1. This table is shown in Fig.

F.1 of Appendix F. The mud mortar may be related to type L2 mortar, the weakest included in the code, however IS 1905 Table 8, given in Fig. F.2 of Appendix F, shows the same prism basic compressive stresses for both types L1 and L2 with minimum strength bricks.

The mortar was mixed in the predefined ratios by volume as measured by the author. The moisture content of the sand was unknown but was moist to the touch. The mason was allowed to add water as he saw fit while he constructed the prisms. The mud mortar was prepared the day before it was used.

C.5.2 Prisms

In total, 30 masonry prisms were constructed. Two local masons built the prisms from August 7 through August 9, 2014 (see Fig. C.5). The first day twelve prisms were built with 1:6 cement:sand mortar, the second day twelve were built using 1:8 mortar, and on the third construction day six prisms were made with mud mortar. Some completed prisms are shown in the curing process in Fig. C.5 (b).

The masons were requested to build the prisms as level as possible to ensure a flat and level testing surface. However, some of the prisms still weren't entirely level. The mortar joints were made 10 mm thick using marked wooden trowels as measuring tools. The ASTM C1552 standard prescribes that the specimens be capped using plaster of Paris, however due to lack of time the prisms were instead capped with a layer of mortar to provide a more uniform, smooth surface than the frogged brick face. Sheets of plywood were also placed between the top and bottom surfaces of the specimens and the testing machine bearing faces as per IS 1905 in order to more evenly distribute the load.

Prisms were built one at a time by the masons and the cement mortar specimens were wrapped, sealed, and labeled immediately upon completion. Subsequently, they were sealed in moisture-tight plastic bags as per ASTM C1314 and remained undisturbed until they were tested. However, the mud mortar prisms were only sealed for three days before they were unwrapped. The mud prisms were pointed with a 1:6 cement:sand mortar, hence they were sealed for three days to allow for the pointing to cure and then unwrapped to allow the mud mortar to dry. A small amount of water was sprinkled on the cement mortar pointing a few days later to help the cement pointing continue to cure. The rest of the prisms were not wetted after being sealed.

(a) (b)

Fig. C.5. Masons constructing prisms (a) and completed prisms in the curing process (b).

C.5.3 Mortar Cubes

Five 70 mm mortar cubes were cast by the author in accordance with ASTM C109 procedures for each mortar type to test the mortar strengths. The cubes were left submerged in a bucket of water to cure until the day of testing. The excess moisture was removed with a rag just before testing.

C.6 Test Setup

C.6.1 Test Specimens and Standards

The masonry prism testing procedures prescribed by the American Society for Testing and Materials standards were followed to as great of an extent as feasible given the resources, labor, and time available for testing of the prisms, bricks, and mortars (ASTM C1314, C67, & C109). Where appropriate or necessary the Indian Standard (IS) 1905 prism testing procedures were followed instead. In total, 30 prisms were tested, ten different sets of prisms were cast with three prisms per set. Parameters which were varied were brick strength, mortar type, and curing period. Two brick strengths were used, three mortar types, and two curing periods. The mud mortar prisms were only tested at one curing period.

Although both brick types had compressive strength less than 3 MPa strength on average, they are classified by their relative strengths (H for high strength and L for low strength). The three mortar types used are designated by their material composition, the first two with cement-to-sand ratios of 1:6 and 1:8, and the third being a mud mortar with no cement, labeled MUD. The two curing periods used were 7 and 28 days. The 7-day prisms were all tested after curing for 7 days, however the 28-day prisms were actually tested after more than 28 days due to technical difficulties with the testing machine. For simplicity the latter prisms will still be labeled as 28 day prisms, for it is assumed that most strength was developed by 28 days.

Each label indicates the parameters that define that specimen and is formatted as such: (Brick Strength)-(Mortar Mix)-(Cure Period), for example *H-1:6-7* labels specimens with high strength bricks, mortar mix of 1:6 cement-to-sand ratio and curing period of 7 days. The numbers (1 to 3) were also added to the end to designate the three prisms within each set, but do not differentiate any specimen properties.

C.6.2 Testing Machine

The prisms were constructed as close as possible to the testing machine and were carefully carried and loaded into the machine by two researchers at a time and using a large container with handles to carry so as to not disturb the specimens prior to testing.

The test machine was an Enkay 2000 kN capacity compression testing machine. The upper and lower bearing surfaces were larger than the surfaces of any of the specimens tested. The lower surface was fixed flat and moved up and down during the test. The upper bearing surface could be cranked down into place before the test and cranked up to release the specimen after testing. The upper surface was on a spherical bearing but had a very minimal angle of rotation, exact specifications were not obtained.

C.7 Test Results

C.7.1 Brick Water Absorption Ratio

Three bricks of each type (H and L) were tested for water absorption. The bricks were dried in an oven at 110°C for 24 hours and then weighed to obtain their dry mass. The bricks were then left submerged in a water bath for 24 hours, removed, wiped with a rag to remove excess moisture, and weighed three minutes after removal from the bath to obtain their wet mass. The scale used provided mass to the nearest hundredth of a kilogram.

The water absorption was determined as a percentage of the dry mass according to IS 3495 by dividing the increase in mass between the dry and wet specimens by the dry mass. The H bricks all had higher water absorption percentages than the L bricks. The average percentage for the H and L bricks were 13.6% and 8.3%, respectively. The results of the water absorption test can be found in Fig. C.6.

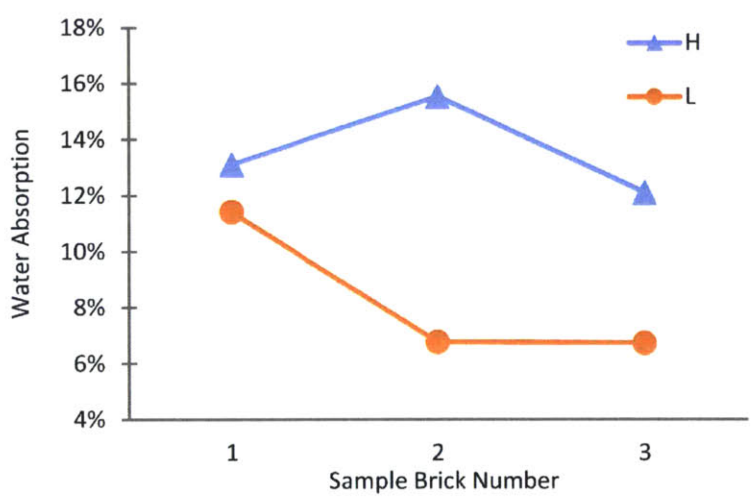


Fig. C.6. Results of brick water absorption test.

C.7.2 Brick Strengths

Eight bricks from each type (H and L) were tested individually in compression. The frogs were filled in with hand compacted sand for a more even testing surface, and plywood sheets were placed between the top and bottom brick surfaces and the bearing faces of the machine for a more evenly distributed load. The testing was performed according to IS 1905.

The higher strength (H) bricks had an average strength of 2.3 MPa with a coefficient of variation of 0.51 and the lower strength (L) bricks had an average strength of 1.9 MPa and a coefficient of variation of 0.70. The H bricks were slightly stronger than the L bricks on average and their results were less variable. The comprehensive results of the brick compression tests are shown in Fig. C.7 and the raw data can be found in Table 4. The low strength and high variability of these bricks in general illustrates the poor quality of bricks used in the region.

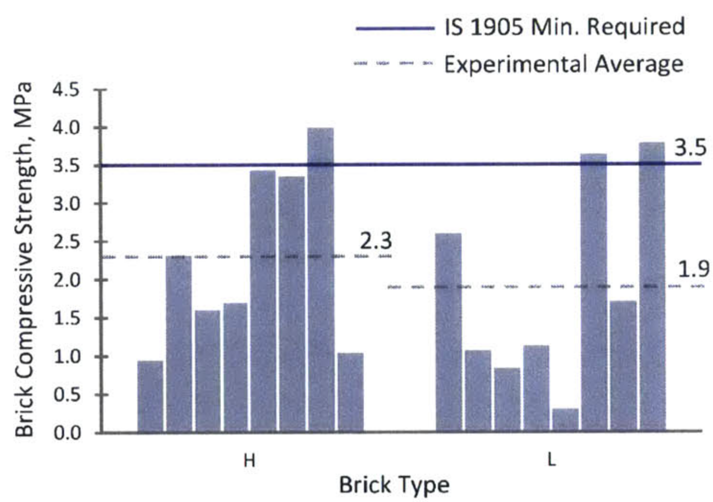


Fig. C.7. Comprehensive strength results for bricks.

C.7.3 Mortar Compressive Strength

Five 70 mm mortar cubes of each mix were cast and tested in compression per ASTM C109 procedures. Since the surfaces up against the faces of the cube mold were the most flat and even, these faces were placed in contact with the bearing platens of the testing machine. Results are shown in Fig. C.8 along with required 28 day strength for these mixes as defined by IS 1905; the full data collected on mortar strength can be found in Table 5. The compressive strengths observed for the cement mortars were significantly higher than the minimum prescribed by IS 1905. The 1:8 cement:sand mortar strength at 28 days was nearly 8 times the minimum prescribed for that mix (L1), indicating that the code is underrating this mix strength. Mud mortar cubes were cast but were too weak to test in the compression machine.

A large increase in mortar strength was observed between the cure periods of 7 and 28 days. However, the difference in 28 day strength observed between the two cement mortar mixes was much less significant, indicating that money could be saved by using a 1:8 cement:sand mortar mix rather than a 1:6 mix.

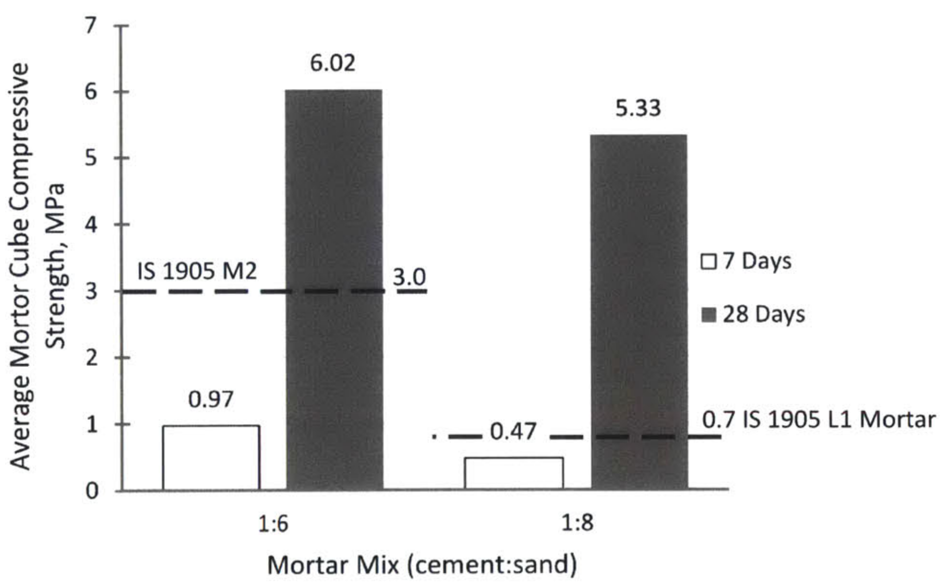


Fig. C.8. Average compressive strengths of mortar cubes.

C.7.4 Prism Compressive Strength

The results show that the average prism strength was inversely related to the quality of the mortar. Prism compression strength results are presented in Fig. C.9 and the full data collected are given in Table 6.

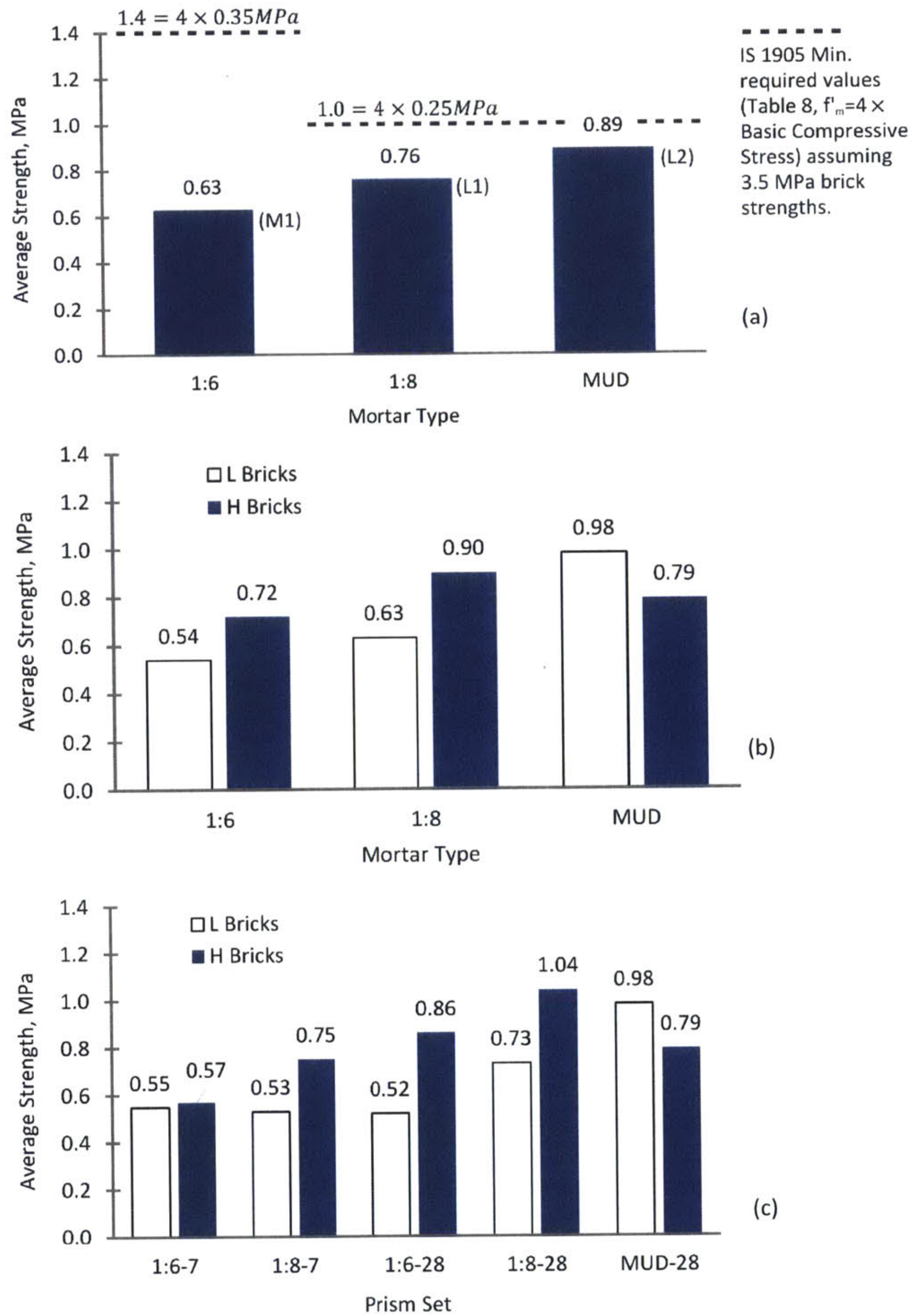


Fig. C.9. Prism strengths by mortar type (a), by mortar type and brick type (b), and averages for all ten sets (c).

During testing, cracking generally initiated in the bricks and propagated vertically through the prism. Cracks usually first appeared in one of the top three bricks of the prism. The vertical cracks were the common failure pattern (see Fig. C.10Fig. C.11). Prior to failure some spalling of bricks on the edges occasionally occurred. The average prism strengths by mortar type are shown in Fig. C.9 (a).

Fig. C.10. The crack initiated at the bottom of the second brick from the top and propagated in both directions.

It may be observed from Fig. C.9 (a) that the experimental results are less than the minimum IS 1905 strength values. For example, prisms with 1:6 mortar had average strengths of 0.63 MPa which is less than 50% of the minimum code requirement. This indicates that masonry compressive strength could be significantly overestimated if the minimum IS 1905 brick strength is assumed in low-tech environments with uncertain brick properties. This observation can be linked with the mortar compressive strength results shown in Fig. C.8. The experimental mortar compressive strength (6.02 MPa) is twice the IS 1905 minimum compressive strength for that mix. This proves the hypothesis that higher mortar strength does not contribute to higher masonry strength when low-strength bricks are used.

Perhaps a more accurate representation of the behavior is revealed when the average strength results are divided by brick type (H or L), as shown in Fig. C.9 (b). This graph clearly shows that for the low strength bricks the prism strength increased as the cement content in the mortar decreased. Interestingly, the highest average strength was reported for the mud mortar prisms. The prisms with high strength bricks showed an increase in strength as the mortar cement-to-sand ratio changed from 1:6 to 1:8, however the average strength then decreased between the 1:8 mortar and the mud mortar prisms. The mud mortar prisms with high strength bricks still showed a higher average strength than prisms with 1:6 mortar.

C.8 Discussion and Conclusions

The prisms with 1:8 cement:sand mortar and mud mortar showed greater compressive strength than those with 1:6 cement:sand mortar for both brick types. For the low strength brick prisms the mud mortar presented the greatest average strength. In general, the strengths of the prisms went up between the 7 day curing period and 28 day curing period, which applies only to cement mortar prisms. Prisms made with the higher strength bricks generally had a higher compressive strength as well. Therefore, it is clear that both the mortar type and brick strength affect the prism strength even when low quality bricks are used. Gumaste et al. (2007) observed similar relationships between mortar strength and prism strength and concluded it was due to two main factors: i) large variation in the brick strengths, which caused initiation of crushing failure in the weakest brick, thus superseding the effect of the mortar strength, and ii) variability in bond strength. Other research has also suggested that flaws in the brick material makeup can initiate cracking at stress levels well below theoretical strength values (Drysdale et al. 1994). Many failures in the study by Gumaste et al. (2007) were due to loss of bond between bricks and mortar, hence the variability of the bond strength proved to be more critical than mortar strength. In the current study failure was usually initiated by vertical cracking in the bricks; this is similar to the failure mechanism observed by Gumaste et al. (2007) in their prism testing.

In a study on compressive strength of masonry prisms with mud mortars by Sarangapani (1992), masonry prisms with bricks of 3.87 MPa compressive strength and mud mortar showed higher strength than prisms with the same bricks and a 1:6 cement:sand mortar. They concluded that mud mortar of 2:1 soil:sand proportion and clay content of approximately 15% can be used as an alternative to conventional 1:6 cement:sand mortar.

The higher masonry compressive strength with leaner mortar may be due to an increase in bond strength. Samarasinghe and Lawrence (1992) found that leaner mortars resulted in higher bond strength than richer mortars and mortars with lime when low strength bricks were used. They concluded that this was due to the fact that leaner mixes have higher water content than richer mortars with the same workability, thus more excess water could be absorbed by the bricks while allowing for adequate mortar hydration, and ultimately a better bond. Although mud mortar was not used in the study by Samarasinghe and Lawrence (1992), a similar explanation could pertain to mud mortar, as unlike cement mortar mud mortar does not achieve its strength through hydration, but rather it achieves strength by drying. This could also offer an explanation for the higher strengths in the mud and 1:8 cement:sand mortar prisms than in the 1:6 prisms obtained in the current study.

The author agrees with the conclusions of Gumaste et al. (2007) and Samarasinghe and Lawrence (1992). It is believed that the high variability in the structural composition of the bricks used leads to failure in the weakest brick, thereby superseding any potential benefit from using higher strength mortars. Moreover, the use of the stiffer 1:6 mortar caused stress concentrations in the bricks which were not observed in prisms with more compatible (leaner) mortars. The author hypothesizes that a better bond was achieved in the prisms with the leaner cement:sand mixes and mud mortar, thereby contributing to increased compressive strength.

In this study, 'high' (H) and 'low' (L) quality bricks from rural areas of Gujarat were used and their strengths were typical for common low-cost bricks of that region. Had these bricks been used at a building site, any resources put towards a higher quality cement mortar than L1 (1:8 cement:sand) would be a waste and in fact would have made the masonry weaker. It is concluded that for masonry where bricks have compressive strength less than 4 MPa, either a mud mortar or a lean 1:8 cement:sand mortar would provide equal or better strength masonry at a lower cost than a 1:6 cement:sand mortar.

(a) (b) (c)

Fig. C.11. Failure damage patterns in tested prisms. Specimen L-1:8-7-1, vertical splitting along the center of the wide face (a). Specimen H-1:6-28-1, vertical splitting along the short face (b). Specimen H-MUD-1, top two bricks spalled as crack propagated downward (c).

Vertical splitting along the center of the wide face, as shown in Fig. C.11 (a), was the most common failure. Vertical splitting along the short face (Fig. C.11 (b)) was also common, but less so than on the wider face. Spalling of bricks also occurred occasionally before failure, see Fig. C.11 (c).

Table 4. Full Brick Strength Results

Brick Type	Maximum Load, kN	Cross Sectional Area, mm^2	Max Stress, MPa	Average Max Stress, MPa	Standard Deviation	Coefficient of Variation
H	25.2	26544	0.95	2.30	1.16	0.51
	52.0	22557	2.31			
	36.2	22440	1.61			
	44.0	25942	1.70			
	89.1	25960	3.43			
	88.0	26307	3.35			
	93.9	23520	3.99			
	26.4	25466	1.04			
L	60.0	23100	2.60	1.90	1.31	0.70
	23.9	22351	1.07			
	18.7	22145	0.84			
	24.4	21614	1.13			
	6.8	22995	0.30			
	91.0	25032	3.64			
	41.0	24003	1.71			
	85.0	22451	3.79			

Table 5. Full Mortar Strength Results

Mortar Mix (cement:sand)	Curing Period, days	Maximum Load, kN	Cross Sectional Area, mm^2	Max Stress, MPa	Average Max Stress, MPa
1:6	7	4.60	4899	0.94	0.97
		4.90	4900	1.00	
	28	30.75	4900	6.28	6.02
		27075	4900	5.66	
		30.01	4900	6.12	
1:8	7	2.30	4970	0.46	0.47
		2.40	4970	0.48	
	28	25.80	4900	5.27	5.33
		24.00	4900	4.90	
		28.50	4900	5.82	
MUD			N/A		

Table 6. Full Prism Strength Results

Brick Type	Cure Period, days	Mortar Mix, (cement:sand)	Maximum Load, kN	Cross Sectional Area, mm^2	Max Stress, MPa	Average Max Stress, MPa
H	7	1:6	14.1	23175	0.61	
			15.7	23956	0.66	0.57
			10.2	23278	0.44	
		1:8	17.4	22880	0.76	
			19.0	21900	0.87	0.75
			14.1	22890	0.62	
	28	1:6	16.0	25850	0.62	
			25.0	21412	1.17	0.86
			21.0	26544	0.79	
		1:8	25.0	23088	1.08	
			20.0	22000	0.91	1.04
			25.0	22145	1.13	
		MUD	17.0	21930	0.78	
			20.0	23052	0.87	0.79
			17.0	23296	0.73	
L	7	1:6	15.0	21513	0.7	0.55
			9.1	23436	0.39	
		1:8	7.7	22032	0.35	
			12.7	22236	0.57	0.53
			15.1	22896	0.66	
	28	1:6	15.0	22880	0.66	
			10.0	21930	0.46	0.52
			10.0	22464	0.45	
		1:8	10.0	25300	0.4	
			20.0	21917	0.91	0.73
			20.0	22575	0.89	
		MUD	20.0	24200	0.83	
			24.0	22660	1.06	0.98
			24.0	22575	1.06	

Appendix D – A Study on Brick Strengths in the State of Gujarat, India

D.1 Introduction

In January 2015 60 bricks were collected from 7 different locations across the State of Gujarat to investigate average brick strengths throughout the state (see Fig. D.1). Due to lack of time the author was unable to test the bricks while in India so a local engineering laboratory was retained to do the testing. KBM Engineering Research Laboratory (the same lab that characterized the mud mortar used in the first study) performed the testing. The author collected all samples and designed the experiments to be conducted by KBM. Five to ten bricks each from 9 sources (two brick sources came from Virpur and Bhachau, two red clay brick sources in Virpur and one red clay brick source and one fly ash brick source in Bhachau) were tested for compressive strength and water absorption. The results showed a large dispersion of brick strengths throughout the state. The obtained strength values are in line with those reported by Sarangapani et al. (2002) and GSDMA (2005) (see Table 3).

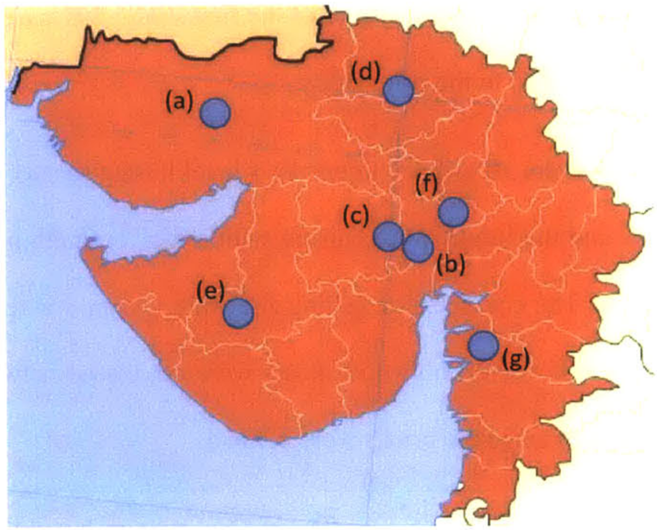


Fig. D.1. Kiln locations in Gujarat where brick specimens were collected:

(a) Bhachau
(b) Kelia Vasna
(c) Chaloda
(d) Gada (near Patan)
(e) Virpur
(f) Bareja
(g) Surat

D.2 Testing Procedures

A combination of ASTM C67 and IS 3495 brick testing procedures were followed in this study. Since the testing was performed by a professional lab, the author was able to specify testing procedures that followed the codes more precisely than the procedures of the first study; this is a possible reason for the higher brick strengths observed compared to the prior study.

The brick bed surfaces were all grinded to ensure a level testing surface per IS 3495. The frogs were filled with cement mortar and the bricks were capped with a high strength plaster of Paris per the ASTM C67 standard requirements. The compression testing and water absorption testing were conducted on full brick specimens per IS 3495; however, plywood sheets were not placed between the specimens and the testing machine bearing surfaces since the bricks were capped.

D.3 Results and Discussion

The results of this study are comparable to those presented in Table 3. The average brick strengths range from 4.5 to 12.3 MPa, see Fig. D.2. The lowest brick strength tested was 3.0 MPa from the Surat kiln (Avg. 4.5 MPa) and the highest was 14.7 MPa from the Gada kiln (Avg. 9.0 MPa). The strengths and water absorption ratios observed are widely variable across Gujarat. This finding highlights the importance of realistic specification of material properties for a given construction project and confirms the fact that in developing countries one cannot assume that brick strengths comply with the code. It should be noted that bricks within each set, or from a specific kiln, showed relatively consistent water absorption properties. The standard deviation for the average brick strengths and water absorption for each kiln are plotted on Fig. D.2 and Fig. D.3, respectively. Full brick compressive strength data collected are provided in Table 7.

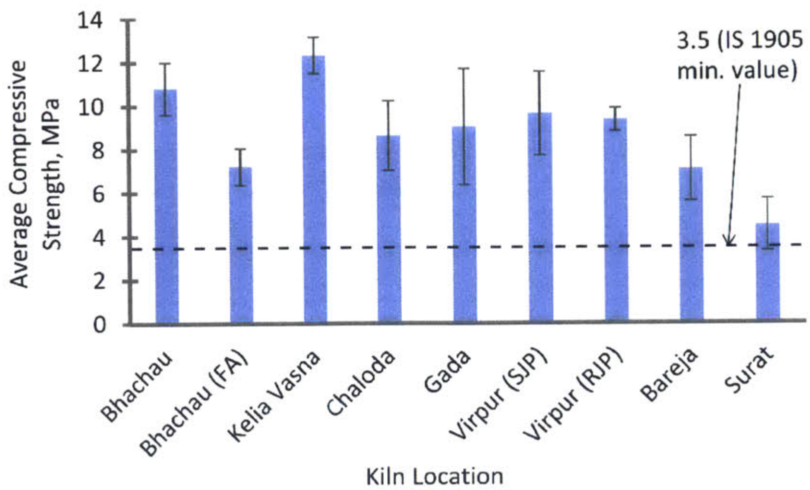


Fig. D.2. Average brick strengths for Gujarat observed for each kiln including standard deviation.

Average water absorption ratios for each kiln are shown in Fig. D.3. Water absorption values ranged from approximately 12 to 25% and were consistent within sets. Compressive strength of all the bricks tested is plotted against water absorption in Fig. D.4, showing a negative relationship between brick strength and water absorption. This is in line with the findings of other studies on Indian bricks (Kaushik et al. 2007).

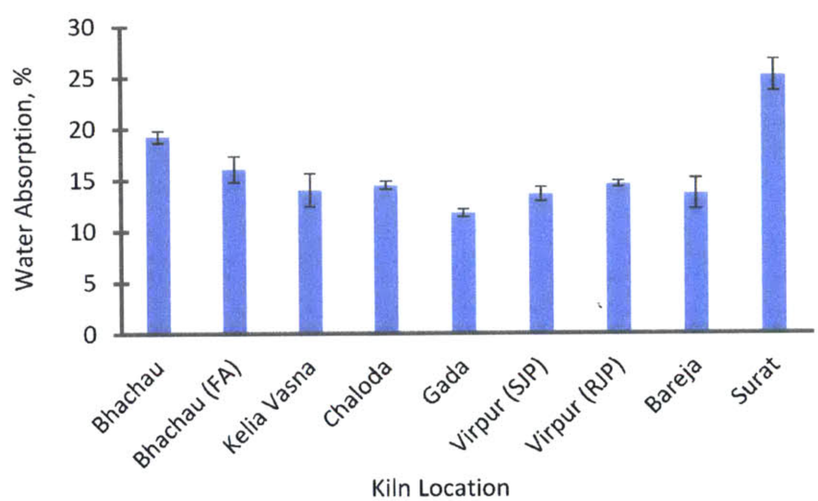


Fig. D.3. Average water absorption ratio for bricks from each kiln including standard deviation.

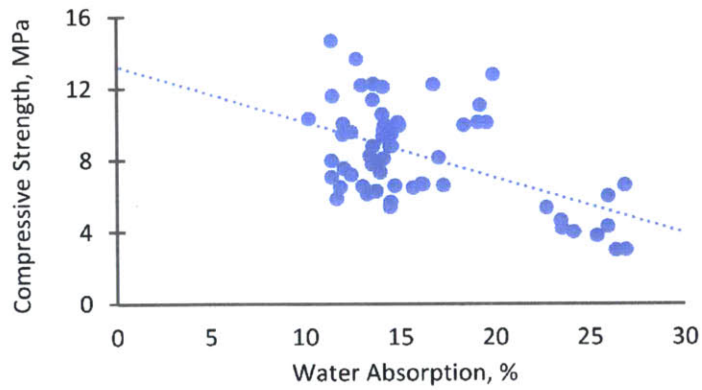


Fig. D.4. Full results of brick strength versus water absorption with a trend line showing an inverse relationship.

Table 7. Full Brick Strength Results

Brick Source	Maximum Load, kN	Cross Sectional Area, mm^2	Max Stress, MPa	Average Max Stress, MPa	Standard Deviation	Coefficient of Variation
Bhachau, Kutch	237.5	21463.47	11.06	10.80	1.19	0.11
	222.7	22039.65	10.10			
	218.5	21597.85	10.12			
	215.5	21660.32	9.95			
	282.9	22136.58	12.78			
Bhachau (fly ash)	189.4	23260.72	8.14	7.20	0.84	0.12
	149.9	23125.65	6.48			
	188.6	23288.00	8.10			
	156.7	23734.76	6.60			
	155.8	23298.25	6.69			
Kelia Vasna, Ta.: Dholka	290.2	23750.42	12.22	12.30	0.82	0.07
	273.4	22609.83	12.09			
	279.4	22944.84	12.18			
	310.4	22772.49	13.63			
	258.9	22793.19	11.36			
Chaloda, Ta.: Dholka	235.3	23261.78	10.12	8.62	1.59	0.18
	177.6	24199.28	7.34			
	212.8	23487.03	9.06			
	235.9	23643.56	9.98			
	152.7	23190.58	6.58			
Gada, Near Patan	176.8	23470.20	7.53	9.02	2.66	0.29
	278.6	24070.00	11.57			
	220.8	23061.36	9.57			
	195.8	24569.20	7.97			
	339.9	23192.10	14.66			
	157.8	24307.50	6.49			
	224.3	23775.05	9.43			
	140.7	24089.51	5.84			
	248.9	24820.68	10.03			
	171.9	24352.74	7.06			

Table 8. Full Brick Strength Results (Cont'd)

Virpur, Jamnagar (SJP)	159.0	22164.44	7.17	9.62	1.91	0.20
	197.9	22533.12	8.78			
	240.2	22774.10	10.55			
	207.4	22164.53	9.36			
	277.9	22673.81	12.26			
Virpur, Jamnagar (RJP)	215.8	22317.60	9.67	9.33	0.53	0.06
	196.5	22345.00	8.79			
	221.8	22320.63	9.94			
	212.4	22446.00	9.46			
	200.5	22862.72	8.77			
Bareja, Ahmedabad	176.3	21372.01	8.25	7.06	1.47	0.21
	171.7	22356.86	7.68			
	171.9	22111.39	7.77			
	142.9	21514.06	6.64			
	124.9	22060.67	5.66			
	142.8	22735.75	6.28			
	132.6	21723.36	6.10			
	126.8	23481.34	5.40			
	231.7	22474.60	10.31			
	146.8	22434.56	6.54			
Surat	71.9	24076.95	2.99	4.49	1.19	0.27
	73.8	24580.96	3.00			
	123.9	23146.63	5.35			
	113.2	24431.55	4.63			
	101.9	24183.25	4.21			
	162.5	24619.99	6.60			
	95.7	25349.45	3.78			
	98.9	24763.05	3.99			
	107.8	25024.38	4.31			
	147.9	24695.60	5.99			

Appendix E – Notation List

A = Building footprint area;

A_h = Seismic coefficient;

A_i = Cross sectional area in plan of wall i;

$A.R.$ = Building plan aspect ratio;

C_M = Center of mass of the structure;

C_R = Center of rigidity of the structure;

D = Base dimension length along direction of applied force;

E_m = Masonry modulus of elasticity;

F_i = Shape factor for wall i;

FAT = Amplification factor for torsion;

G = Masonry shear modulus;

H = Story height;

H_i = Height of wall i;

I = Building importance factor;

L = Length of longer building dimension;

LF = Load Factor to be applied in accordance with the Ultimate Limit Sate Design Method;

N = Number of structural walls in the floor oriented in the direction of analysis;

R = Response reduction factor;

S_a/g = Average response acceleration coefficient;

T = Natural period of the structure;

T_M = Torsional moment;

V_R = Shear capacity of building story;

V_b = Design base shear determined from the SMSA;

V_{di} = Direct seismic shear force in wall i;

V_{t_i} = Torsional shear force in wall i;

V_{tot_i} = Total seismic shear force in wall i;

W = Seismic weight of the structure;

W = Length of shorter building dimension;

Z = Seismic zone factor;

b = Building plan dimension orthogonal to the direction of the applied load;

c_i = Distance of wall i to the center of rigidity, C_R, of the structure;

d = Preliminary required wall density ratio determined from the SMSA;

d_d = Final design required wall density ratio;

e = Normalized eccentricity of the structure;

e_d = Design eccentricity of the structure;

e_s = Static eccentricity of the structure;

f_{AR} = Amplification factor for building plan aspect ratio;

f'_m = Masonry compressive strength;

k_i = Lateral stiffness of wall i;

l_d = Final design required length of structural walls;

l_i = Length of wall i;

l_r = Preliminary required length of structural walls determined from the SMSA;

n = Number of stories in the structure;

t = Wall thickness;

v_m = Masonry shear resistance;

w = Average seismic weight of the structure per unit floor area;

x_i = x-coordinate of wall i;

x_R = x-coordinate of the center of rigidity;

y_i = y-coordinate of wall i;

y_R = y-coordinate of the center of rigidity;

β = Factor for accidental eccentricity;

Δ = Interstory displacement in each direction of the building plan taken at the first story;

α, δ = Dynamic amplification factors;

ζ_i = Dimensionality factor of wall i;

ρ = Normalized radius of gyration;

ϕ = Material resistance factor to be applied in accordance with the Ultimate Limit State Design Method;

Appendix F – Supplementary Materials from the Indian Masonry Code

IS : 1905 - 1987

TABLE 1 MIX PROPORTION AND STRENGTH OF MORTARS FOR MASONRY

(*Clause* 3.2.1)

Sl No.	Grade of Mortar	Mix Proportions (By Loose Volume)					Minimum Compressive Strength at 28 Days in N/mm²
		Cement	Lime	Lime Pozzolana Mixture	Pozzolana	Sand	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	H1	1	¼C or B	0	0	3	10
2(a)	H2	1	¼C or B	0	0	4	7.5
2(b)		1	½C or B	0	0	4½	6.0
3(a)	M1	1		0	0	5	5·0
3(b)		1	1 C or B	0	0	6	3.0
3(c)		0	0	1 (LP-40)	0	1½	3.0
4(a)	M2	1	0	0	0	6	3·0
4(b)		1	2 B	0	0	9	2·0
4(c)		0	1 A	0	0	2	2·0
4(d)		0	1 B	0	1	1	2·0
4(e)		0	1 C or B	0	2	0	2·0
4(f)		0	0	1 (LP-40)	0	1½	2·0
5(a)	M3	1	0	0	0	7	1.5
5(b)		1	3 B	0	0	12	1.5
5(c)		0	1 A	0	0	3	1.5
5(d)		0	1 B	0	2	1	1.5
5(e)		0	1 C or B	0	3	0	1.5
5(f)		0	0	1 (LP-40)	0	2	1.5
6(a)	L1	1	0	0	0	8	0·7
6(b)		0	1 B	0	1	2	0·7
6(c)		0	1 C or B	0	2	1	0·7
6(d)		0	0	1 (LP-40)	0	2¼	0·7
6(e)		0	0	1 (LP-20)	0	1½	0·7
7(a)	L2	0	1 B	0	0	3	0·5
7(b)		0	1 C or B	0	1	2	0·5
7(c)		0	0	1 (LP-7)	0	1½	0·5

Note 1 — Sand for making mortar should be well graded. In case sand is not well graded, its proportion shall be reduced in order to achieve the minimum specified strength.

Note 2 — For mixes in Sl No. 1 and 2, use of lime is not essential from consideration of strength as it does not result in increase in strength. However, its use is highly recommended since it improves workability.

Note 3 — For mixes in Sl No. 3(a), 4(a), 5(a) and 6(a), either lime C or B to the extent of 1/4 part of cement (by volume) or some plasticizer should be added for improving workability.

Note 4 — For mixes in Sl No. 4(b) and 5(b), lime and sand should first be ground in mortar mill and then cement added to coarse stuff.

Note 5 — It is essential that mixes in Sl No. 4(c), 4(d), 4(e), 5(d), 5(e), 6(b), 6(c), 7(a) and 7(b) are prepared by grinding in a mortar mill.

Note 6 — Mix in Sl No. 2(b) has been classified to be of same grade as that of Sl No. 2(a), mixes in Sl No. 3(b) and 3(c) same as that in Sl No. 3(a) and mixes in Sl No. 4(b) to 4(f) same as that in Sl No. 4(a), even though their compressive strength is less. This is from consideration of strength of masonry using different mix proportions.

Note 7 — A, B and C denote eminently hydraulic lime, semi-hydraulic lime and fat lime respectively as specified in relevant Indian Standards.

Fig. F.1. IS 1905 Table 1 containing mortar mixes and respective minimum mortar compressive strength.

IS : 1905 - 1987

TABLE 8 BASIC COMPRESSIVE STRESSES FOR MASONRY (AFTER 28 DAYS)

(*Clause* 5.4.1)

SL No.	MORTAR TYPE (REF TABLE 1)	BASIC COMPRESSIVE STRESSES IN N/mm² CORRESPONDING TO MASONRY UNITS OF WHICH HEIGHT TO WIDTH RATIO DOES NOT EXCEED 0·75 AND CRUSHING STRENGTH IN N/mm² IS NOT LESS THAN											
		3·5	5·0	7·5	10	12·5	15	17·5	20	25	30	35	40
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	H1	8·35	0·50	0·75	1·00	1·16	1·31	1·45	1·59	1·91	2·21	2·5	3·05
2	H2	8·35	0·50	0·74	0·96	1·09	1·19	1·30	1·41	1·62	1·85	2·1	2·5
3	M1	8·35	0·50	0·74	0·96	1·06	1·13	1·20	1·27	1·47	1·69	1·9	2·2
4	M2	0·35	0·44	0·59	0·81	0·94	1·03	1·10	1·17	1·34	1·51	1·65	1·9
5	M3	0·25	0.41	0·56	0·75	0·87	0·95	1·02	1·10	1·25	1·41	1·55	1.78
6	L1	0·25	0·36	0·53	0·67	0·76	0·83	0·90	0·97	1·11	1·26	1·4	1·06
7	L2	0·25	0·31	0·42	0·53	0·58	0·61	0·65	0·69	0·73	0·78	0·85	0·95

NOTE 1 — The table is valid for slenderness ratio up to 6 and loading with zero eccentricity.

NOTE 2 — The values given fof basic compressive stress are applicable only when the masonry is properly cured.

NOTE 3 — Linear interpolation is permissible for units having crushing strengths between those given in the table.

NOTE 4 — The permissible stress for random rubble masonry may be taken as 75 percent of the corresponding stress for coarsed walling of similar materials.

NOTE 5 — The strength of ashlar masonry (natural stone masonry of massive type with thin joints) is closely related to intrinsic strength of the stone and allowable working stress in excess of those given in the table may be allowed for such masonry at the discretion of the designer.

Fig. F.2. IS 1905 Table 8 with minimum values for masonry basic compressive stress for a given mortar mix and brick strength.