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Y. Korany
S. Humphrey

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Preface

The Masonry Chair in Masonry Systems is a three million dollars endowed research chair funded by the Masonry Contractors Association of Alberta–North and the Government of Alberta's Access to the Future Fund. The inaugural chair holder is Dr. Y. Korany. An integral part of the mandate of the MCAA Masonry Chair is to develop and teach academic and professional courses in the design and analysis of masonry systems and masonry building envelopes. The ongoing and planned research projects under the Chair's mandate aim to remove the unjustifiable limitations imposed on masonry construction due to inadequate information, reveal the untapped capabilities of contemporary masonry, and drive innovation in masonry design.

The northern chapter of MCAA was formed on July 23, 1965 and represents both union and non-union contractors. Its mandate is to ensure quality masonry construction, to maintain strong apprenticeship programs, and to promote contemporary masonry through close collaboration with the University of Alberta.

The 2nd Masonry Mini Symposium is part of the graduate level course: *Behaviour and Design of Masonry Structures* that was taught in the spring of 2010. Course participants were asked to work in groups to write technical papers on topics relevant to masonry systems. They presented their findings to a panel of professional, engineers, architects, contractors, and building official on the evening of March 30th 2010. This year's theme was underutilized masonry systems. In this report, the reader will find a compilation of the edited manuscripts of all papers presented during this event. The report is available to the public in PDF format through the online depository of the University of Alberta Library.

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Design and Construction of Interlocking Mortarless Block Masonry

J. Edwards, M. Gayed, M. Pyra and T. Rodriguez

Department of Civil and Environmental Engineering, University of Alberta

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ABSTRACT

In developing countries masonry is still the most prevalent housing material, while a renewed interest in the developed world has helped transform this ancient system into an innovative engineering material with a variety of structural applications. Due to the time-critical nature of the modern construction industry, there is a need to improve upon traditional masonry construction methods, which are labour and time intensive. The pursuit of this has led to the development of several nonconventional methods of masonry construction, including a variety of mortarless systems.

This paper will explore the design and construction aspects of mortarless masonry construction in comparison to traditional methods. An overview is presented for the evolution of new techniques and advancements in the masonry field, in addition to the different mortarless applications for various countries. Furthermore, a detailed classification of mortarless masonry systems is offered, along with a review of the behaviour of some of these systems. A cost comparison between traditional construction and a mortarless system indicates a reduction in labour costs and construction time, which validates this new trend as a cost-effective alternative. Design examples for both traditional and mortarless construction are included for walls subject to in-plane and out-of-plane loading in order to validate the use of mortarless systems from a designer's perspective. The results of these exercises indicate a promising future for the use of mortarless interlocking masonry systems in construction.

Keywords: masonry, mortarless, dry-stack, interlocking, block

INTRODUCTION

Over the last few decades, masonry construction has gone through major changes which have led to improved constructability and performance. These improvements are due to an increase in block size and innovative methods such as surface-bond masonry, fibre-reinforced polymer (FRP) wrapping, partially grouted masonry, and mortarless masonry using interlocking blocks. The primary reason for these changes is the increasing need for masonry construction to compete with other structural building materials such as steel and reinforced concrete.

Masonry performs simultaneous functions of carrying load and enclosing space, while possessing strong properties for fire resistance, thermal and sound insulation and protection against environmental exposure. As a result, masonry is a cost-effective and low-energy alternative when designed appropriately (Ramamurthy and Nabiar, 2004). However, its main shortcoming is that its construction is slow and labour intensive. Furthermore, conventional masonry construction, especially for smaller units, leads to a large number of mortar joints. In order to limit the stresses induced in these joints during construction, the rate at which the height of a wall increases in somewhat restricted.

The introduction of interlocking mortarless masonry has led to a large increase in field productivity and efficiency, as well as a reduction in the requirements for highly specialized labour crews. This paper will present a literature review of interlocking mortarless masonry construction and its inherent advantages in building construction.

DEVELOPMENT OF MORTARLESS MASONRY SYSTEMS

Advancements in Traditional Construction

As one of the oldest construction methods in human history, masonry has become a major competitor in modern construction. The renewed interest in its practice around the world is largely due to its transformation from a brittle and fragile material to one that can successfully endure dynamic loading from earthquake and wind forces. This construction technique underwent its first major modification since Roman times with the introduction of reinforced concrete slabs as floor and roof systems in building construction, allowing the formation of rigid structures. It became apparent that further improvements had to be made when the aftermath of the Long Beach earthquake (California, USA) in 1933 revealed widespread damage to unreinforced masonry structures (Casabonne, 2000). With the incorporation of steel reinforcement into walls, tensile resistance increased significantly. These two advances, coupled with accessibility to quality-controlled masonry units with increased compressive strength, enabled the construction of taller structures and a reduction in wall thickness, which substantially increased the efficiency of masonry construction. Over the last 40 years in North America, and more recently in Latin American countries, renewed interest in masonry has made this material the object of extensive research to understand its behaviour, define its mechanical properties and improve its safety and seismic performance. During this process, masonry expanded beyond its aesthetic applications into a viable structural system that exhibits a greater degree of ductility.

Evolution of Mortarless Construction

In present-day industry, acceleration of the construction process has become an increasingly important factor; traditional masonry methods are labour intensive and time consuming due to the presence of a large number of mortar joints. Early attempts were made to increase the size of masonry units (block instead of brick) and thereby reduce the number of mortar joints and increase the number of layers constructed in a day. Size became limited by the weight of each unit that was reasonable for construction without jeopardizing quality. The need for further acceleration led to the elimination of bedding mortar and the development of non-conventional methods of masonry construction utilizing interlocking blocks.

The market demand for a modern and efficient material has fuelled researchers over the last 4 decades in developing a number of mortarless masonry systems. In conventional masonry construction, blocks are stacked with mortar bedding for both bed and head joints. Mortarless construction, as the name indicates, eliminates the need for mortar by utilizing interlocking geometries or other non-geometric mechanisms such as grouting or surface bonding (Anand and Ramamurthy, 1999). Blocks that interlock to provide levelling and alignment reduce the need for skilled workers and reduce construction time.

Units specifically designed for mortarless construction are available in many different configurations. The latest and most sophisticated designs incorporate face shell alignment features that make units easier and faster to stack plumb and level. Other units are fabricated with a combination of keys, tabs or slots along both horizontal and vertical faces so that they may interlock easily when placed. Most of the commercially available systems vary in geometry, material, and dimensional characteristics. This diversity of blocks caters to variable applications through a range of features including simplicity of shape, interlocking in horizontally or vertically, production through conventional methods, fire resistance, thermal and sound insulation, and environmental protection (Anand and Ramamurthy, 1999).

Classification of Mortarless Systems

Currently in North America, several types of mortarless blocks have been proposed including those that are of interlocking geometry. Most of these are hollow; however, solid interlocking blocks have also been developed as an improvement over the traditional adobe bricks that were prevalent in some African countries during the 20th century. Individual blocks are generally the same size as a typical 200 x 200 x 400 mm concrete masonry block, but each system is unique (Murray 2007). Table 1 outlines several of the most notable mortarless systems that have been proposed. Depending on the application, one system may provide more advantages than another. For example, lintel beams spanning over a door or window opening will require a block suitable for horizontal reinforcement. As it can be seen from table 1, only some systems provide both horizontal and vertical reinforcement capabilities.

Table 1 - Types of Proprietary Interlocking Block Mortarless Masonry Systems Available (Y.M.D. Adedeji, 2008 and Anad, 2000)

Name of System (Country,Year)	Block Type (Material)	Interlocking Mechanism	Reinforcement Type
Haener™ (USA, 1975)	Hollow (concrete)	Nibs in bed joint. Tongue and groove in head joint.	Vertical and horizontal
Whelan (USA, 1985)	Hollow (concrete)	Dovetail arrangement in head joint and projected face shell	Vertical
Sparlock™ (Canada, 1986)	Hollow (concrete)	Geometric interlocking and stacking pattern	Vertical
Mecano (Peru, 1988)	Hollow (concrete)	No geometric interlocking	Vertical and horizontal
Sparfil (Canada, 1989)	Hollow (light-weight concrete)	No geometric interlocking – intent to be used as surface bonded masonry	No reinforcement
Modified H-Block (USA, 1992)	Hollow (concrete)	Grooved face shells on both the head and bed joints	Vertical and horizontal
Whelan-Hatzinikolas-Drexel [WHD] (USA, 1992)	Hollow (concrete)	Rounded dovetail lug on head joint	Vertical
Azar™ (Canada, 1997)	Hollow (concrete)	Three mechanisms: key on top of web fits into recess of block above. Two levels of bearing surface along each face shell at the bed joint. Interlocking at head joint by shiplap geometry.	Vertical and horizontal
Silblock™(India, 1999)	Solid/hollow (concrete)	Geometric interlocking and stacking pattern	Vertical and horizontal

The classification of mortarless masonry can be summarized in Figure 1 as follows:

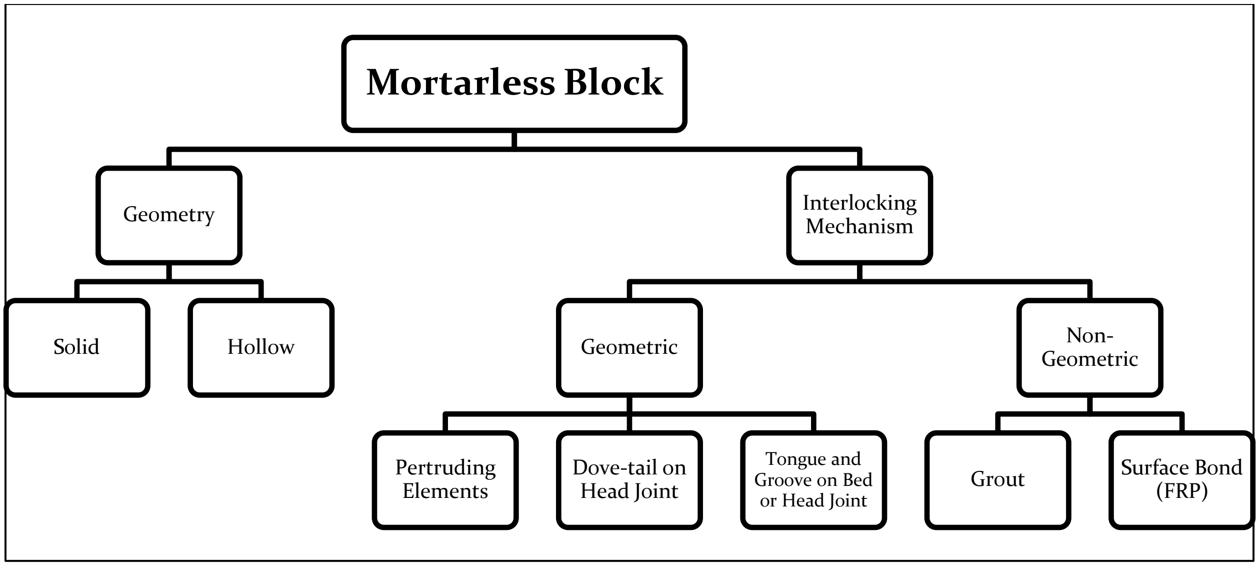


Figure 1 - Classification of Block Mortarless Masonry

REGIONAL APPLICATIONS OF MORTARLESS MASONRY

North American Industry

In North America, some of the most common applications of reinforced masonry include apartments, schools, hotels and hospitals. Though there are a number of mortarless systems available, a few of them have gained a stronger foothold in North American industry than others. At the end of the 20th century, three of the previously mentioned systems were most prevalent (Haener, Azar, and Sparlock), in addition to a number of insulated systems (VanderWerf, 1999). As mortarless construction becomes more prevalent, these systems have begun to expand beyond their traditional applications, but each has achieved its initial success in a different sector of the market.

Haener Block

The Haener block has been on the market longer than any other mortarless system. The webs of the units have raised lugs, which are offset in such a way that they interlock with the courses above and below. One practiced labourer can stack more than 100 blocks in an hour as compared with 40 in an hour from the combined efforts of a labourer and a mason in conventional masonry construction (i.e. 20 per worker per hour). It is the responsibility of the installer to address the issues that arise from variable block heights. It is common practice to plumb the wall through the use of shims such as brick ties or incorporating a layer of mortar every four courses. The Haener system is grouted and reinforced as required, comparable to traditional masonry (VanderWerf, 1999).

Conventional manufacturing processes are utilized with the exception of a contoured shoe to form the lugs and some adjustment to the cubing process. The original system consists of stretcher, corner and half blocks. A newer two-block system uses a combined stretcher-corner block. This was further developed to incorporate space in each block for insulation in the Two-Block Insulating System. The Haener system has been sold for individual consumer use, primarily in Los Angeles, in addition commercial applications throughout North America. There is no single predominant use for the system.

Azar Block

Relatively new to the industry, Azar blocks interlock along both the bed and head joints through a mechanism that resembles a tongue and groove system. Walls are completed using a combination of stretcher and corner blocks and an unskilled labourer can stack up to 100 blocks per hour. The assembled wall is adjusted to plumb using temporary bracing as required and fully grouted to lock the units in place. As a fully grouted system, it requires more grout than conventional masonry in approximately 95% of applications (VanderWerf, 1999).

Even in foundation applications, no damp-proof paring or drainage layers are required. Water is controlled through the use of a water-repellent admixture, water-resistant grout formula, vibration during grouting and scores moulded into the block face that facilitate drainage to the base. Initially, the Azar system was used primarily for house foundations, but it has also been implemented for firewall construction.

Sparlock Block

The Sparlock system circumvents issues caused by variability in block height through the use of a stack bond. Two wythes interlock horizontally with a half-height course at the base of one face (inside or outside) in order to stagger the bed joints. It follows that the three types of blocks are stretcher, half stretcher and corner. The final result is a wall with increased vertical bending strength and improved fire and sound ratings. Walls constructed using the Sparlock system provide a 2-4 hour fire rating and achieve a sound transmission class (STC) of 52 (VanderWerf, 1999). As a point of reference, an interior single-stud wall has an STC of 30-34 (Quiet Solution, 2010).

Labourers familiar with the Sparlock system can stack blocks at a rate equivalent to 40 conventional masonry units per hour. For low or moderate load walls, no further structural work is required and in the event that the surface will be walled over, as with a firewall, no finishing work is required either. The system does not incorporate any special provision for reinforcement, but vertical reinforcement can be installed with a high-slump grout. In manufacturing the units, special moulds are used with standard equipment and some additional consideration is required in cubing. The primary focus for the Sparlock system has been the firewall market.

Integrated Masonry Systems International (IMSI)

In recent history, mortarless construction methods like the IMSI system have integrated insulation directly into the masonry units. The IMSI blocks have two rows of cavities as shown in Figure 2. The outer cavity is for insulation and electrical wires, while the inner cavity is used for grout and additional insulation and wires as needed. Shimming is performed during assembly, in addition to the installation of wiring, electrical boxes, insulation and rebar. Due to the expediency of mortarless construction, this can all be performed at the same rate as a conventional masonry wall without any electrical work or insulation. Grout is used selectively as in traditional methods and once the wall is complete, surface-bonding cement is trowelled onto both faces. The surface bond seals the wall and increases the flexural strength. The IMSI system is used in a range of applications, but its insulative properties and electrical functionality make it especially well-suited for above-ground residential structures (VanderWerf, 1999).

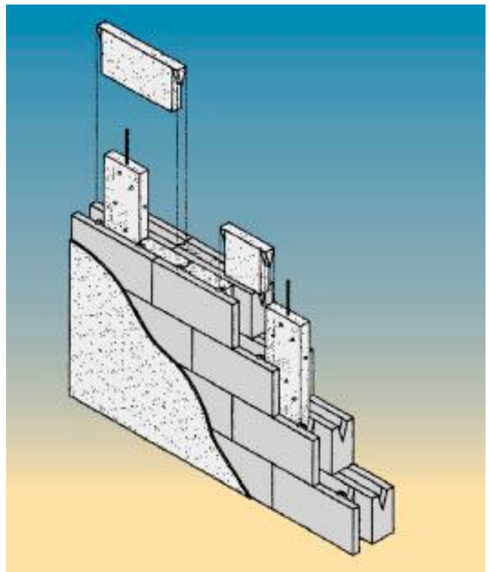


Figure 2 - Cutaway of an IMSI Wall (VanderWerf, 1999)

Durisol and Faswall Systems

The Durisol and Faswall insulated systems are also prevalent in the North American market. The units in these systems are formed from a mixture of Portland cement and mineralized wood fibre. This combination is almost as durable as conventional concrete, though it is easily cut and fastened with common wood saws and nails. The material is lightweight with a typical 200 x 300 x 900 mm Durisol block weighing approximately 14 kg. The ease with which these units can be handled and manipulated is conducive to rapid construction with occasional shimming. Since the compressive strength of the wood fibre concrete is relatively low, walls are fully grouted and the units have partial webs in order to produce a bond beam at each course (see Figure 3). Reinforcement is installed as necessary and mineral-wood inserts can be utilized to achieve a maximum R value of 21 with a 300 mm unit. The primary distinction between the two systems is that the Faswall system is sized to match conventional full-dimension blocks (200 x 300 x 400 mm). Utilized for above-grade walls, these systems are primarily used in construction of houses and medium-rise apartments, though there has been some commercial use as well (VanderWerf, 1999).

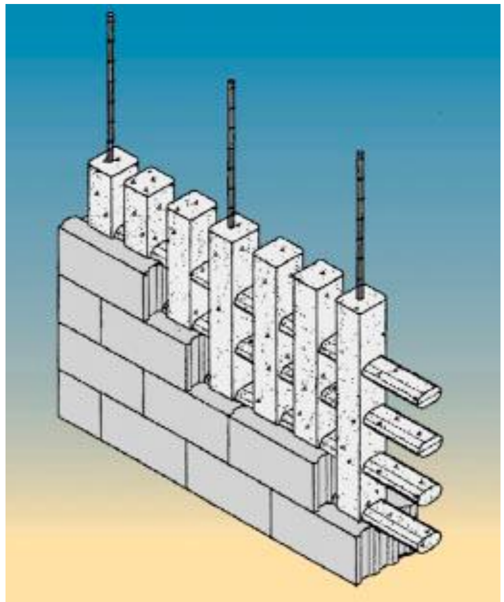


Figure 3 – Cutaway of a Faswall wall (Adapted from VanderWerf, 1999)

International Applications

As a basic requirement for survival, the need for shelter is something that permeates the lives of billions of people worldwide. In developing countries, the cost of material and labour is continually on the increase and often beyond the reach of people in need. As a consequence, there is a widespread deficit in housing construction. This shortage has stimulated efforts to develop alternative building methods that can reduce costs and minimize labour. The implementation of mortarless masonry systems has been very successful in these countries because the overall project costs are reduced and the raw material is often readily available. Nigeria, Peru, and Malaysia are examples of just a few of the many countries that have benefited from mortarless construction.

The building industry in Nigeria is producing various kinds of building systems adapted to local materials, environmental conditions, and city developments. One such system is the adaptation of interlocking masonry for housing construction based on reduced costs and short time implementation. These advantages have created a high acceptability for the mortarless product. The Nigerian Building and Road Research Institute (NBRRI) has developed interlocking block fabrication technology intended to produce blocks with a standard size of 225 x 225 x 112 mm (Y.M.D. Adedeji, 2008). These blocks are manufactured in such a way that they do not require cutting during setting operations, thereby reducing time and eliminating associated waste. This innovative technique has been widely accepted by the building industry in Nigeria and is implemented in housing projects as a cost-effective alternative to conventional masonry construction.

In Latin America, masonry construction is used primarily for the construction of apartment buildings for medium and low income families. The typical height limit is five stories, which coincides with the height restriction for buildings without an elevator in most regional construction codes. Peru, for example, is where the Mecano block system was developed, in which the blocks are stacked in successive layers without any interlocking. All of the voids provided in the blocks are then grouted following the placement of the horizontal and vertical reinforcement. Due to the nature of the system, it is essential that the blocks are smooth and manufactured with consistent dimensions (Casabonne, 2000).

Malaysia has developed another interlocking hollow block system, based on plastic LEGO™ building blocks. Individual units and full-scale walls have been tested at the University Putra Malaysia for different types of loading (Thanoon et al., 2004). Testing on the system indicates that the compressive strength of the individual blocks satisfies Malaysian code requirements. The new system can be used for two-storey building with a satisfactory margin of safety. Similarly, at the Teknologi Malaysia University, a new interlocking block system named Putra Block has been developed. The block shape is simple, resulting in efficient production and easy assembly during construction. Interlocking both horizontally and vertically, this is a self-aligned construction system to ensure accurate and expedient construction. Although the system is still under study, it has the potential to become one of the most widespread construction methods employed in the country due to its efficiency and cost effectiveness.

Case Study #1 – Sparlock Empowerment Program

Sparlock is a Canadian-based mortarless masonry unit developed in the early 1980's. Over the years of research and development, the product has become well established in the North American market. More recently, Sparlock blocks have added to their marketing strategies and become recognized in the housing market of developing nations. The Sparlock Empowerment Program is based on the idea of being able to set up a third world community with the tools and knowledge to be self-sufficient. Sparlock is able to create a mobile production site out of a standard sized shipping container (see Figure 4 below). Steel moulds provide a reusable tool to make the mortarless blocks from local materials.

Figure 4 - Sparlock mobile production container (Sparlock, 2010)

Instructors educate local residents so they are capable of assembling their own residences without any special trades. No mortar joints are required; therefore, the amount of skilled labour that is necessary is drastically reduced. The walls for an 85 m^2 (900 ft^2) house with finished window and door frames may be easily erected by a small work crew in one day (Sparlock, 2010).

CONSTRUCTION

Advantages of Mortarless Masonry

For conventional masonry, the requirement for mortar at the head and bed joints of each block necessitates a larger amount of time and skill in construction. Furthermore, time required for mortar bed curing restricts the height of a wall that may be built in a given day due to the self-weight of the constructed masonry. Therefore, it can be appreciated that the elimination of bedding mortar will accelerate construction, thereby reducing cost and variation due to workmanship. In addition, some other advantages of mortarless construction are:

- Blocks can be laid much easier with less technical skill.
- Decreased risk of moisture damages and shrinkage.
- More stability during construction (especially for interlocking blocks) compared with conventional masonry because the blocks do not "swim" in mortar.
- Walls can be loaded immediately after construction.
- Masonry can be assembled in any weather conditions (i.e. winter season).

In a study done by Ramamurthy and Nabiar in 2004, it was found that the rate of construction can be accelerated significantly. The laboratory experiment involved the construction of 1400 mm square panels from both conventional block masonry and Whelen interlocking block masonry. Test results showed that the completion time for the conventional masonry was 3.5 hours with a crew of one person, whereas the time spent for the Whelan blocks was 1.25 hours. This indicates that the production rate for the mortarless system is approximately 4.1 m^2 per hour, which is 2.5 times that of the traditional system. Manufacturers of Haener blocks state that their system allows blocks to be laid up to five times faster than conventional block masonry, creating labour savings up to 80%. With the Azar system, the Azar group reports that unskilled workers can stack 100 blocks each per hour, compared with approximately 30 to 40 blocks per hour for a crew of three masons and one helper for conventional masonry.

Limitations of Mortarless Masonry

In several studies, it was found that a set mortar was required at the base of a wall in order to facilitate wall levelling and aligning for the first course. Thereafter, the blocks themselves must be manufactured with increasingly high standards and tight tolerance such that the wall will maintain plumb during construction. Current physical tolerances for mortarless concrete units are limited to ± 1.6 mm (1/16 in) which precludes the need for mortaring, grinding of face shell surfaces, or shimming to even out courses during construction.

It is rather difficult to enumerate the main disadvantages of mortarless block since each system has inherent advantages and disadvantages associated with its design; however, a general disadvantage list for the majority of units is as follows:

- Relatively high initial settlement.
- Challenges in controlling the height of the units to achieve accurate running bond.
- Difficulty in keeping grooves from breaking during transportation and construction.

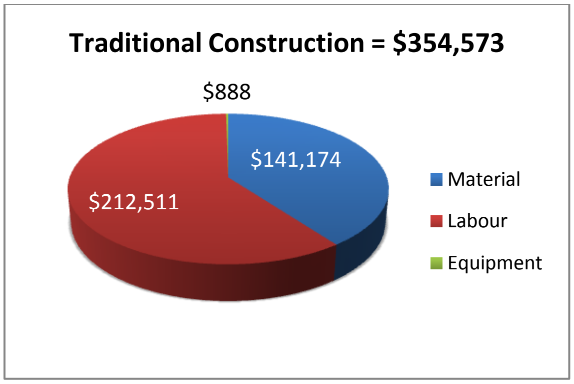
- In running bond pattern, plain interlocking units without grouting have little resistance to vertical bending.
- In stack bond pattern, plain interlocking units without grouting have little resistance to horizontal bending.

In addition to these inherent limitations, regional design and building codes place restrictions on the use of mortarless masonry. Once again, these vary from system to system. For example, Azar construction in Canada has the following constraints (among others) placed upon its use in order to ensure compliance with the National Building Code (NBCC) (CCMC 12873-R, 2001):

- Permitted above and below grade for buildings up to three storeys and an area of 600 m^2.
- Basement walls can be no higher than 2.5 m.
- Wall construction must be fully grouted.
- Exterior above-grade walls and interior load bearing walls are limited to a height no greater than 20 times the wall thickness.
- Non-load bearing internal partitions are limited to a horizontal or vertical span no greater than 5.2 m.

Cost Comparison

As previously mentioned the primary advantages of mortarless masonry are the expediency of construction and reduced labour costs. In order to validate these claims, a cost analysis is contained herein for the Azar Mortarless Building System in comparison with traditional masonry construction. A typical commercial structure in Edmonton, Alberta is used as the basis for comparison, with dimensions of 15 m x 40 m x 4 m (W x L x H) for the building envelope. The wall is reinforced with 20M vertical bars at 600 mm spacing. While the traditional construction only requires grout every third core to correspond with the reinforcement, the Azar system must be fully grouted as indicated during the previous discussion of the system. Three masons and one labourer are required to stack and mortar the conventional wall, while one mason leads a crew of three labourers for the Azar system. The rates of production for these two crews are approximately 30 blocks per hour and 300 blocks per hour, respectively (Azar Mortarless Building Systems, 2010). The remaining cost and production values used in the analysis are based on information from RS Means 2009 and the detailed breakdown is included in Appendix I. Figure 5 illustrates the total costs associated with each type of construction.



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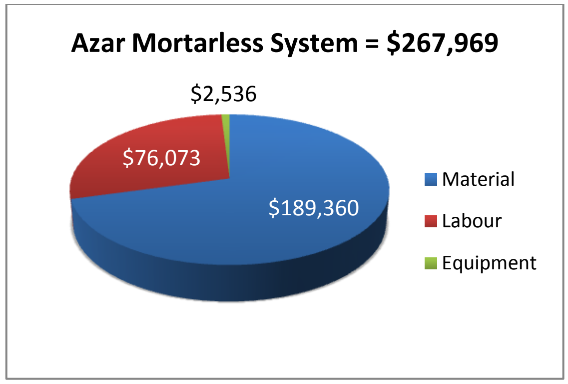

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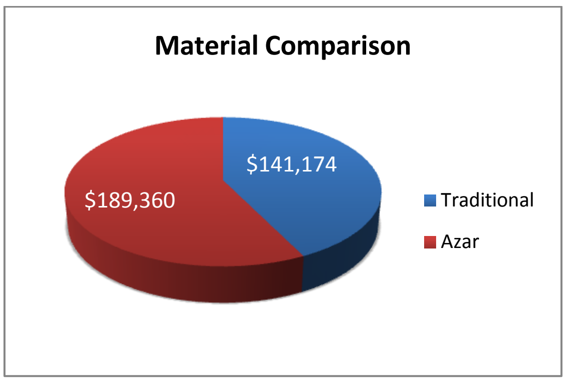

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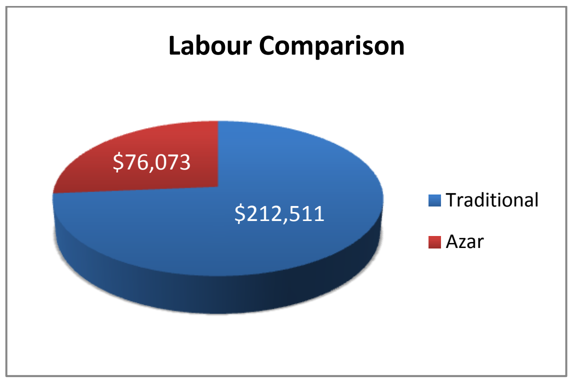

(d)

Figure 5 – Construction costs associated with traditional and Azar mortarless construction

This comparison illustrates significant overall savings in choosing the Azar system over conventional masonry (24%). The material costs for the mortarless system actually exceed those of traditional construction due to the higher cost for the units themselves and the larger quantity of grout. However, the 64% reduction in labour costs more than compensates for this. Additionally, the duration of the construction is reduced from 1377 hours (34.4 weeks) for conventional masonry to 505 hours (12.6 weeks) for the Azar Block wall. Depending on the nature of the project, this offers potential for additional monetary gains from a structure that is operational at an earlier date.

Case Study #2 – FlexLock House in Magnolia, Texas

The FlexLock Wall System, developed by Cercorp Initiatives, was chosen as the primary building material for a project in Magnolia, Texas in order to study the benefits of mortarless blocks. The scope of work for the project consisted of a 275 m^2 (2,958 ft^2) house with construction starting in June 2006 (Cercorp Initiatives, 2010). The project consisted of all of the standard elements included within a typical masonry structure: lintels, jambs, sills, and bond beams. This provided a comprehensive comparison with conventional masonry units.

Figure 6 - Completed building envelope for FlexLock house in Magnolia, TX (Cercorp Initiatives, 2010)

The FlexLock Wall System is comprised of interlocking, mortarless masonry units. The blocks are similar to other products with interlocking mechanisms at both the bed and head joints, but the manufacturer recommends that the walls be post-tensioned. This increases the structural capacity of the wall, eliminates the need for grout, and reduces the amount of initial settlement. Figure 7 depicts a wall of the house being post-tensioned.

Figure 7 - Post-tensioning of wall anchors for FlexLock house in Magnolia, TX (Cercorp Initiatives, 2010).

At the time of construction, the Magnolia region had an average installed price of roughly $75 per m² for conventional masonry units. After breaking down the cost of the FlexLock System project, the price came to $57.24 per m² (Cercorp Initiatives, 2010). This is a reduction of 24% in installed cost. The labour crews were inexperienced with the mortarless product, yet were still capable of stacking blocks at an average rate of 164 blocks an hour. When compared to conventional masonry, this is an increase in productivity of over 100%.

CODES AND STANDARDS

Canada and the United States

Serving as the model for the masonry design standards in many other countries, the United States code covers analysis and design for strength and serviceability, as well as construction of masonry structures. It also includes a chapter on prestressed masonry. The American industry has made a move towards limit states design, but working (allowable) stress design is still widely practiced. In Canada, the code for the design and construction of masonry buildings is regulated by the Canadian Standard Association (CSA) and the current edition is CSA S304.1-04. Furthermore, the National Building Code regulates the design of all types of structures and is in accordance with limit states practices. However, mortarless masonry construction is not specifically addressed in these codes and standards due to the relatively short history of the available systems.

Europe

Similar to North American practice, the Eurocode and accompanying product standards define standard testing procedures and calculation methods. The determination of mechanical characteristics of masonry through adequate testing methods is essential in verifying the load bearing capacity and stability of masonry structures (Tomaževič, 2009).

Latin America

Development of codes and standards for masonry construction was not a priority in Latin American countries until masonry started to be re-evaluated roughly 25 years ago. Most masonry codes, as is the case with reinforced concrete and other structural materials, are adaptations of existing codes in the United States, primarily the Uniform Building Code (UBC).

Peru's current code is based on working stresses and empirical design recommendations. Admissible stresses in this code are based on results of an extensive test program performed on materials to determine basic design parameters. Similarly, the Colombian Earthquake Resistant Buildings Code also indicates that masonry design provisions are based on working stresses and empirical design recommendations (Casabonne, 2000).

The design provisions in Chile's code for reinforced masonry and confined masonry design provisions are based on allowable stress design, but some limit states concepts have also been utilized. Design of reinforced concrete block masonry in this code is similar to that in the UBC, but a special variation on the shear strength of hollow clay brick masonry has been introduced to prevent cracking under moderate earthquakes.

According to the Complementary Technical Norms for Design and Construction, Mexican masonry regulations were updated after the Mexico City earthquake in 1985. Unlike most Latin American countries, current Mexican design provisions are based on Limit States Design like the Canadian Code (Casabonne, 2000).

As demonstrated above, masonry structures in every country are governed by unique regional codes and standards. Because mortarless masonry construction is a comparatively recent development, there are few, if any, design provisions specific to its use. In practice, results from empirical testing are used to validate the application of existing design standards.

DESIGN ASPECTS

Mortarless and interlocking blocks differ from conventional blocks in that the units are assembled together using geometric features incorporated in the unit without the aid of mortar. Thus, the stress distribution and ultimately the strength for an assembly of blocks will differ significantly from that of mortar bedded masonry. As an example, the efficiency factor (masonry strength/brick strength) for "brick" mortarless masonry is reported to be around 0.9, which is significantly higher than that of conventional masonry construction, roughly 0.3 to 0.4 (Ramamurthy, 2004). In addition, uninspected and unskilled workmanship for conventional masonry construction may result in incompletely filled bed joints, deeply furrowed bed joints, thicker mortar layers and deviations from plumb, which further reduces the strength of standard masonry (Syrmakezis, 2001).

Engineering Properties

General building code recommendations can serve as reference documents in the design of mortarless masonry systems; however, the current National Building Code of Canada (2005) does not directly address mortarless masonry construction. Test data collected from a pool of literature primarily published by Drysdale (1999 and 2005) will be briefly presented in this paper; further information can be retrieved from referenced papers.

Compressive Strength

Units used for mortarless masonry construction are usually manufactured from the same concrete mixes used for conventional masonry units. The compressive strength of typical concrete masonry units (CMU's) varies between 13.8 MPa and 27.6 MPa, although, manufacturers provide different grades of blocks depending on the demands of a specific project.

For interlocking hollow block mortarless walls, the characteristic strength, also known as the masonry compressive strength, of a wall must be evaluated to proceed with design. While there is no current standard or code provision available to provide a correlation between the compressive strength of mortarless wall assemblies with the compressive strength of block units, the current Canadian masonry standard, CSA S304.1-04, outlines two methods for determining the required compressive strength of masonry, f'_m. First, the compressive parameter may be determined by testing prisms in accordance with Annex D of S304.1-04. Alternatively, the f'_m values can be conservatively based on the tabulated compressive strength values given in Table 4 of the standard. These tabulated values have been shown to be punitively conservative and much less than the strength values measured from prism testing for mortar bedded masonry. Furthermore, these tabulated values are based on triaxial stress that exists in the block-mortar interface, which is non-existent in mortarless masonry. This triaxial state can be explained by the fact that mortar tends to be more flexible than the block unit due to a lower modulus of elasticity. In effect, tensile stresses normal to the direction of applied compressive load will develop, thereby decreasing the overall compressive capacity.

In a study by Jaafar and others in Malaysia, an experimental program was conducted to obtain the strength correlation between individual blocks, prisms, and basic wall panels for load bearing interlocking mortarless hollow block masonry (Jaafar et al., 2005). It was found that the correlation between block strength and masonry prism compressive strength is a ratio of 0.47. In another comparative study by Drysdale and Hamid for eccentrically loaded mortarless masonry, it was concluded that the ratio of compressive strength at a given eccentricity to the axial compressive strength (i.e. f'_{me}/f'_m) is, on average, higher than unity. This rather high ratio is explained in terms of the physical interlocking of the blocks, which seem to increase the extreme fibre failure stress for an eccentricity of $t/_6$ by about 75-80%, compared with a value of 43% for standard solid masonry. This study illustrates high axial capacity of mortarless masonry in comparison with conventional construction. However, a recent study at the University of Witwatersrand in Johannesburg, South Africa was performed on a proprietary mortarless masonry system under the name of Hydraform. Axial compression tests were conducted on walls which were 3 m wide and 2.5 m tall. A control wall was built using traditional concrete block techniques and tested for comparison. Results of these tests showed a 65% increase in axial strength when mortar was used in the bed joints. The difference in strength was attributed to a difference in failure mode. The mortarless masonry walls tended to fail in shear and splitting of the head joints. Alternatively, when mortar was used in the bed joints, the mortar resisted the shear, which slightly increased the axial capacity of those walls.

Grout

Grouted mortarless masonry has a plethora of applications, especially when out of plane forces are predominant. Grout strength has an influence on development length for reinforcing bars and, to a lesser extent, on the strength of the masonry itself (Drysdale, 2005). The experiments concluding these results are from prism tests of mortar-bedded masonry. To an extent, applying these results to mortarless masonry would be incorrect without conducting further experiments.

From laboratory testing, grout is known to have higher compressive strength in the field than in tests conducted in the laboratory on non-absorbent cylinders. This is attributed to the fact that the masonry blocks absorb water from the grout during construction. Without additional water introduced by mortar on the bed and head joints, blocks used in mortarless construction are able to absorb a larger quantity of water from the grout. This lowers the water to cement ratio which leads to much higher grout strength. Common grout strength observed in the field is around 16 MPa; this value will be used in shear design checks.

It can be concluded from the above literature review that there is currently no consensus on mortarless masonry design parameters that may be used for general design. The characteristics of mortarless masonry under compressive loads and the overall behaviour of the system are still not fully understood. The design equations presented herein will be in line with those established by the Azar block proprietary system, which is manufactured and distributed in Canada. Experimental studies on this specific proprietary mortarless masonry system have been conducted at McMaster University and published on the manufacturer's website.

A Discussion of the Resistance Factor, φ_m

In limit states design, the intent of a resistance factor is to take into account the statistical variability of masonry resistance to ensure a uniform level of safety. Since its inception, much limit states design literature has been published to aid in the calibration of resistance factors. A brittle material such as masonry, reported to impose failure without much warning, shall have a target safety index in the range of 4.0 to 4.5. Further investigation and analytical work is needed to evaluate an appropriate resistance factor for mortarless masonry. However, for lack of a more representative value, the resistance factor given in S304.1-04 for mortar bedded masonry construction shall be used, i.e. $\varphi_m = 0.6$. The rationale behind this assumption is that mortarless masonry behaviour shall be similar to that of mortar bedded construction, especially for reinforced grouted cells. Some experimental work undertaken for mortarless systems has shown that its measured compressive strength is actually higher than those of conventional masonry (Jaafar et al. 2006; Thanoon et al. 2007).

Flexure

As in conventional masonry design, wall strength and stability are greatly enhanced with grouting as it provides additional effective area to resist forces applied parallel and perpendicular to the bed joints. In addition, grouting provides a medium for confining vertical and horizontal steel reinforcement for resisting tensile stresses. The strength of grouted mortarless walls may also be enhanced through pre-stressing, post-tensioning, or external fibre-reinforced surface coatings such as surface-bonding fibre reinforced polymer. This will briefly be discussed later in this paper.

Unreinforced grouted construction will resist flexural stresses due to out-of-plane bending in the grout cores. However, the tensile stress capacity of the wall will be minimal without steel reinforcement so unreinforced construction may not be appropriate depending on the design application at hand. In-plane flexure behaviour will not be discussed in this report. The reader is encouraged to use design methods in the Canadian standard S304.1 for such cases.

The following design approach and associated design equations are representative of those expressed by the National Concrete Masonry Association and those published in Azar Block design manuals. They only serve as a guide to the structural design of mortarless masonry construction. Further testing and data for a specific proprietary masonry system may be required to give a more accurate and representative design.

Fully Grouted - Unreinforced Construction (Out-of-Plane)

The lack of mortar to bond two stacked block units will preclude the development of tensile stresses between the face shells. Flexural strength is then fully realized from the grouted core and compressive face shell bearing strength of consecutive masonry units. For out-of-plane bending of fully grouted walls, the estimated flexural strength based on linear elastic behaviour is:

$$\frac{M_f}{S_n} - \frac{P_f}{A_e} \leq \varphi_m f_t \quad [1]$$

The net geometric section modulus, S_n, should take into consideration the reduction in wall thickness at the bed joints due to the lack of mortar on the tension bed-face. If the "virtual eccentricity" of the wall is less than that of the Kern eccentricity, $e_k = \frac{t}{6}$, the strength of the wall will be governed by the masonry's compressive strength. Thus, flexural strength based on masonry compressive stress should also be checked such that:

$$\frac{P_f}{P_{r,max}} + \frac{M_{ftot}}{\varphi_m S_n f'_m} \leq 1 \quad [2]$$

Slenderness effects shall also be checked in accordance with clause 10.7.4.3 of S304.1-04.

Grouted – Reinforced Construction (Out-of-Plane)

In conventional mortared masonry design, masonry tensile strength is neglected; therefore we can expect mortarless masonry to behave in essentially the same way provided that the units subjected to compressive stress are in good contact. Thus, the design principles outlined in the Canadian Masonry Standard can be utilized. The compressive strength, f'_m, can be taken from the manufacturer's recommended design guides; if this value is not provided, it can be taken as 47% of block strength for lack of testing (Jaafar et al., 2006). The Azar mortarless system has been tested and the established prism to block strength is taken as 0.56; this corresponds to a characteristic prism compressive strength of 16.2 MPa for a 30 MPa block.

Shear

For hollow mortar-bedded masonry construction, shear strength for out-of-plane bending is usually not a concern since flexural strength typically governs the design. This generalization may also be extended to mortarless construction due to instability issues posed by eccentric loads out of plane. However, more complex behaviour will be developed for in-plane shearing loads for mortarless masonry due to interlocking geometry and the non-existence of mortar.

In grouted construction, whether reinforced or not, the grout cores are expected to interlace units and provide additional resistance to shear forces beyond that provided by frictional forces developed along the horizontal joints. Therefore, it is recommended that mortarless masonry construction be grouted for any shear resisting application.

Out-of-Plane Shear Resistance

As mentioned above, it is seldom that shear resistance is the governing case in design for out of plane load, but a quick check is necessary to ensure that strength requirements are satisfied. Failure for this type of loading may be due to either shear failure of the block or sliding failure between bed joints. For both unreinforced and reinforced masonry, the following expression given by CSA S304.1 (Clause 10.10.2) can be used for shear strength of the block:

$$V_r = \varphi_m 0.4\sqrt{f'_m}\, A_{ev} \qquad [3]$$

In the above equation, the parameter A_{ev} is the effective grout area, which is taken as the effective width of the block minus two face-shells. Another failure mode may occur due to sliding between two consecutive courses, especially in cases of low axial compression. Again, the shear bond strength component is neglected for mortarless joints and the shear resistance will be due to frictional forces developed between masonry interfaces. Thus, the sliding shear resistance is taken as:

$$V_r = \varphi_m \mu P_d + V_{rs} \qquad [4]$$

In this expression, the friction coefficient, μ, is taken as 1.0 for a masonry-to-masonry sliding plane and a load factor of 0.9 is applied to the beneficial effects of the dead load, P_d. Finally, the parameter V_{rs} is the shear strength of vertical steel reinforcement (if provided), which can be taken as $\varphi_s 0.577 A_s f_y$.

In-Plane Shear Resistance

In 1998, Marzahn conducted a study on the shear strength of mortarless walls subjected to "static" in-plane shear forces, i.e. shear walls. Due to the various masonry wall geometries and loading conditions, it was found that different failure behaviour can occur as depicted in Figure 8 below.

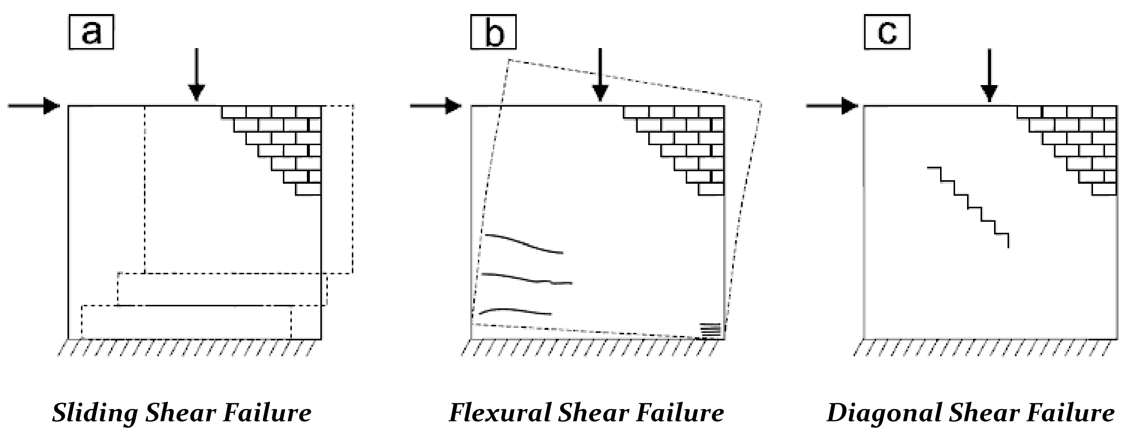

Figure 8 - Types of masonry wall failures induced by in-plane lateral loads (Marzahn, 1998)

As indicated above, shear walls subjected to horizontal in-plane loads may fail in one of three ways: a) sliding shear failure, b) flexural shear failure, or c) diagonal shear failure. These modes will be discussed in the following sections.

Sliding Shear Failure

This type of failure is attributed to "squat" walls where the height of the wall is equal to or less than the length. Treatment of out-of-plane shear strength can be extended to this case where the wall is pending failure at course interfaces; thus, equation number 4 above can be modified to include the uplift effect on frictional forces. With a larger lateral force and height of the wall, we expect less frictional forces to develop at the masonry-to-masonry interface due to uplifting tensile forces beneath the neutral axis. Thus, it would be unconservative to take the entire dead load of the wall into consideration, and a reduction factor is applied as follows

$$V_r = \varphi_m \mu (0.6 P_d) + V_{rs} \qquad [5]$$

The 0.6 factor is representative of the ratio of "effective" contact area between frictional surfaces. All other parameters have been defined previously.

Flexural Shear Failure

In this case the wall behaves as a vertical cantilever under lateral bending. Failure of a wall will be due to either cracking of the masonry in tension or crushing at the wall toe. Therefore, the governing case will be flexural stresses; however, a shear check should be performed according to equation 5 to ensure capacity in shear is not exceeded.

Diagonal Shear Failure

Diagonal shear failure is characterized by a critical combination of principal tensile and compressive stresses, where the tensile face stress exceeds that of the masonry strength. This type of failure is highly influenced by the in-plane moment-to-shear ratio. Higher moments will increase tensile stresses and cause cracking, which in effect will reduce the shear strength of the wall. The design equations found in the Canadian standard S304.1-04 can be utilized in lieu of other methods which have yet to be experimentally validated. These design equations found in clauses 7.10.1 and 10.10.1 for unreinforced and reinforced walls, respectively, must be used with the grout compressive strength, f'_d, instead of the given masonry compressive strength, f'_m. This is due to the fact that mortarless masonry does not provide any bond strength along the bed and head joints; thus, only grout strength contribution can be considered and, to a lesser effect, the bonding stresses between the grout and masonry block. Some mortarless masonry manufacturers will provide a masonry shear strength value for design; the designer is encouraged to use these values instead if a specific proprietary system.

Innovative Methods for Increasing Capacity

Prestressing

Similar to conventional masonry, mortarless masonry demonstrates good resistance against compression but relatively small resistances to shear and bending as a result of the poor bond between bed joints of individual blocks. However, tests have shown that added axial compressive stresses applied through tension tendons will increase flexural and shear capacity (Marzahn, 1999). These tendons may be bonded to grout, or unbounded post-tensioned tendons depending on the design preference. Flexural strength will increase due to the higher induced axial force, which causes a stabilizing effect. In addition, shear strength will be increased as additional frictional forces are sustained between block interfaces for greater sliding resistance and higher axial forces improve diagonal shear resistance.

It must be noted that those sustained forces in the masonry through pre- or post-tensioning will induce creep effects that must be taken into account. Research on the long-term behaviour of mortarless masonry has shown that creep effects may be accentuated as a result of stress concentrations at the contact points of adjacent courses (Marzhan and Konig, 2002). Creep coefficients were found to be dependent on the degree of roughness along bed joint surfaces and the level of applied stress. As a result, larger losses in prestressing forces are expected for mortarless masonry. However, contrary to mortar-bedded masonry, creep effects due to settlement of mortar joints will not take place in mortarless masonry.

An additional benefit of prestressing mortarless masonry is the fact that the large initial loading deformations between blocks at the onset of loading may be diminished. It was found that as mortarless masonry is first loaded large initial deformations occur due to closing of gaps, balancing of uneven surfaces and notches, etc. Thus, at the onset of prestressing stresses applied to a wall assembly, unwanted initial settlements will occur under controlled and manageable conditions.

The pre- or post tensioning of mortarless walls demonstrates how masonry has evolved into a new structural material, which is suitable for a wide range of engineering applications.

Surface bonding

The use of surface bonding, similar to fibre reinforced polymer (FRP) coating applications, is an innovative method to increase the strength of a masonry wall. Surface bonds develop their strength through the tensile resistance of small fibreglass fibres (approximately 3.8 mm) contained in a surface bonded cement-plaster coating which is trowelled onto both sides of a wall (ASTM C-887, 2001). Test data has shown that surface bonding can result in net flexural tension strength on the order of 2.07 MPa (TEK 14-22, 2003). Flexural capacity based on this experimental value exceeds that for conventional, unreinforced mortared masonry construction; therefore, it is considered conservative to apply S304.1-04 design principles to calculate flexural capacity.

In-plane shear strength of surface-bonded walls is attributed to frictional stresses developed along the bed joints resulting from vertical compression in addition to diagonal tension strength of the fibre coating.

Comparative Design Examples

Two design examples were undertaken to illustrate design methods and the strength comparison between traditional masonry and the Azar mortarless system. The two examples demonstrate the design of a flexural wall and a shear wall using 30 MPa blocks and steel reinforcement of 20M bars at 400 mm centre to centre. Table 2 summarizes the design.

Table 2 - Summary of Design Examples

	Shear wall		Flexure Wall (per meter)	
	Traditional	**Mortarless**	**Traditional**	**Mortarless**
Axial Resistance (kN)	3348	4315	1046	1348
Moment Strength (kN-m)	1668	1680	**Inadequate** 13.2	**Adequate** 17.4
Shear Reinforcement	Yes (15M@600)	No	N/A	N/A
Diagonal Shear (kN)	295	315	N/A	N/A
Sliding Shear (kN)	542	542	115	115

Refer to Appendix II for completed design examples.

As it can be seen from table 2, mortarless masonry is approximately 30% higher in axial capacity compared with the traditional system. This may be attributed to the higher compressive strength of masonry for mortarless systems. With traditional systems, mortar beds are in a triaxial stress state and lateral tensile stresses in the face shells and webs will prematurely fail the block. It should also be noted that the traditional shear wall required steel horizontal reinforcement to resist the applied shear force, whereas, the mortarless system did not. Thus, it can be concluded that the mortarless masonry system used, Azar block, has equivalent if not greater strength than that provided by the traditional masonry systems. The relatively low capacities experienced by the traditional masonry system can be attributed to the over-conservatism in the current Canadian Masonry Design Standard S304.1 (Hou, 2007).

BUILDING ENVELOPE ATTRIBUTES

All structures must be designed for strength and stability considerations, but functionality is equally as important. Considerations that account for moisture control, fire resistance, heat loss, durability, and sound transmission are among the serviceability requirements. Exploring the general behaviour of a building can be focused more closely on the materials being used and how they are connected to overcome different environmental conditions. Depending upon the climate or building application, different options may be more appropriate for certain situations.

Mortar joints are often considered to be the most vulnerable element when dealing with masonry walls and building science issues. When workmanship deficiencies are present with regards to the mortar, maintenance and repair costs decrease the economic feasibility of projects. Some common workmanship problems include moving a unit after its initial set, improper tooling, incompletely filling of head and bed joints, omitting horizontal wire reinforcing, and leaving mortar droppings in cores to be grouted (Hines, 1995). It is imperative for structures to be designed with attention to the overall cost of the project, including initial construction and ongoing operation and maintenance costs.

Water Permeation

Water permeation through a block wall is critical when considering the effectiveness of the building envelope. Over a period of time, water within the wall can develop into many different potential problems for a building leading to high repair costs and poor serviceability. Some issues that are prevalent in masonry structures that have experienced water penetration are mould, efflorescence, and cracking or spalling due to freeze-thaw cycles or salt crystallization.

The American Society for Testing and Materials (ASTM) has developed a standard test method for determining "water penetration and leakage through masonry." Developed in 1990, ASTM E514 provides guidelines for testing the amount of water permeation for masonry blocks. The test is based on a minimum 4-hour constant water spray that is applied with a wind pressure to simulate environmental conditions similar to that of a rain storm. A diagram of the testing apparatus has been presented in Figure 9. In order to account for the large amount of variability between testing atmospheres and conditions, an absolute wall leakage rating standard is impractical and discouraged (ASTM E514-04).

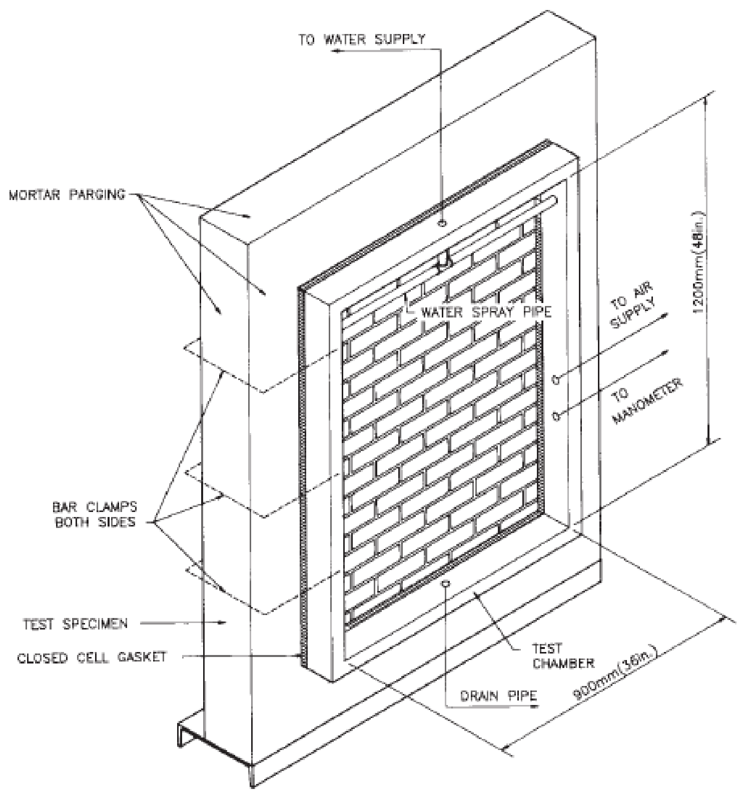

Figure 9 - Isometric projection of testing chamber (ASTM E514-04)

A study conducted by K.B. Anand and K. Ramaurthy was published in 2001 in the Journal of Architectural Engineering, among other journals. The technical portion focuses on comparing the water permeation through mortarless masonry, thin-jointed masonry, and mortar-bedded masonry. The tests conducted for the study also accounted for the effect of different surface finishes. Silblock masonry blocks were used in the experiment due to their simple geometry. The system interlocks in the horizontal and vertical directions, with discontinuity of the bed joint and cross joint from the inner to the outer faces (Anand and Ramaurthy, 2001). Tests performed as per ASTM E514 were conducted for dry-stacking of the blocks, thin-jointing (2-3 mm joints), and mortar bedding (10 mm joints). The study focuses on test results addressing two main criteria: dampness and leakage. Dampness of the specimen correlates to the visible area on the back of the wall, expressed as a percent of the chamber area. Leakage can be defined as the total water collected from each trough (ASTME 514, 2004). This is based on complete saturation, with liquid emerging on the back side of the wall.

The goal of the comparison is to understand how dry-stacking performs relative to conventional mortared blocks. The effects of a surface finish are dependent on variables such as the type of coating applied and the thickness of the application. In order to control porosity, all of the specimens were constructed of the same material and cured under identical conditions and duration. The results of the tests indicated that high levels of leakage and early dampness were observed with no surface finish and no additional pressure on the wall (Anand and Ramamurthy, 2001). Figure 10 outlines the observed results for dampness. The mortar-jointed blocks (both thin joints and regular joints) were not as effective as the mortarless system. This has been attributed to the discontinuity existing between blocks of successive layers in the mortarless specimen.

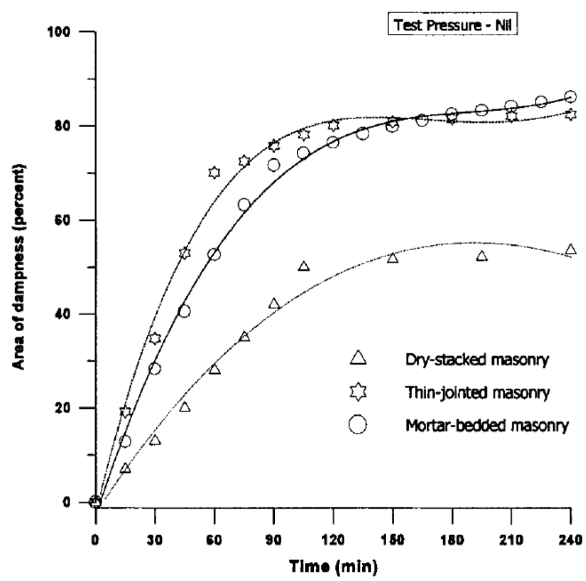

Figure 10 - Variation in area of dampness – no surface finish (Anand and Ramamurthy 2001)

Leakage through the wall can be measured and reported with greater accuracy than the visual estimates for the area of dampness. Figure 11 shows the recorded data for the variation of leakage that was present for the three specimens. The rate of leakage remained reasonably linear over the course of the experiment for all of the results. Only marginal variation in total leakage was observed between these specimens so the result is inconclusive in assessing the advantages of one system over another (Anand and Ramamurthy, 2001).

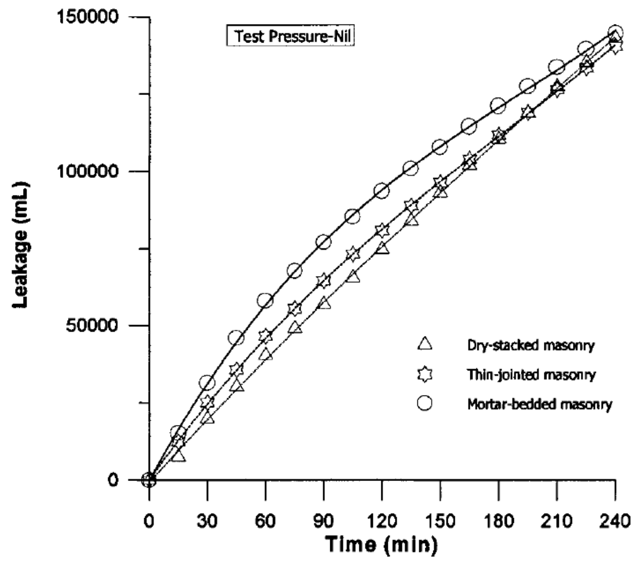

Figure 11 - Variation in leakage – no surface finish (Anand and Ramamurthy 2001)

The main focus of examining this study is the difference in joint quality between mortarless block and mortar jointed masonry with respect to water permeation. As previously noted, mortarless masonry performed better in the overall level of dampness. The study contributed more experimental data that showed improvements in the results with the use of different surface finishes. Unfinished walls are acceptable in applications that are protected against rain or for interior walls. The use of surface finishing or other techniques, such as construction with a multi-whythe block system, are recommended for exterior walls exposed to the environment.

Fire Resistance

Masonry structures are known to excel with regards to passive fire protection. Concrete block units are non-combustible, which helps prevent the spread of fires and allows for key structural components, such as block walls and columns, to remain standing during a fire event. Interlocking masonry blocks provide additional fire resistance by eliminating mortar. Mortar joints often appear to be more affected by fire exposure than the adjacent surface of the masonry units (TEK 7-5A, 2006). While exposed to high levels of heat, the joints experience levels of dehydration that can leave the mortar chalky or brittle. Loss of mortar is often enhanced by firefighters trying to control the blaze with high pressure water hoses. The water often flushes out any weakened mortar. This has been seen in tests performed in accordance with ASTM E 119, "Test Methods for Fire Tests of Building Construction and Materials" (TEK 7-5A, 2006). Deterioration of the structure may lead to loss in overall capacity or sufficient cracks where the fire gains potential to spread.

Sparlock Technologies Inc has created a specialized interlocking product that offers superior resistance to fire. This is because two Sparlock units are required to make an equivalent thickness of wall constructed of conventional units, which results in a heavier more substantial wall structure (Sparlock Technologies, 2010). A typical Sparlock interlocking wall is shown in Figure 12. Sparlock offers different block systems with fire ratings of 2, 4, and 6 hours.

Figure 12 - Sparlock Interlocking Block wall (Sparlock Technologies, 2010)

SUSTAINABLE DESIGN

Growing awareness for environmental health has become increasingly prevalent in recent years. At a global level, leaders are looking for ways to contribute towards making the planet a better place. Leadership in Energy and Environmental Design (LEED) evolved as a response to environmental concerns in order to encourage sustainable design. Established by the US Green Building Council in 1998, the organization has grown to become adopted by 30 countries as a standard rating system for building design (ICPI, 2005). During the design phase, a building applies to become a "LEED Certified Building" through the design team. As part of the development process, certain aspects of the design are evaluated for the number of LEED credits that can be earned.

Based upon the total number of credits achieved, the building is awarded a status of Certified, Silver, Gold, or Platinum. Having a higher level of certification will have greater initial costs, usually due to higher levels of technical systems or specialized materials, but the increased levels of efficiency result in lower operation and maintenance fees. Additionally, government programs or regional sponsors have been known to offer grants for the completion of a more environmentally friendly building.

Interlocking mortarless masonry may help contribute to a building's LEED status over and above the use of mortar-jointed masonry through two different ways. LEED has potential credit opportunities for "Innovation in Design;" if successful in achieving this, interlocking masonry would gain one additional credit. The intent of the credit is defined as (USGBC, 2005).

> "Provide design teams and projects the opportunity to be awarded points for exceptional performance above the requirements set by the LEED Green Building Rating System and/or innovative performance in Green Building categories not specifically addressed by the LEED Green Building Rating System."

For the specific innovative application of mortarless masonry, this credit may be awarded based on varying interpretations of its contribution to sustainable design. Construction processes often have an environmental impact through increased pollution: the use of vehicles, equipment, and possibly heaters can add up to a significant amount depending on the project. By greatly increasing the efficiency of the construction, the amount of pollution can be kept to a minimum.

Another interpretation could be that the overall amount of material used on the project has decreased. Mortar is usually a composition of Portland cement, water and sand. Portland cement production creates emissions that have proven to be harmful to the environment. By not requiring any mortar at all, the project has decreased pollution. For either interpretation, the larger the project is, the more significant the impact is on sustainability.

CONCLUSIONS

Interlocking mortarless masonry systems have been competing successfully against reinforced concrete in North America for years. Based on the market demand for efficiency in the construction process, one of the main limitations of the traditional procedure is that it is slow and labour intensive. In utilizing interlocking units without mortar, an unskilled crew with appropriate guidance can place more units in a given period of time. The combination of higher productivity and a less expensive labour force reduces labour costs significantly. A cost comparison revealed these potential savings to more than outweigh the additional material costs associated with interlocking units, and this was validated with a comprehensive case study.

The current editions of the National Building Code of Canada and the Canadian masonry design standard do not directly address mortarless masonry and this trend is reflected internationally as well. However, results from empirical testing can be used to validate the application of existing codes and standards. The onus is generally on the manufacturers of the various systems to provide guidance to the designer and ensure that their product conforms to regional regulations. The Azar™ Building Systems Company has gone through this process in Canada, and comparative design examples revealed the strength of this mortarless system to exceed that obtained from a conventional masonry wall.

For all its advantages, mortarless masonry is not without its limitations and it is not suitable for every project. However, there is a great deal of unrealized potential for this type of construction. In the appropriate applications, mortarless masonry has shown itself to be a cost-effective and viable alternative to conventional construction methods.

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LIST OF SYMBOLS

a	Depth of the assumed rectangular stress distribution, taken as 0.8c
A_e	Net section area
b	Width of the section resisting the shear
C_m	Compressive force in masonry for rectangular analysis
$C_{m,f}$	Compressive force in the flange for a T-section analysis
$C_{m,w}$	Compressive force in the web for a T-section analysis
d	Depth of reinforcement measured from the extreme compression fibre
e	Loading eccentricity
f_m'	Masonry compressive strength
f_{me}'	Masonry compressive strength under eccentric load
f_t	Masonry tensile strength in flexure
F_v	Unfactored specified shear strength, given as the lesser of 3 calculated values
I	Moment of inertia about the centroid of the section
M_r	Factored moment resistance
M_{ftot}	Factored moment taking secondary effects into consideration
P_f	Factored axial load
$P_{r,max}$	Factored Axial Resistance
Q	First moment of area about the centroid of the section
S_n	Net section modulus
t_f	Effective flange thickness of the block for T-section analysis
φ_m	Masonry resistance factor, taken as 0.6

Appendix I: Cost Comparison

Building Dimensions

Width =	15	m	Surface area =	3000	m^2
Length =	50	m			
Height =	4	m			

Traditional Construction

Block Dimensions: (Nominal)

Length =	400	mm	Surface area =	0.0800	m^2/block	Rebar spacing =	600	mm
Height =	200	mm	Mortar width =	32	mm			
Thickness =	200	mm	Mortar vol. =	3.78E-04	m^3/block			

Item	Quantity	Units	Material
Stack and mortar blocks	3000	m^2 (surface area)	Blocks: 37500 blocks @ $2.80 ea. = $105,000 Mortar: 14.2 m^3 @ $490 /$m^3$ = $6,938 $111,938 **Subtotal = $111,938**
Place vertical rein. (20M)	130	m (wall length)	4000 mm bars @ 600 mm = 867 m 2.355 kg/m @ $3.30 /kg = $7.77 /m **Subtotal = $6,735**
Grout cores (every 3rd core, 0.079 m^3/m^2)	1000	m^2 (surface area)	1000 m^2 @ $22.50 /$m^2$ = $22,500 **Subtotal = $22,500**
		Totals	**$141,174**

Labour	Equipment	Totals
3 bricklayer(s) @ $40.50 /hr = $121.50 /hr 1 brick helper(s) @ $32.15 /hr = $32.15 /hr $153.65 /hr 37500 blocks @ 30 blocks/hr = 1250 hrs **Subtotal = $192,063**	**Subtotal = $0**	**$304,001**
1 bricklayer(s) @ $40.50 /hr = $40.50 /hr 2041 kg @ 36.9 kg/hr = 55 hrs **Subtotal = $2,240**	**Subtotal = $0**	**$8,975**
1 bricklayer(s) @ $40.50 /hr = $40.50 /hr 2 brick helper(s) @ $32.15 /hr = $64.30 /hr 1 equip. oper. @ $39.05 /hr = $39.05 /hr $143.85 /hr 1000 m^2 @ 7.90 m^2/hr = 127 hrs **Subtotal = $18,209**	1 grout pump @ $126.80 /day 79.0 m^3 @ 12 m^3/day = 7 days **Subtotal = $888**	**$41,596**
$212,511	**$888**	**$354,573**
	Construction Time =	**1377 hrs**

Azar Mortarless Construction

Block Dimensions: (Nominal)

Length =	400 mm	Surface area =	0.0800 m^2/block	Rebar spacing =	600 mm
Height =	200 mm				
Thickness =	200 mm				

Item	Quantity	Units	Material
Stack blocks	3000	m^2 (surface area)	Blocks: 37500 blocks @ $3.07 ea. = $115,125 *Note: Azar unit prices are based on 2005 values (from Azar) that have been adjusted for 2009 with the Bank of Canada Inflation Calculator* **Subtotal = $115,125**
Place vertical rein. (20M)	130	m (wall length)	4000 mm bars @ 600 mm = 867 m 2.355 kg/m @ $3.30 /kg = $7.77 /m **Subtotal = $6,735**
Grout cores (fully grouted, 0.079 m^3/m^2)	3000	m^2 (surface area)	3000 m^2 @ $22.50 /$m^2$ = $67,500 **Subtotal = $67,500**
		Totals	**$189,360**

Labour	Equipment	Totals
3 bricklayer(s) @ $40.50 /hr = $121.50 /hr 1 brick helper(s) @ $32.15 /hr = $32.15 /hr $153.65 /hr 37500 blocks @ 300 blocks/hr = 125 hrs **Subtotal = $19,206**	**Subtotal = $0**	**$134,331**
1 bricklayer(s) @ $40.50 /hr = $40.50 /hr 2041 kg @ 36.9 kg/hr = 55 hrs **Subtotal = $2,240**	**Subtotal = $0**	**$8,975**
1 bricklayer(s) @ $40.50 /hr = $40.50 /hr 2 brick helper(s) @ $32.15 /hr = $64.30 /hr 1 equip. oper. @ $39.05 /hr = $39.05 /hr $143.85 /hr 3000 m^2 @ 7.90 m^2/hr = 380 hrs **Subtotal = $54,627**	1 grout pump @ $126.80 /day 237.0 m^3 @ 12 m^3/day = 20 days **Subtotal = $2,536**	**$124,663**
$76,073	**$2,536**	**$267,969**
	Construction Time =	**505 hrs**

Note: Construction times are calculated assuming that rebar installation is concurrent with block placement and grouting does not take place until construction is complete.

Appendix II: Design Examples

Design Example 1: In-plane Shear Wall

Given:

A shear wall 3.2 m long by 10.0 m high is constructed from fully grouted 200 mm (nominal thickness) concrete blocks. Design the Vertical and horizontal reinforcement required providing sufficient resistance against the following unfactored loads:

- Dead load = 300 kN (assume to act at wall centerline)
- In plane base shear force due to wind = 200 kN
- In plane overturning moment due to wind = 1200 kN-m

Notes:

- Take steel F_y=400 MPa

Solution:

1) Traditional Masonry Construction

A comparison is undertaken for both a standard block and a mortarless masonry system, Azar Block™. For consistency, both blocks have a compressive strength of 30 MPa.

Design will be according to Canadian Standard S304.1-04. From table 4, f'_m=13.5 MPa.

Factored loads:

- Dead load: P_f= 1.25*300 = 375 kN
  - P_f= 0.9*300=270 kN
- Wind load: V_f= 1.4*200 = 280 kN
- Moment: M_f= 1.4*1200 = 1680 kN

Determine amount of steel required:

$$A_{s,required} = \frac{M_f}{\phi_s f_y (0.67) l_w} = \frac{1680 * 10^6}{0.85 * 400 * 0.67 * 3000} = 2458 \text{ mm}^2$$

Try 10-20M bars, A_s=3000 mm²

Check minimum and maximum reinforcement ratios, taken as 0.13% and 2.0%, respectively.

The steel ratio is $\rho_{provided} = \frac{A_{steel}}{A_{gross}} = 0.45\% \;\therefore$ <u>OK</u>

The steel reinforcement layout will be 20M bars @ 400mm c/c with double bars at the end.

a) Flexural Strength

TRY – a N.A. between bars number 6 and 7. Say, c=800mm and ignore contribution from bars 6, 7, and 8 since these bars are not tied and may buckle upon compression loading.

Check strain level for the fifth bar: $\frac{0.003}{800} = \frac{\varepsilon_5}{500}$, we get $\varepsilon_5 = 0.001875 < \varepsilon_y$

Tension in the fifth bar will be $T_5 = AE\varepsilon_5 = 300 * 200{,}000 * 0.001875 = 106.8 \text{ kN}$

All other remaining bars, $T_1 = T_2 = T_3 = T_4 = T = 120 \text{ kN}$

$C_m = 0.85\phi_m f'_m \beta_1 cb = 0.85 * 0.6 * 13.5 * 0.8 * c * 190 = 1.0465c$

$P_f = C_m - T_r = 270 = 1.0465c - 4 * 400 * 300 * 10^{-3} - 1 * 375 * 300 * 10^{-3}$

Solving for c, we obtain 824mm. This is very close to our assumption. We must readjust our T_5 value:

Try c=816mm; thus, T_5=105.8 kN.

$$M_r = T * \left[2\left(d_1 - {}^{l}\!/_{2}\right) + \left(d_2 - {}^{l}\!/_{2}\right) + \left(d_3 - {}^{l}\!/_{2}\right) + \left(d_4 - {}^{l}\!/_{2}\right)\right] - T_5\left(d_5 - {}^{l}\!/_{2}\right) + C_m\left({}^{l}\!/_{2} - \frac{c * \beta_1}{2}\right)$$

Total moment resistance is found to be **Mr = 1668 kN** ~ Mf = 1680 kN → Under design by 1%

b) Shear Checks

i. Diagonal shear

$\frac{M_f}{V_f d_v} = \frac{1680}{280 * 0.8 * 3.2} = 2.34$, therefore, take factor as 1.0

$V_{r,max} = 0.4\phi_m\sqrt{f'_m}b_w d_v = 0.4 * 0.6 * \sqrt{13.5} * 190 * 2560 = 429\ kN$

$v_m = 0.16\left(2 - \frac{M_f}{V_f d_v}\right)\sqrt{f'_m} = 0.588 \text{ MPa}$

$V_m = \phi_m(v_m b_w d_v + 0.25 * P_d) = 0.6(0.588 * 190 * 2560 + 0.25 * 270) = 172\ kN \ll V_f = 280 \text{ kN}$

The wall will require horizontal reinforcement to resist the shear forces. Use 15M bars, A_v=200 mm²

$V_{s,required} = V_f - V_m = 280 - 172 = 108 \text{ kN} = 0.6\phi_s A_v f_y \frac{d_v}{s}$

Solving for s, we obtain a spacing of 683 mm or less. CHOOSE <u>s = 600 mm c/c.</u>

Therefore,

$$V_{s,provided} = 0.6^2 * 200 * 400 * \left[\frac{0.8 * 3200}{600}\right] = 122.9\ \text{kN}$$

$V_r = V_m + V_s = 295\ \text{kN} > V_f = 280\ \therefore$ OK

ii. Sliding Shear

$V_r = \phi_m \mu P_2 = \phi_m \mu (0.9 * \text{dead load} + \phi_s A_s f_y)$

Take $\mu = 0.7$ conservatively.

$V_r = 0.6 * 0.7(270 + 0.85 * 3000 * 400 * 10^{-3} = 542\ \text{kN} > V_f = 280\ \therefore$ OK

c) Check Axial Compression

$P_{r,max} = 0.8[0.85\phi_m f'_m tb] = 0.8 * 0.85 * 0.6 * 13.5 * 190 * 3200 = 3348\ kN \gg P_f$ OK

2) Mortarless Masonry Construction – Azar Block

Using Azar block,

f'_b=30 MPa, with f'_m=16.2 MPa. v_m=1.29 MPa (as per Azar design guide)

Using the same wall loading; thus, keep the same steel layout as above. 10-20M bars spaced at 400 mm c/c with double bars at each end.

a) Flexure

Assume N.A. is between bars 6 and 7. TAKE c=650 mm, C_m=930.5kN

Therefore, we must calculate the stress in bar 6 by: $\varepsilon_6 = \frac{250*0.003}{650} = 0.00115\ mm/mm < \varepsilon_y$

$$M_r = T * [2(d_1 - l/2) + (d_2 - l/2) + (d_3 - l/2) + (d_4 - l/2)] - T_5(d_5 - l/2) - T_6(d_6 - l/2) + C_m\left(l/2 - \frac{c * \beta_1}{2}\right)$$

$M_r = 120 * [2(3.1 - 1.6) + (2.7 - 1.6) + (2.3 - 1.6) + (1.9 - 1.6) - (1.6 - 1.3)] - 69(1.6 - 0.9) + 930.5\left(1.6 - \frac{0.8*0.65}{2}\right) = 1775\ \text{kN} - \text{m} \gg \mathbf{M_r = 1680\ kN - m}$ → Overdesign of 5.6% ∴ ***OK***

b) Shear check

For the shear checks, the manufacturer has specified shear strength of 1.29 MPa; however, a maximum shear value based on the compressive strength of the masonry is used as a ceiling values for diagonal shear capacity.

i. Diagonal Shear

$V_m = \phi_m(v_m b_w d_v + 0.25 * P_d) = 0.6(1.29 * (204 - 2 * 38) * 0.8 * 3200 * 10^{-3} + 0.25 * 270) = 486 \text{ kN}$

Check that the maximum shear of the grout is not exceeded at mortarless joints.

$$V_{r,max} = 0.4\phi_m\sqrt{f'_m} b_w d_v = 0.4 * 0.6\sqrt{16.2} * (204 - 2 * 38) * 0.8 * 3200 = 315\ kN\ V_{r,calculated}$$

The maximum shear resistance governs and is taken as 315 kN $> V_f = 280$ kN $\therefore$ OK

Since the wall vertical shear strength is greater than the applied loading, NO shear reinforcement is required.

ii. Sliding Shear

$V_r = \phi_m \mu P_2 = \phi_m \mu(0.9 * \text{dead load} + \phi_s A_s f_y)$

$V_r = 0.6 * 0.7 * [270 + 0.85 * 0.577 * 3000 * 400 * 10^{-3} = 542 \text{ kN} \gg V_f \therefore$ OK

c) Check Axial Resistance

$P_{r,max} = 0.8\left[0.85\phi_m f'_m tb\right] = 0.8 * 0.85 * 0.6 * 16.2 * 204 * 3200 = 4315\ kN \gg P_f$ OK

The Mortarless Masonry wall design is adequate with all respects and exceeds that of conventional masonry design.

Design Example 2: Out-of-plane Flexural Wall

Given:

A fully grouted 200 mm (nominal thickness) wall is 4.0 meters high and reinforced with 20M vertical bars at 400mm centre-to-centre. Determine if the wall is sufficient to carry the following unfactored loads:

- Dead load: 150 kN/m
- Live load: 100 kN/m

Notes:

- Take $I_{eff} = 0.25I_o$
- Apply minimum eccentricity as 10% of wall thickness
- Take F_y=400 MPa

Solution:

1) Traditional Masonry Construction

Again, reinforcement will be 20M bars @400 mm (in this case, double bars will not be used at the end).

Design will be according to Canadian Standard S304.1-04. From table 4, f'_m=13.5 MPa.

Check steel ratio: $\frac{A_s}{A_g} = \frac{750}{1000*190} = 0.39\% \ > 0.13\%$

Check Slenderness is less than 30:

$\frac{kh}{t} = \frac{4000}{190} = 21 \ < 31$, and we must account for slenderness effects.

Factored loads: P_f= 1.25*150+1.5*100= 338 kN/m

Apply load at minimum eccentricity of 10% of wall thickness, the factored primary moment is:

$M_{fp=}$0.1t*Pf=19*338 kN/m=6.42 kN-m/m

Using the Moment Magnifier method to account for P-δ effects,

$$M_{f,tot} = \frac{M_{fp}C_m}{1 - \frac{P_f}{P_{cr}}}$$

For simple supports use C_m=1.0 and take $I_{eff} = o.25I_0 = 0.25 * \frac{1000*190^3}{12} = 142.9 * 10^6 \ mm^4$

And $\beta_d = \frac{1.25*150}{338} = 0.555$

$$P_{cr} = \frac{\pi^2 \phi_{er} E_m I_{eff}}{(1+0.5\beta_d)(kh)^2} = \frac{\pi^2 0.75*11475*142.9*10^6}{(1+0.5*0.55)(4000)^2} = 593.8 \text{ kN}$$

$$M_{f,tot} = \frac{M_{fp} C_m}{1-\frac{P_f}{P_{cr}}} = \frac{6.42*1}{1-\frac{338}{593.8}} = 14.9 \; kN - m/m$$

Axial Resistance:

$$P_{r,max} = 0.8[0.85\phi_m f'_m tb] = 0.8*0.85*0.6*13.5*190*1000 = 1046 \; kN \gg P_f \;\; \underline{\text{OK}}$$

Calculating Moment Resistance:

$$C_m = 0.85\phi_m f'_m \beta_1 cb = 0.85*0.6*13.5*0.8*c*1000 = 5.508c$$

$$T_r = \varphi_s A_s F_y = 0.85*750*400 = 255 \; kN/m$$

$P_f = C_m - T_r$

c=107.7mm and a= 0.8c= 86.1 mm

$M_r = \varphi_s A_s F_y \left(d - \frac{a}{2}\right) = 0.85*750*400(95-86.1/2)$=13.24 kN-m/m < $M_{f,tot}$ ∴ **N.G.**

2) Mortarless Masonry Construction – Azar Block

From Azar block design guide: f'_m=16.2 MPa; E_m=18479 MPa

M_{fp}=0.1t*Pf=20.4*338 kN/m=6.895 kN-m/m

$$P_{cr} = \frac{\pi^2 \phi_{er} E_m I_{eff}}{(1+0.5\beta_d)(kh)^2} = \frac{\pi^2 0.75*18479*102.4*10^6}{(1+0.5*0.55)(4000)^2} = 685 \text{ kN}$$

$$M_{f,tot} = \frac{M_{fp} C_m}{1-\frac{P_f}{P_{cr}}} = \frac{6.895*1}{1-\frac{338}{685}} = 13.61 \; kN - m/m$$

Axial Resistance:

$$P_{r,max} = 0.8[0.85\phi_m f'_m tb] = 0.8*0.85*0.6*16.2*204*1000 = 1348 \; kN \gg P_f \;\; \underline{\text{OK}}$$

Calculating Moment Resistance:

$$C_m = 0.85\phi_m f'_m \beta_1 cb = 0.85*0.6*16.2*0.8*c*1000 = 7.02c$$

$$T_r = \varphi_s A_s F_y = 0.85*750*400 = 255 \; kN/m$$

$P_f = C_m - T_r$

c=84.5mm and, a= 0.8c= 67.6 mm

$M_r = \varphi_s A_s F_y \left(d - \frac{a}{2}\right) = 0.85 * 750 * 400(102 - 67.6/2)$ = 17.4 kN-m/m > $M_{f,tot} = 13.6$ ∴ **OK.**

Therefore, for the given loading, the traditional masonry block construction is inadequate with respect to moment capacity, whereas the Mortarless masonry system is sufficient.

Autoclaved Aerated Concrete (AAC) Masonry: Production, Construction and Design Methodology. Is AAC Suitable for the Canadian Environment and Market?

S. Alam, A.S. Dhamija and M. Raghami

Department of Civil and Environmental Engineering, University of Alberta

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ABSTRACT

Autoclaved Aerated Concrete (AAC) is a light weight building material produced from the natural resources available all around the world. It has several useful structural and architectural characteristics that make it a good choice for a wide variety of structural application. This paper firstly presents the materials for production and properties of AAC. It has a good thermal insulation and fire resistance in comparison with conventional concrete masonry unit. The production procedure and the structural design methodology of AAC are then explained. No toxic material is produced during the manufacturing process and energy consumption during production is less than that of some other building materials. Finally the applicability of AAC in the Canadian construction industry is investigated.

Keywords: AAC, Masonry unit, Thermal resistance, Strength Design.

INTRODUCTION

Autoclaved Aerated Concrete (AAC) is comparatively a new building material in North America especially in Canada, though in European countries AAC has been used for over 80 years. In the U.S., AAC was first introduced as a building material about 30 years ago. Then over the years, research and experiments have been conducted in the U.S. and codes and specifications developed for the production and uses of AAC. A number of plants are now in operation for the production of AAC in the U.S. and Mexico. AAC is now well accepted as a building material in the construction industry in the U.S. In Canada, AAC is now being used to a limited extent. This could be attributed to the higher cost of construction because of the unavailability of AAC locally, due to the abundant availability of wood as building material and, possibly, due to the lack of proper patronage.

Autoclaved Aerated Concrete can be used for both non-load bearing and load bearing structural elements. Durability, good thermal insulation and fire resistant properties makes AAC economical compared to some other building materials in the long run.

AAC is now being used in every continent of the world as a building material. In developed countries as well as in developing countries, AAC is considered as a first choice for building material because of the easy availability of the raw materials and simple manufacturing process. AAC is fully recyclable and has no adverse effect on the environment during the production or during the uses.

Section 1: Development of Autoclaved Aerated Concrete (AAC)

Autoclaved Aerated Concrete (AAC) was first developed in Sweden in the mid 1920's by Max Ginsberg [2.1]. It was patented there in 1923[2.2]. However, this invention was preceded by a discovery by the Swedes of a mixture of cement, lime, water and sand that expands by the addition of aluminium powder. Germany is the first country to follow Sweden in this technology and much improvement in this technology had been done by the Germans. Autoclaved Aerated Concrete (AAC) turned out to be the exact building material the German construction industry was looking for to meet the strict energy codes of Germany.

It was the Germans who took the first initiative to promote the AAC on an international level. After almost 45 years of AAC's first invention, Hebel, a German company, created a business relation with Asahi Chemical Materials Co., Japan. The AAC industry became well established in Japan, especially, after the unexpected stability of AAC buildings during the 1995 Kobe earthquake [2.3].

In 2005, the production of AAC nearly reached 26 million cubic yards, produced by over 95 manufacturers in 50 countries worldwide. The accessibility of raw material sources in almost any country of the world and the low energy consumption in its production process makes the AAC the material of choice even for developing countries and countries with low national energy supply [2.3]. In different countries, AAC is known by different names, *e.g.* Autoclaved Concrete, Autoclaved Cellular Concrete (ACC), Porous Concrete, Hebel (Aus), Aircrete and Thermalite (UK), BCA (Romania) [2.1].

For the US market, AAC was first imported in the beginning of the early eighties by some private home builders and they exerted a lot of effort for the approval of this new material as per the local and national building codes[2.2]. From 1999 a rising demand on AAC materials evolved which resulted into a wider acceptance of AAC in the US construction industry. Engineers of Hebel, Ytong and Babb International Inc. placed their efforts in consulting government code agencies, as various material samples and whole wall sections were tested in laboratories to reach full ASTM & UL acceptance [2.2].

<u>Section 2</u>: Production of AAC

2.1 Materials

Autoclaved Aerated Concrete (AAC) is made from abundantly available natural resources [2.4]. The raw materials are available in almost all countries of the world. The basic raw materials are Portland cement, limestone, aluminium powder, water and a larger proportion of a silica-rich material, usually fine sand or fly ash. AAC is made with all fine materials- nothing coarser than fine sand [2.5]. AAC made of fly ash are of slightly grey colour and the AAC made of quartz sand are of whiter colour. The toughness of the AAC could be improved by about 5% by including fly-ash based zeolites which also increases the freeze thaw resistance of the final product. In addition to the improved toughness, the zeolite AAC could absorb water vapour and perhaps even bacteria/viruses [2.6].

2.2 Production

The basic raw materials are weighed and mixed to a slurry in a mixer, addition of 0.05% - 0.08% aluminium powder (paste) gives the material the pre-specified density and spurs some important chemical reactions[2.7]. Aluminium (Al) powder is added at the last minute of mixing, just prior to moulding. Once the aluminium powder is added, the mix is stirred for about 30 seconds and then poured into well greased wagon like moulds. In the moulds, the cement and the lime react with the water. The cement dissolves and a small amount of hydrate is formed. The lime slakes and produces $Ca(OH)_2$ which removes some water from the mixture, thickening the slurry. The p^H of the solution rises to about 12[2.8]. Then the aluminium paste begins to react with the alkaline solution to produce hydrogen gas bubbles (bubbles up to 1/8 inch in diameter). The sample rises to about twice its original size. This is the aeration process that is responsible for the cellular character of the AAC [2.8]. At the end of the foaming process, the hydrogen (produced by the reaction of Al with $Ca(OH)_2$ and water) escapes to the atmosphere and is replaced by the air. AAC reinforced elements, such as floor panels, wall panels, lintels etc., contain steel reinforcement mesh that is treated for corrosion with water based acrylic, then placed in the mould before the slurry is added [2.4].

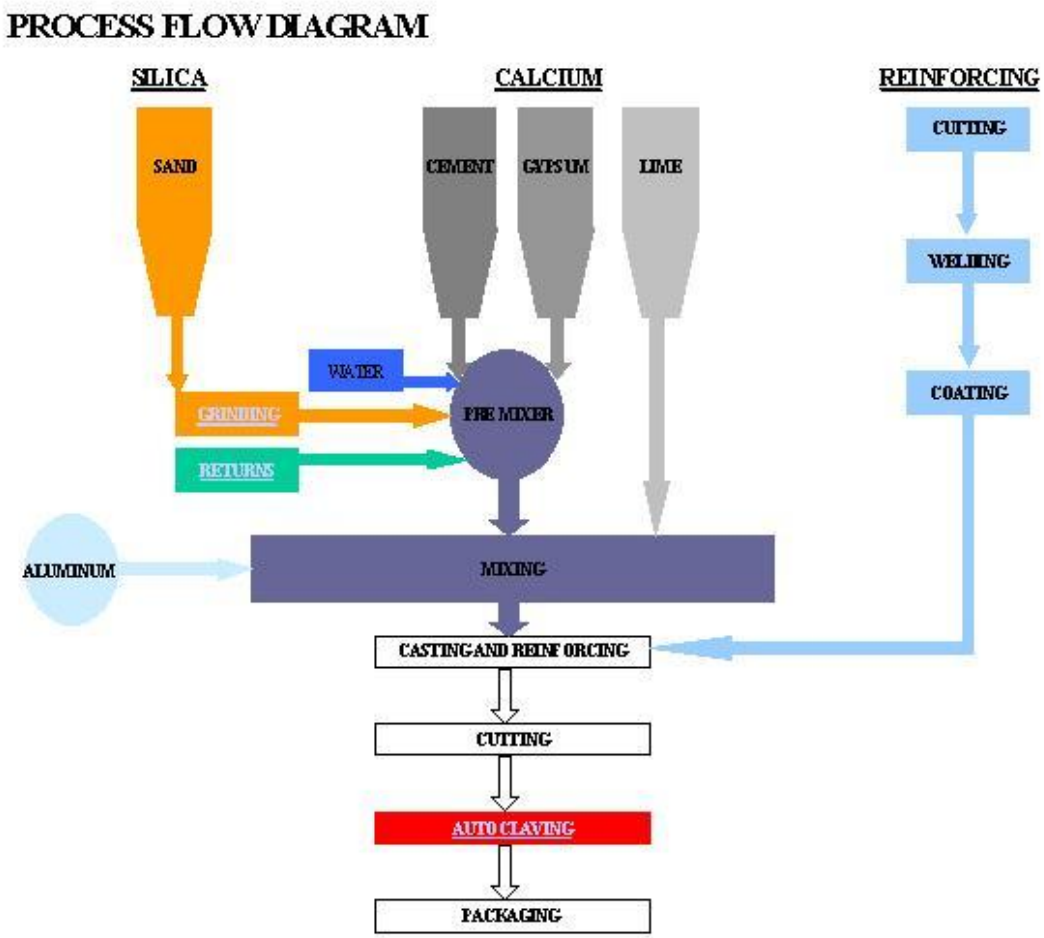

Fig. 2.1: Process Flow Diagram [2.7]

After a setting time which ranges from 30 minutes to 4 hours, the foam-like material stops increasing in volume and is hard enough to be wire cut into the desired shapes and move into an autoclave for curing. Depending on its density, up to 80% of the volume of the finished AAC is air; weight of AAC is 20% to 25% of normal weight concrete. This low density characteristic of AAC accounts for the low structural compressive strength which is around 1.2 ksi, approximately only about 10% of the compressive strength of regular normal weight concrete[2.1].

2.3 Curing and Handling

At the end of setting time the moulded material is cut to the pre specified shape of blocks and panels and moved to the autoclave for curing for a period of 8 to 14 hours. The autoclave uses high-pressure (8 to 12 bars) steam at temperature of about 356° F (180° C) to accelerate the hydration of the concrete and spurs chemical reactions that gives the AAC its strength, rigidity, and dimensional stability. Autoclaving can produce in 8 to 14 hours concrete strength equal to the strength obtained in a concrete moist-cured for 28 days at 70° F (21° C)[2.5]. The final products are shrink-wrapped in plastic and transported directly to the construction sites.

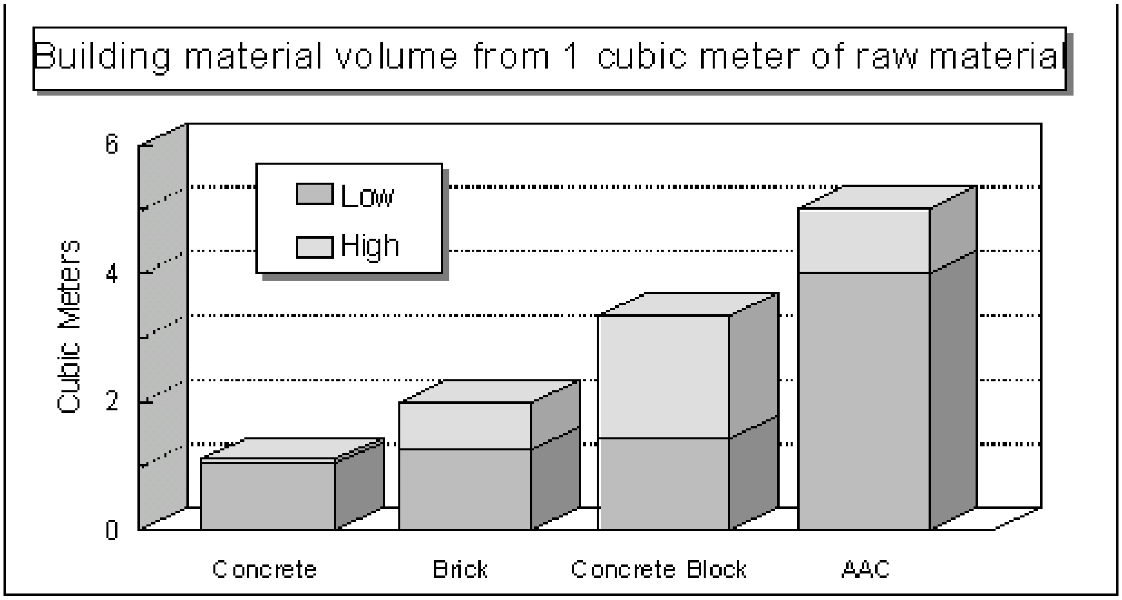

Fig. 2.2: Comparison of different building materials by volume [2.7]

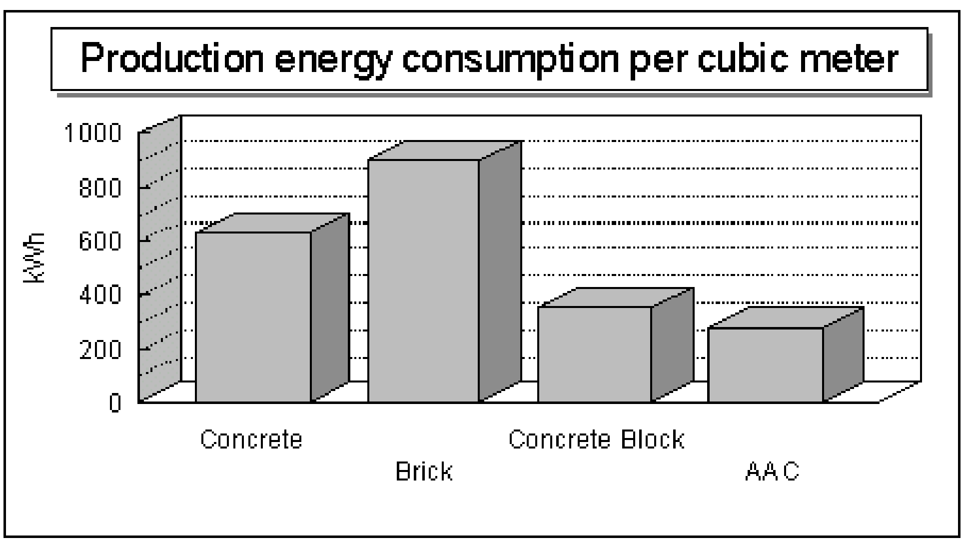

Fig. 2.3: Comparison of production energy consumption for different type of building materials [2.7]

From Fig. 2.3, it's clear that in comparison to that of normal weight concrete, the energy consumption during the production process of AAC is less than 50%.

AAC is an inert, non-toxic substance that has an energy-efficient and pollution free manufacturing process. The electric utility industry generates more than 50 million tons of fly ash each year as by-product – only a fraction of which can be recycled [2.5]. The use of fly ash in the manufacture of AAC keeps the fly ash out of the world's landfills and it also saves the AAC manufacturers the time and cost of ball milling quartz sands [2.9]. In addition to quartz sand and fly ash, AAC can be manufactured from other by-products such as silica rich mine tailings and recycled glass cullet as well as alternate sources of lime and Portland cement including ground granulated blast furnace slag and cement kiln dust. In fact, it is possible to produce AAC entirely from these by-products (2.9).

Section 3: Properties of AAC

3.1 Physical Properties

3.1.1 Thermal Insulation (3.1)

Thermal Conductivity "k" (Btu in/h ft^2 °F):

It is a measure of one of the thermal characteristic of a material as tested in thelaboratory that measures the heat flow through that material under steady and constant climate conditions. It is important to remember that laboratory conditions don't reflect the normal climate cycle. The lower the "k" value of the building material the higher the insulation. Table 3.1 gives values of thermal conductivity of AAC and other materials.

Table 3.1: Thermal Conductivity "k" of AAC and Other Materials

Designation	Thermal Conductivity, "k" (Btu in/h ft^2 °F)
AAC – 2/400 (25 pcf)	0.80 (1)
AAC – 4/500 (31 pcf)	0.97 (1)
Concrete Density 150 pcf	9.98 (2)
Insulation Board (polystyrene)	0.20 (2)
Steel	329.0
Water	4.15

(1) Based on ESR 2447 in accordance with ASTM C 1386
(2) ASHRAE (American Society of Heating, Refrigeration and Air-Conditioning Engineers)

The figures for AAC in Table 3.1 consider the typical moisture content of AAC during the lifespan. The moisture content at equilibrium for AAC depends upon the bulk density and climate conditions. Typically the moisture content ranges from 3% to 5% if the surrounding conditions are 23°C and 80% relative humidity. The conversion factor between a dry value of AAC and the equilibrium value (shown in Table 3.1) is about 1.05, i.e. in order to calculate the values for dry ones, the values in Table 3.1 have to be divided by 1.05 (3.1).

3.1.2 Thermal Resistance (R-Value)

It is defined as the inverse of the time rate of heat flow through a body from one of the bounding surfaces to the other surface for a unit temperature difference between the two surfaces, under steady state conditions, per unit area (h ft^2 °F/Btu). Thermal Resistance "R" is the opposite of the thermal conductivity and it is the resistance of a material to conduct or allow the heat flow. It is a measure of how well a material or a series of materials retards heat flow. It is rated as R-values. As R-value of a material or an element or assembly increases, the heat loss or gain through that material decreases [(3.1)]. Table 3.2 shows the thermal resistance calculated based on equation (1) for AAC [(3.1)].

$$R = (1/k) \times \text{Wall Thickness (inch)} \quad [1]$$ [(3.1)]

Table 3.2 Thermal Resistance of AAC

Designation	Thermal Resistance "R" (h.ft².°F/Btu)
8" AAC – 2/400 Wall	10.00
10" AAC – 2/400 Wall	12.50
12" AAC – 2/400 Wall	15.00
8" AAC – 4/500 Wall	8.25
10" AAC – 4/500 Wall	10.31
12" AAC – 4/500 Wall	12.37

In order to achieve an R_T - value (total assembly R – value) for AAC construction the additional resistance of the wall assembly are added together. Table 3.2 (a) shows an example calculation of a single – Wythe wall of AAC with render and stucco [(3.1)]. According to a report issued in 1989 by the Council of American Building Officials (BOCA) regarding the German ACC manufacturer, YTONG, the R- value per inch of ACC manufactured by YTONG is 1.66 per inch or 13.28 for an eight inch thick block (BOCA, 1989). In comparison, an eight inch thick traditional concrete block has an R- value of 1.20, which is 11 times lower than that of the same size ACC[(3.9)].

Table 3.2 (a): Calculation of an R_T - value of a Single – Wythe Wall with AAC – 4/500 (31 pcf)

Material	Thickness **(inch)**	Thermal Conductivity **K(Btu.in/h.ft².°F)**	R-value **(h ft² °F/Btu)**
Surface inside			0.68
Plaster inside	0.19	6.74	0.028
Hebel AAC – 4/500	10.00	0.97	10.31
Stucco	0.63	2.40	0.26
Surface outside			0.17
∑R = R_T-value			11.45

3.1.3 Coefficient of Heat Transmission

Represented as U – Factor (Thermal Transmission)

The U – Factor is defined as the coefficient of heat transmission (air to air) through a building component or assembly, which is equal to the time rate of heat flow per unit area and unit temperature difference between the warm side and cold side air films (Btu . ft² . °F) [W/ (m² .K]

Thermal transmittance, U-Factor, is a measure of how well a material or series of materials conduct heat. Low U-Factor values represent high insulation.

$U = 1/ (R_T\text{-value})$ (3.1) [2]

Table 3.3 shows a calculation of the U-value based on the data presented in Table 3.2(a).

Table 3.3: Calculation of R_T-value of a Single – Wythe Wall with AAC-4/500 (31 pcf) (3.1)

Material	**Thickness (inch)**	**Thermal Conductivity k (Btu.in/h.ft². °F)**	**R-value (h.ft².°F/Btu)**
Surface air film inside			0.68
Plaster inside	0.19	6.74	0.028
Hebel AAC-4/500	10.00	0.97	10.31
Stucco	0.63	2.40	0.26
Surface air film outside			0.17
∑R= R_T-value			11.45
U-value			0.87 (Btu/h.ft². °F)

3.1.4 Dynamic Thermal Performance

The thermal protection requirements for the external wall elements as per buildings energy code specifications are categorized into two groups, either summer or winter. In both cases the most important property of the material is a low thermal conductivity (or high R-value) combined with heat storage capacity. Both properties lead to a lower heat flux through the wall and therefore to a lower heating and cooling demand [3.1].

The thermal capacity of aerated concrete, which can delay influx of heat for up to 8 – 9 hours, is of considerable advantage in the desert climate. The heat of the day will not have any noticeable effect on the indoor environment but is retained in the AAC and released at night when the outdoor air is cool. Aerated Concrete provides variations as to the traditional building materials in these areas, e.g. compacted soil or clay [3.2].

3.1.5 Thermal Expansion and Contraction

It is important to give some consideration to thermal movements in the climates with high daily and yearly variations of the temperature.

The thermal expansion coefficient of aerated concrete is $8x10^{-6}/°C$, which is somewhat lower than for dense concrete (about $10\text{-}12x10^{-6}/^{o}C$). For AAC the effect of temperature is considerable. Due to the insulating properties of aerated concrete it will deform differently from, say, metal or dense concrete. The two latter materials mainly shows elongation or contraction with temperature changes. An aerated concrete unit exposed to thermal expansion and contraction exhibits a camber towards the side with increasing temperature. The elongation and contraction are quite small. The camber caused by solar heat can be as important as that caused by static load.

It is important to consider these deformations in a building, either by allowing the movements to take place without causing damage, or by preventing exposure to temperature by protection from solar radiation [3.2].

3.1.6 Fire Resistance Properties

AAC is a non- combustible material which provides a safer structure in the case of fire. AAC structures provide high level of fire containment to delay the spread of fire to other areas of the building. For example, a wall with a thickness of 8 inches made out of Hebel AAC provides a fire rating of four hours. During a fire, no toxic gases or vapours are emitted by AAC. By using fire-resistant materials like AAC, the owner of the building qualify for a possible reduction in fire insurance premiums.

AAC contains bound water which escapes at about 800 °C and tobermorite transforms to another mineral (β-wollastonite). The melting point is reached at approximately 1200 °C. Compressive strength of AAC increases (up to a maximum limit) with rising temperature; meanwhile shrinkage increases in this process which can cause the appearance of cracks [3.1]. A study conducted in UK shows that a four inch non-loadbearing wall of ACC without surface finishes has a fire resistance of four hours. In comparison, a four inch thick non-loadbearing dense concrete block has a fire resistance of two hours[3.9].

3.1.7 Moisture Resistance

Moisture resistance is needed for protecting a building against water to ensure a long life cycle and a healthy indoor climate. Moisture can get into a structure in various ways. It can be a result of the manufacturing process of building components as well as storage, transportation and installation (i.e. rain during construction) of materials. People, animals and plants increase humidity inside a building. Another possibility for the appearance of moisture inside a structure is condensation on cold surfaces of building materials. Although building components can absorb a certain amount of moisture, it is important to protect them from excess humidity to avoid damage [(3.1)]. In areas prone to high-moisture weather conditions, ACC combined with certain construction methods, greatly reduces dampness and condensation as compared to traditional concrete. For example, in the UK, ACC is used in combination with cavity wall construction, leaving conduits to prevent the interior of the home from becoming damp. A resin based moisture protective coating can also be applied as a moisture repellent[(3.9)].

3.1.8 Resistance to Pelting Rain

Building components have to be protected against climate influences like pelting rain. Therefore appropriate measures like rendering, coating, flashing, or cladding are necessary to secure the quality of a building. Minimum thickness for weather coverings on unit masonry are defined in the International Building Code. If stucco or exterior Portland Cement Plaster is used then a minimum thickness of 0.625 inch is required for a three-coat application and 0.5 inch for a two-coat application.

In general it is important for the rendering and coating to use materials which are suitable for the use on the material on which they are applied. This refers to mechanical and physical properties like compressive strength and vapour permeability. Otherwise the surface treatment can be damaged by these influences and moisture can penetrate the structure of buildings [(3.1)].

3.1.9 Resistance to Condensation Moisture

The amount of water vapour which can be absorbed by air depends on the air temperature. The warmer the air the more water vapour can be absorbed. When air reaches 100% humidity at a certain temperature this is called dew point. If this air cooled down from this condition fog develops. If this air meets a cold surface condensation forms there.

We would like to know the temperature in the middle of the wall.

$T = T_W - (q.R_1)$

Where

T = temperature on the wanted point in °F

T_W = temperature on the warmest side (in this case outside)

R_1 = thermal resistance – sum from the warmest side to the desired point

The R_1 is: 0.17 (air film outside) + 6.25 (½ R width of the wall) = 6.42

T = 90 °F – (1.5x6.42) = 80.37 °F or 26.9 °C > dew point

Conclusion: the condensation within the wall under extreme summer conditions occurs in the second half of the wall.

In the case of condensation within the AAC block there is no need for worry because AAC has a tremendous capability for storing water. AAC can compensate extreme conditions from outside of the surface. This leads to a good interior climate and to a lower demand of dehumidification and humidification provided by the air-conditioning systems. This is the primary reason that vapour barriers are not needed in most applications (3.1).

3.1.10 Diffusion Behaviour (3.1)

For all load carrying components made of mineral materials, AAC has the lowest vapour diffusion resistance. Monolithic AAC walls don't need additional layers of thermal insulation. This fact helps to build simple, permanently insulated wall assemblies. Basic rules which help to get diffusion behaviour of building materials are listed below:

- Permeability of a material must increase from inside to outside (S_D value must decrease from inside to outside).
- Penetrated moisture must be able to diffuse out.
- For multi-layered construction assemblies, the outer layer should be ventilated (3.1).

3.1.11 Water Absorption

Water absorption of materials can be classified by the water absorption coefficient. The water absorption coefficient gives information about how much water is absorbed in a defined time period. AAC has a very low water absorption coefficient in comparison to other building materials. The inner structure of AAC is of special nature; it consists mainly of closed pores (micro pores and macro pores), prevents the capillary transport of moisture over long distances. The water absorption coefficient for some selected building materials are shown in Table 3.4 (3.1).

Table 3.4: Water Absorption Coefficients for Different Building Materials [3.1]

Material	Water Absorption Coefficient $[kg/(m^2.h^{0.5})]$
gypsum	37 - 70
solid bricks	20 - 30
hollow bricks	9 - 25
solid sand – lime bricks	4 - 8
Hebel AAC	2.5 - 7
concrete	0.1 – 0.5
gypsum	35
lime-cement plaster	2 - 4
cement plaster	2 - 3
dispersion coating	0.05 – 0.2

3.1.12 Acoustic Properties

Noise control in buildings is of great significance for the health and well-being of the occupants, especially in residential dwellings, since they must provide an environment that is relaxing. The building envelope must also maintain privacy for the occupants. Noise control is also an important factor in other types of buildings such as schools, hospitals, and offices.

AAC, a porous concrete material, provides a sound insulation value up to 7 dB greater than other building materials of the same weight (/mass per area). The surface mass of AAC coupled with mechanical vibration energy damping within its porous structure produces a building material with exceptional sound insulation properties.

In addition to using a wall material with superior sound insulation properties in relation to its mass per area, it is always essential to construct the wall in a manner that closes off air leaks and paths by which noise can go around or through the assembly. Hairline cracks or small holes will increase the sound transmission through the wall at the higher frequencies. The simple construction methods of AAC and its details help to eliminate these cracks and holes in the walls, thus providing a final wall assembly, which offers superior sound insulation characteristics for the occupants[3.1]. Due to millions of independent air cells, which dampen sound transmission, AAC has excellent sound insulation and absorption quality[3.8].

The sound pressure level is the most important physical value to describe or quantify airborne noise inside and outside buildings. It is defined as the ratio between a base sound pressure in our atmosphere and the sound pressure caused by noise. The threshold of pain corresponds to a sound pressure of approximately 100,000,000 μPa, similar to a jet plane taking off at a distance of approximately 50 yards. The relationship of sound pressure to sound level is represented using a logarithmic scale [3.1].

3.2 Chemical Properties

3.2.1 Chemical Resistance

AAC is an alkaline material with a pH between 9.0 – 10.5, so it does not cause any corrosion to building materials. Like other types of concrete, AAC construction must be protected from high concentrations of carbon dioxide, sulphates, chlorides and strong acids [(3.1)]. AAC made from fly ash shows higher resistance to sulphate attack which is a common problem for civil engineering structures in the arid regions of the world[(2.7)].

3.2.2 Toxicity

AAC does not have any toxic substances or emit odours. Its production, management and disposal do not represent any health risks or damage to the environment [(3.1)].

Section 4: Design of AAC Structures

4.1 Introduction

AAC has been used in Europe for a long time and the structural design has been carried out according to manuals prepared by CEB (Euro-International Committee for Concrete or Comité Euro-International du Béton) and a number of European codes. However, as AAC is a new material in North America, new design standards were introduced in the U.S.A only recently.

The Masonry standards Joint Committee (MSJC), which is sponsored by the American Concrete Institute (ACI), the American Society of Civil Engineers (ASCE), and The Masonry Society (TMS), is responsible for development of masonry design provisions in the US [(4.1)].

The first set of proposed design provisions, commentary, and “super-commentary” was introduced to the ACI subcommittee 523A (i.e. Autoclaved Aerated concrete) in the fall of 2002. Because ACI 523A was a relatively new subcommittee at that time, the design provisions, commentary, and “super-commentary” were introduced as appendices to a non-mandatory design guide on AAC[(4.1)]. Table 4.1 provides a list of some of the common codes in the US for AAC structural design. ASTM codes specify technical specification for AAC material, and some experiments for quality verification under different circumstances. On the other hand, MSJC and ACI523 contain structural design provisions and the corresponding equations.

As AAC is not widely used in Canada currently, CSA S304.1 does not contain any design provision of AAC masonry and we have used American codes and standards in this report. In particular, in this section, we aim to introduce mechanical properties of AAC, some provisions of MSJC code and some structural design examples for AAC in comparison with CMU. SI units have been used for the structural design.

Table 4.1: American standards and codes

Code Name	Description
MSJC 2005a	Building Code Requirements for Masonry Structures, (ACI530-05 / ASCE 5-05 / TMS 402-05)
ACI523.4-R09	Guide for Design and Construction with Autoclaved Aerated Concrete Panels
ASTM C 426 Drying Shrinkage of Concrete Masonry units	Determine Material Shrinkage Characteristics
ASTM C 1386 Precast Autoclaved Aerated Concrete Wall Construction Units	Specification for Physical Requirements for AAC Block Products
ASTM C 1452 Reinforced autoclaved aerated concrete elements	Specification for Physical Requirements for Reinforced AAC Elements
ASTM E 72 Strength Tests of Panels for building construction	Determine wall flexural strength
ASTM E 447 Compressive Strength of Masonry Prisms	Determine compressive strength
ASTM E 518 Flexural Bond Strength of Masonry	Determine Flexural Bond strength
ASTM E 519 Diagonal Tension (Shear) in Masonry assemblage	Determine Wall Shear strength

4.2 Mechanical Properties

AAC products (i.e. blocks and panels) are produced in densities ranging from 400 kg/m^3 to 800 kg/m^3 and minimum design compressive strengths from 2.0 to 6.0 MPa. Table 4.2 shows the range of densities and compressive strengths for different Strength class.

Table 4.2: AAC Strength Class according to ASTM C1386 [4.5]

Strength Class	Minimum Compressive Strength (MPa)	Nominal Dry Bulk Density (kg/m^3)	Density Limits (kg/m^3)
AAC-2	2.0	400 500	350-450 450-550
AAC-4	4.0	500 600 700 800	450-550 550-650 650-750 750-850
AAC-6	6.0	600 700 800	550-650 650-750 750-850

Unlike CMU that tensile strength is independent of compressive strength, in AAC the tensile strength is a function of AAC compressive strength. Also, in AAC the modulus of elasticity depends on compressive strength. According to provisions provided by MSJC code[4.3], other mechanical properties of AAC can be summarized as Table 4.3.

Table 4.3: AAC mechanical properties

Strength category	AAC2	AAC4	AAC6	unit
Minimum Compressive Strength, f'_{AAC}	2.0	4.0	6.0	MPa
Shear nominal capacity $\left(0.066\sqrt{f'_{AAC}}\right)$	0.10	0.13	0.16	MPa
Modulus of Rupture $\left(0.4\sqrt{f'_{AAC}}\right)$	0.57	0.8	0.98	MPa
Modulus of Elasticity	1500	2040	2600	MPa
Coefficient of Thermal Expansion	8.1×10^{-6}	8.1×10^{-6}	8.1×10^{-6}	$/^{o}C$
Dry Density of Masonry Unit	400	500	650	kN/m^3

Note that there are some additional provisions about modulus of rupture in MSJC code.

4.3 Structural Design Methodology

AAC design approach is a strength approach that is similar to other masonry structural design and reinforced concrete design approaches. AAC masonry units can be considered as reinforced or unreinforced. When it is unreinforced, tensile and compressive stresses are resisted by AAC flexural tensile or by compressive strengths. Alternatively, in reinforced AAC tensile stresses are sustained by reinforcement.

4.3.1 Unreinforced AAC Masonry

Unreinforced AAC masonry should be designed to remain uncracked and the following assumptions should be considered for designing AAC as an unreinforced element [(4.3)]:

- "Strength design of members for factored flexure and axial load should be in accordance with principles of engineering mechanics.
- Strain in masonry shall be directly proportional to the distance from the neutral axis.
- Flexural tension in masonry shall be assumed to be directly proportional to strain.
- Flexural compressive stress in combination with axial compressive stress in masonry shall be assumed to be directly proportional to strain.
- Nominal compressive strength shall not exceed a stress corresponding to 0.85 f'_{AAC} .
- The nominal flexural tensile strength of AAC masonry shall be determined from MSJC A.1.8.3 section."

Also, nominal axial strength of unreinforced AAC wall is a function of: wall cross section, compressive strength of f'_{AAC} , and wall slenderness, and can be evaluated by the following equations:

$$P_n = 0.8 \times 0.85 A_n f'_{AAC} \left[1 - \left(\frac{h}{140r} \right)^2 \right] \quad \frac{h}{r} < 99 \quad \text{Equation(1)}$$

$$P_n = 0.8 \times 0.85 A_n f'_{AAC} \left(\frac{70r}{h} \right)^2 \quad \frac{h}{r} > 99 \quad \text{Equation(2)}$$

where,

An= net cross-sectional area of masonry, (mm^2)
f'_{AAC} =specified compressive strength of AAC, (MPa)
h= effective height of column, wall or pilaster, (mm)
r= radius of gyration, in. (mm)
Pn=nominal axial strength, (N)

4.3.2 Reinforced AAC Masonry

Similar to reinforced concrete and concrete masonry unit, there are some design assumptions for reinforced AAC as below [(4.3)]:

- There is strain continuity between the reinforcement, grout, and masonry such that applicable loads are resisted in a composite manner.
- The nominal strength of singly reinforced masonry cross sections for combined flexure and axial load shall be based on applicable conditions of equilibrium.
- The maximum usable strain, ε_{mu} at the extreme masonry compression fiber shall be assumed to be 0.003.
- Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis.
- Stress in reinforcement shall be taken as E_s times steel strain but no greater than f_y.
- The tensile strength of masonry shall be neglected in calculating flexural strength but shall be considered in calculating deflection.
- The relationship between masonry compressive stress and masonry strain shall be assumed to be defined by the following: Masonry stress of 0.85 f'_{AAC} shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance a = 0.67c from the fiber of maximum compressive strain. The distance c from the fiber of maximum strain to the neutral axis shall be measured perpendicular to that axis.

Nominal axial strength of reinforced AAC wall can be evaluated by the following equations:

$$P_n = 0.8 \times \left(0.85\left(A_n - A_s\right)f'_{AAC} + A_s f_y\right)\left[1 - \left(\frac{h}{140r}\right)^2\right] \quad \frac{h}{r} < 99 \quad \text{Equation (3)}$$

$$P_n = 0.8 \times \left(0.85\left(A_n - A_s\right)f'_{AAC} + A_s f_y\right)\left(\frac{70r}{h}\right)^2 \quad \frac{h}{r} > 99 \quad \text{Equation (4)}$$

where,

A_S= effective cross-sectional area of reinforcement, (mm^2)
f_y = specified yield strength of steel for reinforcement, (MPa)

As AAC was a new construction material in the US a decade ago and as CEB does not contain design provisions in some categories like shear wall design for in-plane load, seismic design etc. some investigation and research needed to be done in order to generate new provisions. For instance, a research program at the University of Texas was developed to test 19 shear wall specimens made from a variety of AAC elements, including reinforced and unreinforced panels, laid either horizontally or vertically [(4.2)]. During experiments where axial and lateral forces were applied on the full scale shear walls, the five following modes of failure were observed.

- Flexural cracking
- Flexure-shear cracking
- Web-shear cracking
- Crushing of the diagonal strut
- Nominal flexural capacity

The test setup and result are presented in Tanner et al. (2005b)

Similar to other types of concrete masonry and reinforced concrete, nominal shear strength of reinforced AAC consist of masonry and steel shear strength. Equation (5) shows this.

$V_n=V_{n,AAC}+V_S$ Equation (5)

Also, there is a maximum limitation for shear force according to the following equations:

$$V_n \le 0.5A_n\sqrt{f'_{AAC}} \qquad \frac{M_u}{V_u d_v} \le 0.25 \quad \text{Equation (6)}$$

$$V_n \le 0.33A_n\sqrt{f'_{AAC}} \qquad \frac{M_u}{V_u d_v} \ge 1.0 \quad \text{Equation (7)}$$

where,

D_v=actual depth of masonry in direction of shear considered, (mm)

V_n= Nominal shear strength, (N)

M_u= factored moment, (N.mm)

V_u=factored shear force, (N)

As the shear wall tests have shown, there are different failure modes for shear. Therefore, nominal shear strength of AAC masonry is the minimum nominal shear strength corresponding to web-shear, crushing of diagonal strut and sliding shear.

Web-shear failure depends on if AAC masonry is used with mortared head joints or with un-mortared head joints. Hence, nominal masonry shear strength should be computed according to the following equations:

$$V_{n,AAC} = 0.08 l_w t\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{0.2 l_w t\sqrt{f'_{AAC}}}} \quad \text{AAC with mortared head - joints} \quad \text{Equation (8)}$$

$$V_{n,AAC} = 0.055 l_w t\sqrt{f'_{AAC}}\sqrt{1+\frac{P_u}{0.2 l_w t\sqrt{f'_{AAC}}}} \quad \text{AAC with unmortared head - joints} \quad \text{Equation (9)}$$

$$V_{n,AAC} = 0.075\sqrt{f'_{AAC}}A_n + 0.05P_u \quad \text{AAC in other than running bond} \quad \text{Equation (10)}$$

where,

l_w=length of entire wall or of the segment of wall considered in the direction of shear force, (mm)

P_u=factored axial force, (N)

t= nominal thickness of wall, or overall depth of member cross-section, (mm)

Crushing of the diagonal strut should be checked only when $\frac{M_u}{V_u d_v} < 1.5$ according to the follow equation:

$$V_{n,AAC} = 17 \times 10^4 f'_{AAC}\, t \frac{h l_w^2}{h^2 + \left(\frac{3 l_w}{4}\right)^2} \quad \text{Equation (11)}$$

And finally sliding shear at an unbounded interface, nominal shear strength shall be checked by:

$$V_{n,AAC} = \mu_{AAC} P_u \quad \text{Equation (12)}$$

where,

μ_{AAC} =coefficient of friction of AAC

When there is an out of plane load, shear strength should be check based on the following equation.

$$V_{n,AAC} = 0.066 \times bd\sqrt{f'_{AAC}} \quad \text{Equation (13)}$$

where,

b= width of section, (mm)
d= distance from extreme compression fibre to centroid of tension reinforcement, (mm)
Also, there are some special provisions for AAC beam; column, pier and wall design that make some differences in AAC design with CMU (concrete masonry unit) design.

Here we just point out some of them briefly for beam and column and reference (4.3) contains more information.

Special provisions for AAC beam design are as follows:

- The maximum value of axial compressive force on a beam should not be greater than $0.05 A_n f'_{AAC}$
- The minimum nominal flexural strength of a beam should be greater than 1.3 M_{cr}.
- When $V_u > \varphi V_{n,AAC}$, transverse reinforcement is needed. Minimum area of transverse reinforcement is $0.0007bd_v$.
- The first transverse bar should be placed in a space less than one-fourth of the beam depth, from the end of the beam.
- The maximum spacing of the transverse bars should be less than the one-half the depth of the beam or 1200 mm.
- The minimum depth of a beam is 200 mm and beams should be full grouted.

Special provisions for AAC column design:

- The maximum distance between lateral supports of a column should be less than 30 multiplied by its nominal width.
- The minimum depth of a column is 200 mm and the maximum depth should be less than 3 multiplied by its width.
- Columns should be fully grouted.

4.3.3 Reduction Factors

Canadian codes (CSA A.23 and S403) consider different strength reduction factors for concrete and steel in order to evaluate the strength of the concrete or masonry sections, however American codes (ACI and MSJC) use the same reduction factor for both concrete and steel but they are different in different failure modes.

Strength reduction factors according to MSJC[(4.3)] should be considered as described in the Table 4.4.

Table 4.4: Strength reduction factor extracted from MSJC

Strength Reduction Factor	Failure Case
Flexure, axial load	Reinforce AAC 0.9
	Unreinforced AAC 0.6
Shear	0.8
Bearing	0.6

4.3.4 Load Combinations

Similar to reinforced concrete and steel structures there are some mandatory load combinations that should be checked for masonry structures. ASCE 7[(4.7)] and IBC2003[(4.8)] determine load combination for different types of structures. Some of them that usually are used in structural design for masonry buildings are as follows [(4.4)]:

$1.4D$

$1.2D + 1.6W + L + 0.5\ (L_r \text{ or } S \text{ or } R)$

$1.2D + 1.6\ (L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$

$0.9D + (1.6W \text{ or } 1.0E)$

Where:

D= Dead load

L= Live load

L_r= Roof live load, including permitted reductions

S= Snow load

R= Rain load

W= wind load

E= Earthquake load

4.4 Structural Design Examples

As the design of walls is an important aspect in masonry structural design, in this section, we discuss the design procedure of AAC wall via two different examples. These examples are chosen from reference 4.4 with some modification for simplicity and brevity. In both of these examples, the walls are subjected to an eccentric axial load and out of plain wind pressure which results to combination of axial load and out of plain bending moment. Further examples for other cases can be found in reference 4.1 and 4.4.

Example No.1:

Design an unreinforced wall that is subjected to dead and live loads as shown in Fig. 4.1. The wall is simply supported at both ends and has a thickness of 200 mm (solid unit, $f'_{AAC} = 6$ MPa) .

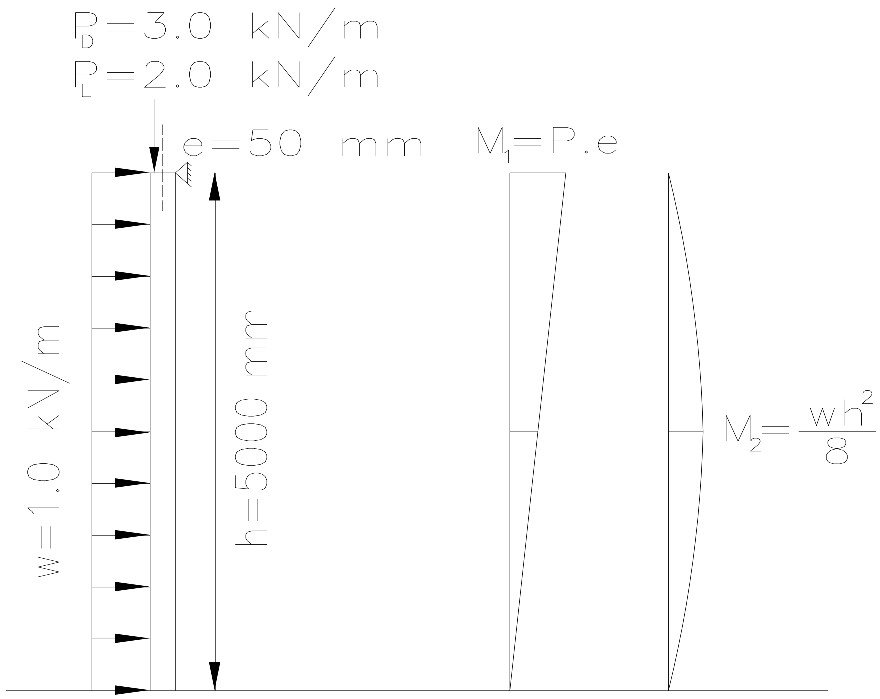

Fig.4.1 AAC masonry wall in example No.1

At first critical point and critical load combinations should be determined. It is obvious that critical point is at the mid-height of the wall. For compression control 1.2D + 1.6L and for tension control 0.9D + 1.6W should be checked. Similar to unreinforced concrete masonry unit, tension limitations usually governs the design but here we checked both of them.

The following conditions should be checked for controlling the wall section:

- Maximum ultimate axial load should be less than the slenderness-dependent values given by equation 1 or 2.
- Maximum compressive stress should be less than $\varphi \times 0.85 f'_{AAC} = 0.6 \times 0.85 \times 6 = 3.1$ MPa

- Maximum tension stress should be less than the modulus of rupture in the extreme tension fiber

$$A = 250 \times 10^3 \ \text{mm}^2/\text{m}$$

$$I = \frac{1000 \times 250^3}{12} = 1.3 \times 10^9 \ \ \text{mm}^4/\text{m}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1.3 \times 10^9}{250 \times 10^3}} = 72.1 \ \text{mm/m}$$

$$\frac{h}{r} = \frac{5000}{72.1} = 69.2 < \ 99 \longrightarrow \text{Equation 1 should be used}$$

$$P_u = 1.2D + 1.6L = 1.2 \times 3 + 1.6 \times 2 = 6.8 \ \text{kN/m}$$

$$\varphi P_n = \varphi \times 0.8 \times 0.85 A_n f'_{AAC} \left[1 - \left(\frac{h}{140r} \right)^2 \right]$$

$$\varphi P_n = 0.6 \times 0.8 \times 0.85 \times 250 \times 10^3 \times 6 \times \left[1 - \left(\frac{5000}{140 \times 69.2} \right)^2 \right] \times 10^{-3}$$

$$\varphi P_n = 449 \ \text{kN/m} >> P_u$$

So the wall section satisfies slenderness limitation. Now we should check maximum compression and tension stresses.

$$M_u = 0.9 \frac{P_D e}{2} + 1.6 \frac{wh^2}{8} = \left(0.9 \frac{3 \times 50}{2} + 1.6 \frac{1 \times 5000^2}{8} \right) \times 10^{-6} = 5.1 \ \text{kN.m}$$

$$\sigma = 0.9 \frac{P_D}{A_n} + \frac{M_u}{S_n}$$

$$= 0.9 \frac{3 \times 10^3}{250 \times 10^3} + \frac{5.1 \times 10^6}{1000 \times 250^2/6} = 0.01 + 0.49 = 0.6 \ \text{MPa}$$

$$\sigma_{allowable} = \varphi \times 0.8 \times f'_{AAC} \ = 0.6 \times 0.85 \times 6 = 3.1 \ \text{MPa} > \ 0.6 \ \text{MPa} \ \ \text{ok}$$

Maximum compression stress due to 1.2D+1.6L+0.8W load combination is 0.26 MPa which is less than 2.04 MPa.
As mentioned before, the tension control also should be checked as follows:

$$\sigma = -0.9\frac{P_D}{A_n} + \frac{M_u}{S_n}$$

$$= -0.9\frac{3\times10^3}{200\times10^3} + \frac{5.1\times10^6}{1000\times200^2/6} = -0.01+0.49 = 0.48 \text{ MPa}$$

$$\sigma_{allowable} = \varphi\times0.4\times\sqrt{f'_{AAC}} = 0.6\times0.4\times\sqrt{6} = 0.59 \text{ MPa} > 0.48 \text{ MPa ok}$$

Now, out of plane shear should be checked.

$$V_u = 1.6\times\frac{wh}{2} = 1.6\times\frac{1.0\times5}{2} = 4.0 \text{ kN/m}$$

$$\phi V_n = \phi\times0.066\sqrt{f'_{AAC}}\times bd = 0.8\times0.066\sqrt{6.0}\times1000\times250\times10^{-3} = 32.3$$

$$\phi V > V_u$$

Therefore, wall section satisfies all design provisions.

<u>Example No.2</u>:
Design a 200 mm reinforced AAC wall with 20M@600 (vertical bars) that is subjected to dead, live and wind pressure as shown in Fig. 4.2.

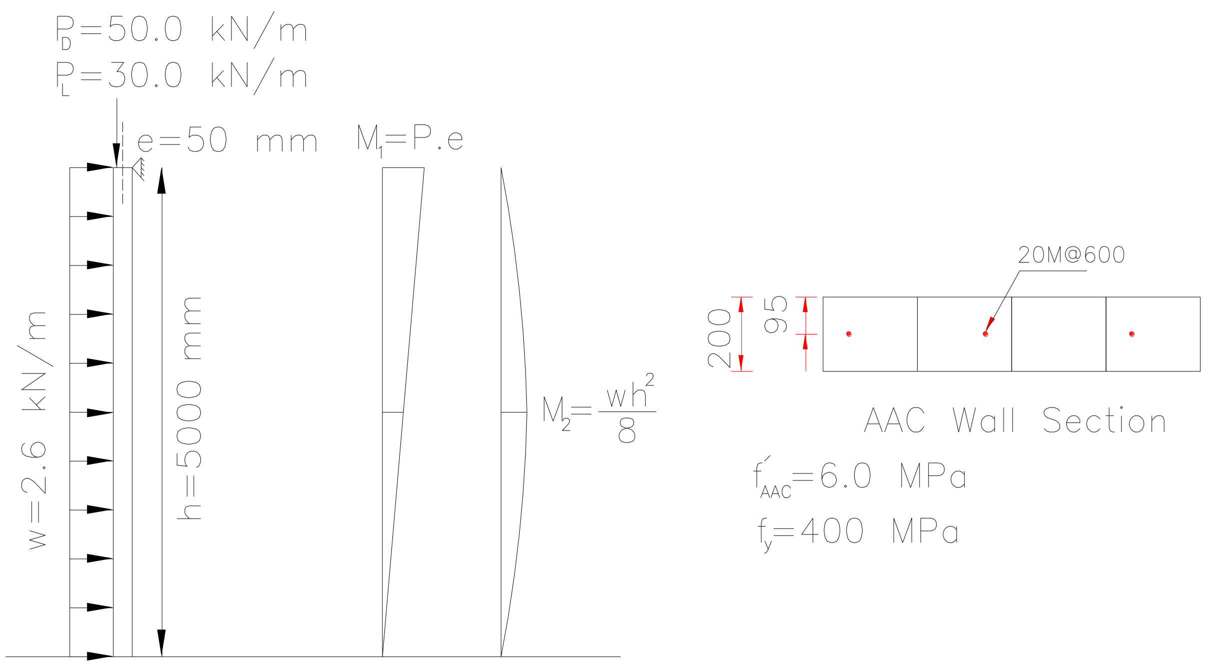

Fig.4.2 AAC masonry wall in example No.2

According to ASCE7(4.7), the following load combinations are considered :

1- 1.2D+0.5L+1.6W

2- 1.2D+1.6L+0.8W

3- 0.9D+1.6W

Corresponding axial loads and bending moments to the load combinations are shown in the following table:

P (kN/m)	M (kN.m/m)
75.0	14.9
108.0	9.2
45.0	14.1

In order to construct the interaction diagram, MSJC provisions should be considered and compression force in the vertical reinforcement will be neglected. By using the following equations in a spreadsheet, interaction diagram is constructed as shown in Fig. 4.3.

$$\frac{h}{r} = \frac{5000}{57.7} = 86.7 < 99$$

$$\varphi P_n = 0.9 \times 0.8 \times \left(0.85 f'_{AAC}(A_n - A_s) + f_y A_s\right) \times \left(1 - \left(\frac{h}{140r}\right)^2\right)$$

$$\varphi P_n = 0.9 \times 0.8 \times \left(0.85 \times 6.0\left(200 \times 10^3 - 500\right)\right) \times \left(1 - \left(\frac{5000}{140 \times 57.7}\right)^2\right)$$

$$\varphi P_n = 473.2 \text{ kN}$$

$$C = 0.67c \times 0.85 f'_{AAC} \times b$$

$$T = A_s f_s \quad , \quad f_s = E_s \varepsilon_s \quad , \quad \varepsilon_s = 0.003 \times \frac{d-c}{c} \leq 0.002$$

$$\varphi M_n = 0.9 \times \left(T \times \left(d - \frac{h}{2}\right) + C \times \left(\frac{h}{2} - \frac{\beta_1 c}{2}\right)\right)$$

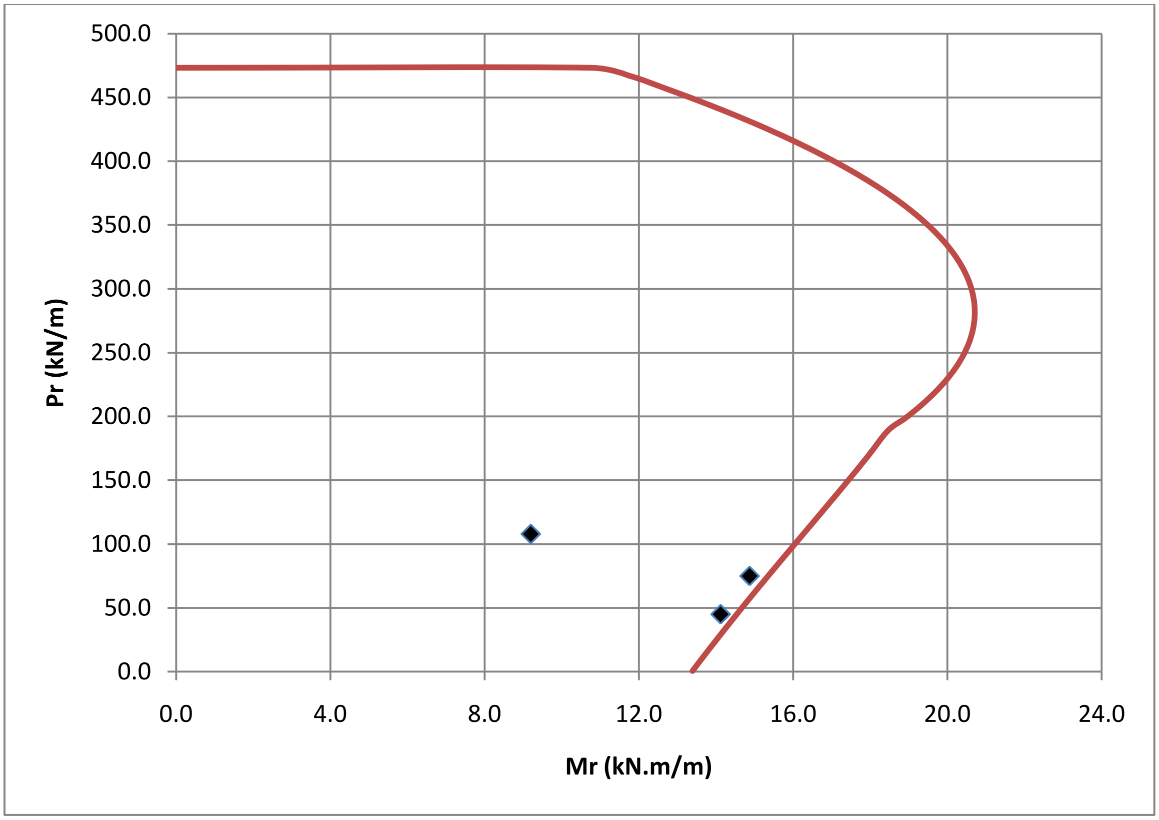

Fig.4.3 Interaction diagram for AAC wall in Example No.2

As it can be seen from Fig. 4.3, all points of load combinations lie within the moment-axial force interaction diagram and AAC wall with the given specification can carry out its loading.

4.5 Comparison of Structural Design of AAC and CMU

Section 4.2 briefly explained some mechanical properties of AAC. As some of these mechanical properties are different from CMU properties, AAC masonry units need some different structural design provisions compared to CMU design. In this section, we highlight some of these differences and compare the design of AAC wall and CMU wall.

Among the factors that make the design of AAC different than CMU, the two most import ones are: compressive strength and tensile strength. In CMU, compressive strength is much higher than in AAC units. This will force some limitations in some cases for AAC. On the other hand, tensile strength in CMU is lower than in AAC and this will make AAC a good choice for some other cases as will be explained more in this section.

Following, we discuss two examples, one demonstrating a case where AAC is a better choice than CMU, and another where CMU works better.

The first example is the example No.1 discussed in section 4.5. For AAC, the values for all parameters are the same as the values given in previous section and the design solution was provided in that section. For CMU, the thickness and the compressive strength of the wall are

200mm (solid unit) and 10MPa respectively. For solving this example for CMU case, the following steps should be done:

At first, according to Table 4 and Table 5 of CSA S304.1[4.6], factored compressive and tensile strength are as follows:

$$\varphi_m f'_m = 0.6 \times 5.0 = 3.0 \ \text{MPa}$$
$$\varphi_m f_t = 0.6 \times 0.4 = 0.24 \ \text{MPa}$$

Next, slenderness control should be checked based on the following equations:

$$e = 50 \text{ mm} > \ e_{min} = 0.1t = 19 \text{ mm ok}$$
$$\frac{kh}{t} = \frac{1.0 \times 5000}{190} = 26.3 \ > 10 - 3.5\frac{e_1}{e_2} = 8.7 \ , \ \frac{kh}{t} < 30$$
$$A_e = 190 \times 10^3 \ \text{mm}^2/\text{m} \ , \ S_x = 6.02 \times 10^6 \ \text{mm}^3/\text{m} \ , \ I_x = 572 \times 10^6 \ \text{mm}^4/\text{m}$$

The above equation shows that slenderness effects should be considered.

Also, the critical load combinations must be checked according to CSA S304.1[4.6] as shown in the following:

$$\text{Load Combination 1: } 1.25D + 1.5L + 0.4W$$
$$P_f = 1.25 \times 3 + 1.5 \times 2 = 6.75 \ \text{kN/m} \ , \ M_f = 6.75 \times \frac{0.05}{2} + 0.4 \times \frac{1.0 \times 5.0^2}{8} = 1.42 \text{ kN.m/m}$$
$$\text{Load Combination 2: } 1.25D + 0.5L + 1.4W$$
$$P_f = 1.25 \times 3 + 0.5 \times 2 = 4.75 \ \text{kN/m} \ , \ M_f = 4.75 \times \frac{0.05}{2} + 1.4 \times \frac{1.0 \times 5.0^2}{8} = 4.49 \text{ kN.m/m}$$

By using moment magnifier method, the wall section should be checked for the worst case of the load combinations:

Load Combination 1:

$$M_{fTOT} = M_{fp} \frac{C_m}{1-\frac{P_f}{P_{cr}}} \quad , \quad C_m = 0.6 + 0.4\frac{M_1}{M_2} = 0.6 + 0.4\frac{e_1}{e_2} = 0.6 + 0.4 \times \frac{19}{50} = 0.752$$

$$P_{cr} = \frac{\pi^2 EI}{(kh)^2}$$

$$EI = \frac{\varphi_e E_m I_{eff}}{1+0.5\beta_d} \quad , \quad \varphi_e = 0.65$$

$$E_m = 850 f'_m = 850 \times 5 = 4250 \text{ MPa} < 20 \times 10^3 \text{ MPa}$$

$$I_{eff} = 0.4 \times I_o = 0.4 \times 572 \times 10^6 = 228.8 \times 10^6 \text{ mm}^4 / m$$

$$\beta_d = \frac{D_f}{D_f + L_f} = \frac{1.25 \times 3.0}{6.75} = 0.56$$

$$EI = \frac{0.65 \times 4250 \times 228.8 \times 10^6}{1+0.5 \times 0.56} = 4.938 \times 10^{11} \text{ N-mm}^2 / m$$

$$P_{cr} = \frac{\pi^2 \times 4.938 \times 10^{11}}{(5000)^2} \times 10^{-3} = 195 \ kN \quad , \quad P_f = 6.75 \ kN$$

$$M_{fTOT} = 1.33 \times \frac{0.752}{1-\frac{6.75}{195}} = 1.04 \longrightarrow M_{fTOT} = 1.33 \ kN.m/m$$

Load Combination 2:

$$P_f = 4.75 \text{ kN/m} \quad , \quad M_{fTOT} = 4.43 \text{ kN.m/m}$$

Compressive stress control:

$$\sigma_1 = \frac{P_f}{A_e} + \frac{M_f}{S_e} = \frac{6.75 \times 10^3}{190 \times 10^3} + \frac{1.42 \times 10^6}{6.02 \times 10^6} = 0.27 \text{ MPa} < \varphi_m f' = 3.0 \text{ MPa}$$

$$\sigma_2 = \frac{P_f}{A_e} + \frac{M_f}{S_e} = \frac{4.75 \times 10^3}{190 \times 10^3} + \frac{4.49 \times 10^6}{6.02 \times 10^6} = 0.77 \text{ MPa} < \varphi_m f' = 3.0 \text{ MPa}$$

For controlling tensile stress, dead load factor should be considered as 0.9 rather than 1.25:

$$\sigma_1 = -\frac{P_f}{A_e} + \frac{M_f}{S_e} = -\frac{5.7 \times 10^3}{190 \times 10^3} + \frac{1.4 \times 10^6}{6.02 \times 10^6} = 0.2 \text{ MPa} < \varphi_m f_t = 0.24 \text{ MPa}$$

$$\sigma_2 = -\frac{P_f}{A_e} + \frac{M_f}{S_e} = -\frac{3.7 \times 10^3}{190 \times 10^3} + \frac{4.47 \times 10^6}{6.02 \times 10^6} = -0.76 \text{ MPa} \triangleleft \varphi_m f_t = 0.24 \text{ MPa} \ ***$$

As it can be seen factored tensile strength is less than flexural tensile stress. Hence, the CMU wall cannot satisfy the design requirements. This is because tensile stress usually has an important role in the unreinforced masonry wall, and the value of tensile stress in the conventional masonry unit is much less than in the AAC unit (i.e. 0.24 MPa compared with 0.59 MPa). Therefore, it was predictable that a conventional masonry unit cannot carry out the same loads as AAC could in the example shown in previous section.

The second example for comparing design of AAC and CMU is the example No. 2 of the previous section. Again, the parameters and the solution for the AAC case are as given in that section. For CMU, wall is 200mm thick with type S mortar $\left(f'_m = 10.0 \text{ MPa}\right)$.

Table 4.5 shows the values of axial loads and bending moments corresponding to load combinations according to CSA S304.1(4.6).

Table 4.5: load combinations for the second example- CMU case

Load Combination	P (kN/m)	M (kN.m/m)
1.25D+0.5L+1.4W	77.5	13.3
1.25D+1.5L+0.4W	107.5	5.9
0.9D+0.5L+1.4W	60.0	12.9

After trying some different reinforcement sections, an optimized reinforcement section will be found as shown in Fig. 4.4.

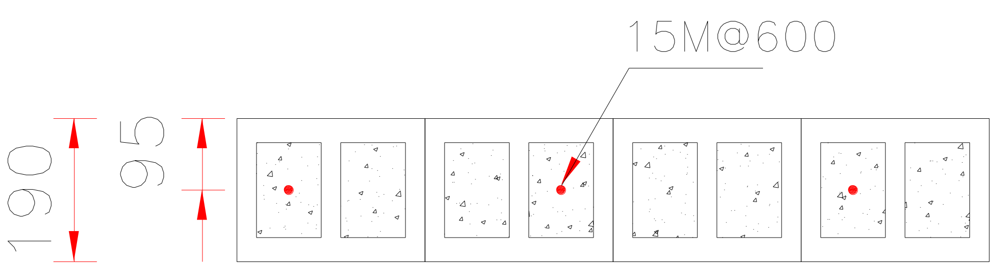

Fig.4.4 Reinforcement of CMU wall for the second example

Interaction diagram for CMU wall which reinforced with 15M@600 (As=333 mm²/m) is constructed according to CSA S304.1(4.6) and is shown in Fig. 4.5. As it can be seen in this figure, all points of load combinations are within the interaction diagram which shows the chosen reinforcement satisfied the design conditions.

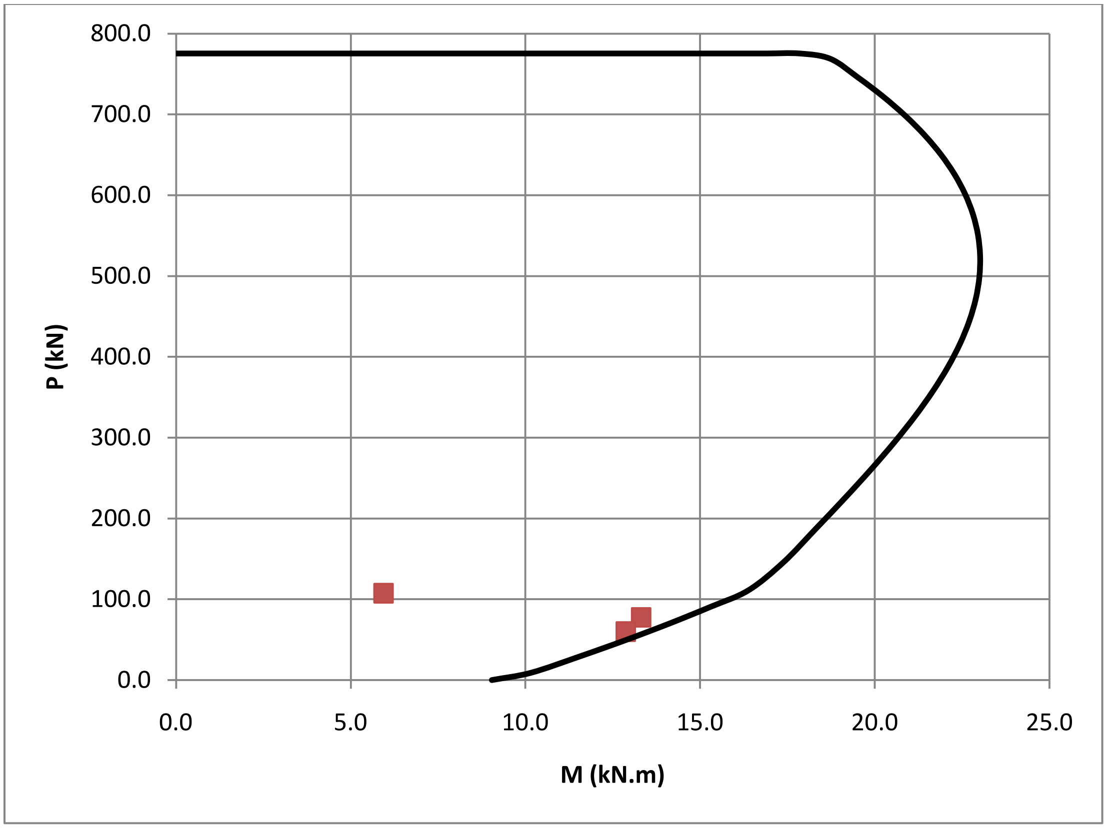

Fig.4.5 Interaction diagram for CMU wall with 15M@600 for the second example

The interaction diagram showed that by choosing CMU instead of AAC the wall reinforcement can be decreased significantly (i.e.67%).

In order to have a better comparison between reinforced AAC and CMU wall, the interaction diagram of the same reinforced section (20M@600) is shown in Fig.4.6. Comparing these interaction diagrams in Fig. 4.6 shows that reinforced CMU wall can carry out higher axial force and bending moment than reinforced AAC wall.

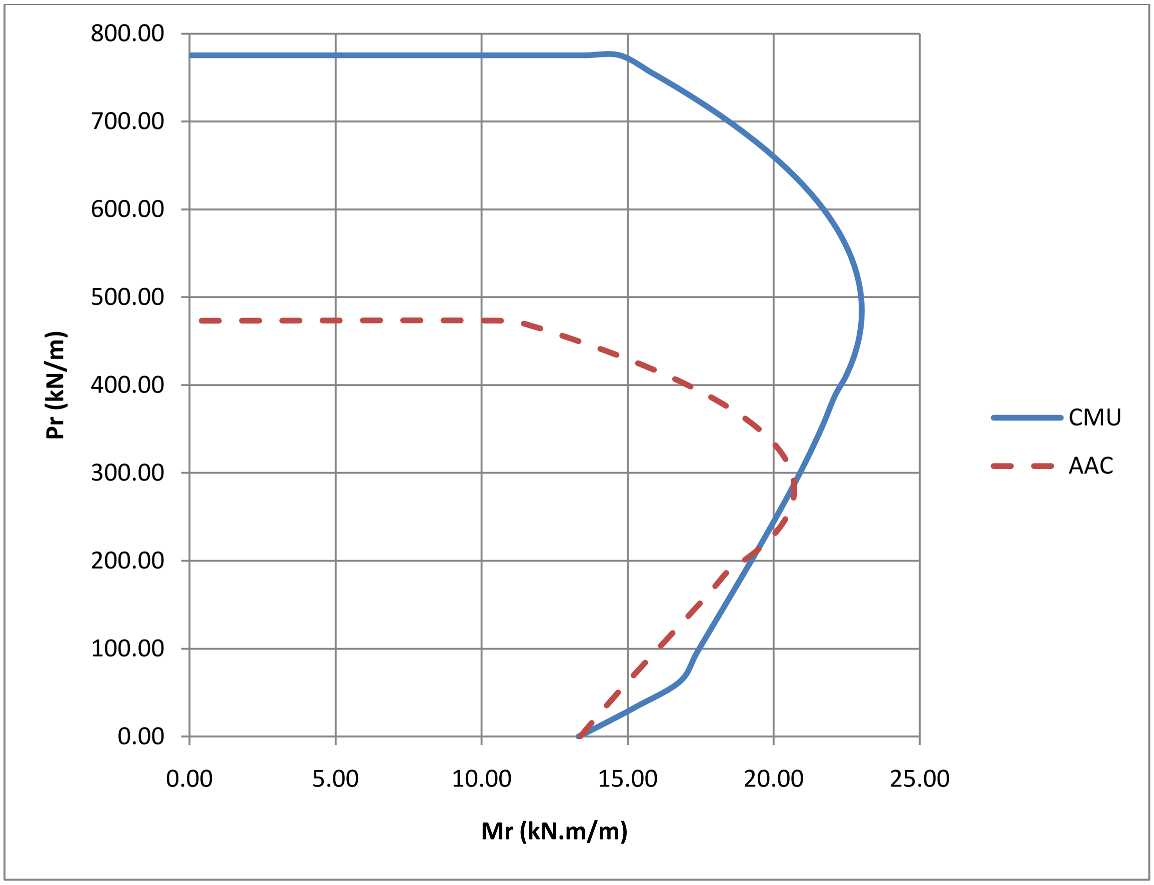

Fig.4.6 comparison of Interaction diagrams of CMU and AAC walls reinforced with 20M@600

Considering the two examples given in this section under unreinforced and reinforced conditions, it can be seen that:

1) In unreinforced walls higher allowable tensile strength in AAC will result to:
   - Reducing wall width
   - Increasing of bearing lateral loads (specially in non-load bearing wall)
2) In reinforced walls higher allowable compressive strength in CMU will result to:
   - Reducing reinforcement and wall width
   - Higher axial loads can be sustained

Section 5: Construction of AAC Structures

5.1 AAC Construction Products

Standard Block

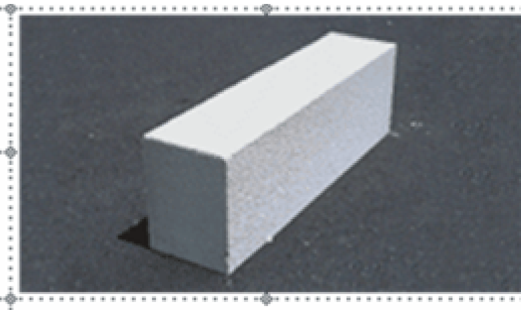

Fig. 5.1 AAC Standard Block [3.6]

DIMENSIONS	PROPERTIES
Length: 2 ft. Height: 8 in. Thicknesses: 4,5,6,7,8,10 and 12 in.	Class: AAC-2 ; AAC-4 Dry Density (Max) : 31 pcf ;44 pcf Compressive strength: 355 psi ; 710 psi (3.6)

AAC Standard Blocks are solid pieces used to build both load bearing and non-load bearing masonry walls. Their installation does not require specialized labour. This is a product that offers unique thermal insulation properties against cold and heat. They have up to 4 hours fire rating (direct exposure). These are available in various dimensions [3.1].

Semi-Jumbo Block

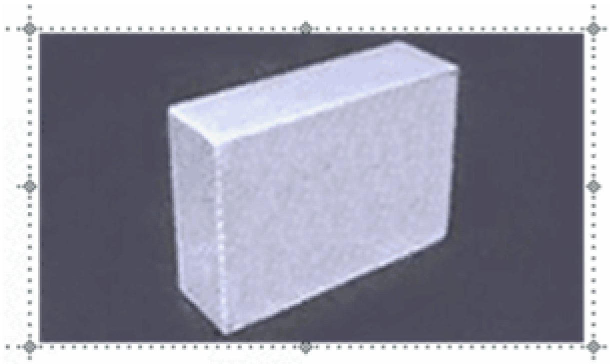

Fig. 5.2 AAC Semi Jumbo Block [3.6]

DIMENSIONS	PROPERTIES
Length: 2 ft. Height: 16 in. Thicknesses: 4, 5, 6, 7, 8, 10 and 12 in.	Class: AAC-2 ; AAC-4 Dry Density (Max): 31 pcf ; 44 pcf Compressive Strength: 355 psi ; 710 psi[3.6]

AAC Semi-Jumbo Blocks are solid pieces twice as high as Standard Blocks, used to build both load bearing and non-load bearing masonry walls. They require less consumption of thin-bed mortar (adhesive) [3.1].

Jumbo Block

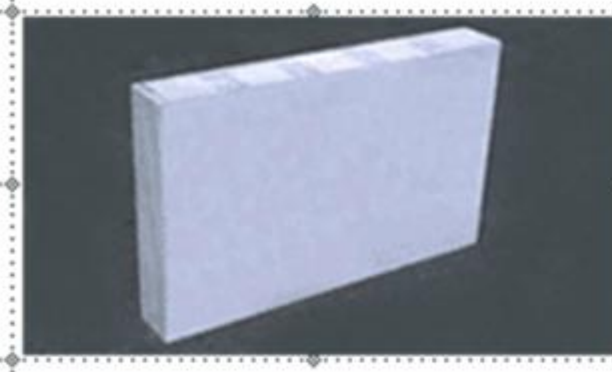

Fig. 5.3 AAC Jumbo Block (3.6)

DIMENSIONS	PROPERTIES
Length: 3.28 ft. Height: 2 ft. Thicknesses: 6, 7, 8, 10 and 12 in.	Class: AAC-2 ; AAC-4 Dry Density (Max): 31 pcf ;44 pcf Compressive strength: 355 psi ; 710 psi(3.6)

AAC Jumbo Blocks are unreinforced solid pieces for adjustments in doors, windows or ends in the Wall Panel System, or to build both load bearing and non-load bearing masonry walls. They have unique thermal insulation properties against cold and heat (3.1).

O – Block

Fig. 5.4 AAC O – Block (3.6)

DIMENSIONS	PROPERTIES
Length: 2 ft. Height: 8 in. Thicknesses: 6, 7, 8, 10 and 12 in.	Class: AAC-2 Density:* 31 pcf Compressive strength: 355 psi (3.6)

AAC O-Blocks are solid pieces with a hole on one side (3-9/16 core diameter) used for tie down installation in reinforced masonry. They avoid the use of wood-forms in confined masonry. Use of AAC O-Blocks reduces cost and increase cleanliness in construction [3.1].

U-Block

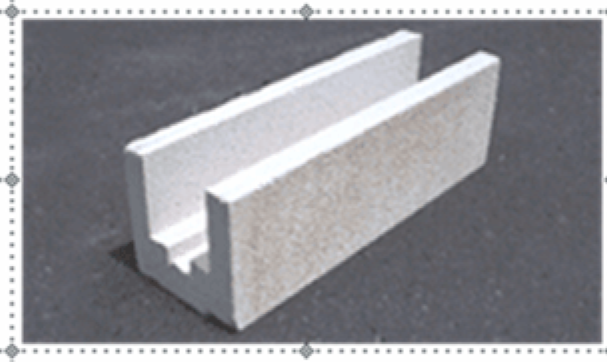

Fig. 5.5 AAC U – Block (3.6)

DIMENSIONS	PROPERTIES
Length: 2 ft. Height: 8 in. Thicknesses: 6, 7, 8, 10 and 12 in.	Class: AAC-2 Dry Density (Max): 31 pcf Compressive strength: 355 psi(3.6)

AAC U-Blocks have the same dimensions as Standard Blocks, but with a hollow center for reinforced concrete. These are used as bond-beams or short beams to cover spans of doors and window openings in load bearing and non-load bearing walls. They avoid the use of wood-forms in confined masonry. They do not require hardening waiting times therefore enable faster construction (3.1).

Reinforced Products:

Slab Panel

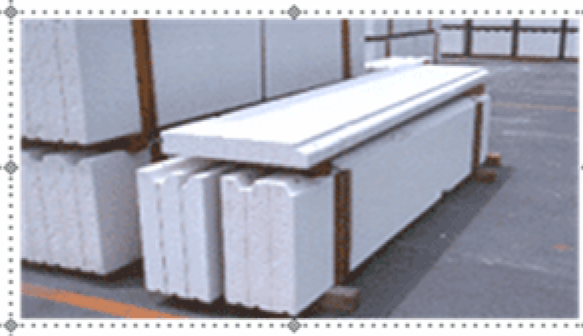

Fig. 5.6 AAC Slab Panel – Reinforced [3.6]

DIMENSIONS	PROPERTIES
Length: Up to 20 ft. Width: 2 ft. Thicknesses: 4, 5, 6, 7, 8, 10 and 12 in.	Class: AAC-3.3; AAC-4 Dry Density (Max): 37 pcf; 44 pcf Compressive strength: 497 psi ;710 psi [3.6]

AAC Slab Panels are reinforced units used to build roof and floor slabs that work simply supported by masonry walls, and also by steel, concrete or wood structures. Their design is based on span-load requirements. Their high thermal insulation capacity results in important energy savings. Fast installation, up to 2500 sq. ft. placed per day, without need of wood-forms or temporary supports. They have up to 4 hours fire endurance (direct exposure) [3.1].

Wall Panel

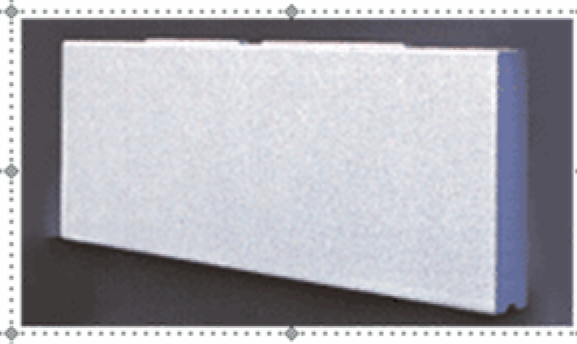

Fig. 5.7 AAC Wall Panel – Reinforced (3.6)

DIMENSIONS	PROPERTIES
Length: Up to 20 ft. Width: 2 ft. Thicknesses: 4, 5, 6, 7, 8, 10 and 12 in.	Class: AAC-3.3;AAC-4 Dry Density (Max): 37 pcf;44 pcf Compressive strength: 497 psi;710 psi(3.6)

AAC Wall Panels can be used as curtain walls in industrial installations, warehouses or commercial buildings. They work as simply-supported beams bearing over steel or concrete structures, and are designed based on span-load requirements. One of the main advantages of AAC Wall Panels is its thermal property that protects against the cold and heat, resulting in important savings in air conditioning equipment and energy consumption. They have up to 4 hours fire endurance (direct exposure) (3.1).

Lintel

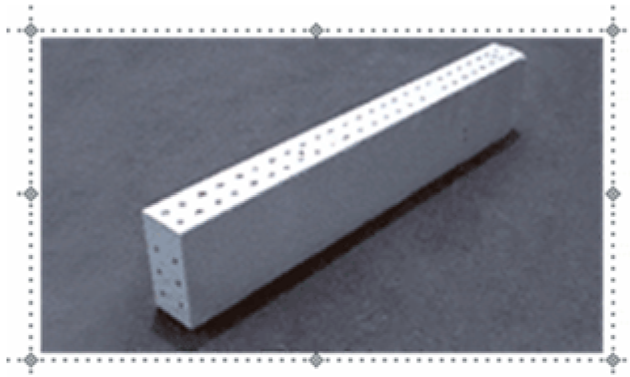

Fig. 5.8 AAC Lintel [3.6]

DIMENSIONS	PROPERTIES
Length: Up to 7 ft. Height: 8, 10 and 12 in. Thicknesses: 4, 5, 6, 7, 8, 10 and 12 in.	Class: AAC-4 Dry Density (Max): 37 pcf Compressive strength: 497 psi[3.6]

AAC Lintels are precast reinforced beams used to cover spans of door and window openings on both load and non-load bearing masonry walls. They are easy and fast to install. Thereby, avoids the use of wood forming and waiting times for hardening [3.1].

AAC Board

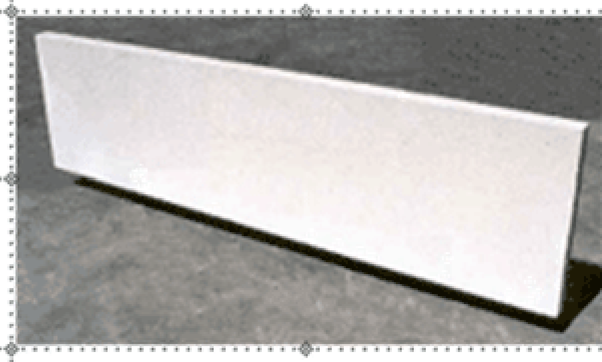

Fig. 5.9 AAC Board [3.6]

DIMENSIONS	PROPERTIES
Length: 8 ft. Width: 2 ft. Thicknesses: 2 and 3 in.	Class: AAC-3.3 Dry Density (Max): 37 pcf Compressive strength: 497 psi [3.6]

AAC Board Panels are reinforced units used as cladding over steel or wood frame construction, in exterior and interior walls. They are light weight, fire resistant, moisture resistant, easy and fast to install, and do not degrade with time. Different acrylic base-coats, stucco and texture can be applied [3.1].

AAC Fence

AAC Fence Panels are reinforced units with chamfer edges used as a prefabricated fencing system. They are light weight, fire resistant, moisture resistant, easy and fast to install [3.1].

Mouldings

The final details of a building are fundamental to its complete appearance. Hebel Mouldings provide excellent details with the appearance of traditional quarry style, enhancing the beauty of a construction project. Hebel Mouldings are light and easy to install, for use both in interior and exterior details, and with more than 25 models to choose from [3.1].

5.2 Construction process

Typical Domestic Construction

Fig. 5.10 AAC Block - Typical Construction Site (3.5)

AAC Block Construction process (3.4)

All structural design should be prepared by a competent person, and may require preparation and approval of a qualified engineer. Qualified professionals, architects and designers provide years of experience and access to intellectual property that has the potential to save house builders time and money as well as help ensure environmental performance. All masonry construction has to comply with the Building Code of Standards, e.g. all masonry walls are required to have movement/expansion joints at specified intervals. The standard block size is 200mm high by 600mm long. Block thickness can range from 50mm to 300mm but for residential construction the most common block widths used are 100mm, 150mm and 200mm. AAC blocks can be used in a similar manner to traditional masonry units like bricks and can be used as a veneer in timber frame and as one or both skins in cavity wall construction. The standard panel size is 600mm wide by 75mm thick with lengths ranging from 1200mm to 3000mm. AAC panels are typically used as a veneer cladding over a timber-framed construction. AAC manufacturers provide a wealth of detailed technical advice that, if followed, should help to ensure successful use of the product.

Movement joints (3.4)

Movement joints must be provided at 6m horizontal centres maximum (Continuously measured around rigid corners) Refer to manufacturer's guidelines for further information.

Footings (3.4)

AAC block construction requires level footings designed for full or articulated masonry in accordance with Code and Standards. Stiff footings are preferred because the wall structure of thin-bed AAC acts as if it were a continuous material and cracking tends not to follow the mortar beds and joints like it does in traditional masonry walling. Thick-bed mortar AAC walls do act more like traditional masonry but are not the preferred method for AAC.

Fig. 5.11 AAC Block Construction on Concrete Footing Base (3.1)

Frames (3.4)

Frames may be required for various structural reasons. Earthquake provisions tend to require multi-storey AAC structures to have a frame of steel or reinforcement to withstand potential earthquake loads that may induce strong, sharp horizontal forces. It is a relatively simple matter to build AAC block work around steel frames but embedding reinforcement rods can be costly and difficult.

Joints and Connections (3.4)

AAC manufacturers provide proprietary mortar mixes. Although more conventional thick-bed (10mm approx.) mortar can be used with AAC, the manufacturer's approved option is a proprietary 'thin-bed' mortar. Using thin-bed mortar, the procedure of laying the blocks is more like gluing than conventional brickwork construction. This is why many traditionally trained bricklayers may experience a need for a period of adjustment to a different method of working. In addition, brickies are used to lifting bricks with a single hand and AAC blocks often require two-handed manipulation. Although this may appear a slower construction process to lay masonry units, an AAC block is equivalent to five to six standard bricks.

Load bearing walls (3.4)

AAC is available in blocks of various sizes and in larger reinforced panels. These are sold as part of a complete building system that includes floor and roof panels in addition to interior and exterior walls.

Fixings (3.4)

AAC has low compression strength. The use of mechanical fasteners is not recommended, as repeated loading of the fastener can result in local crushing of the AAC and loosening of the fastener. There are proprietary fasteners that are specifically designed to accommodate the nature of the material by spreading the forces created by any given load, whether it is a beam, shelf or a picture hook. There are a number of proprietary fixings for AAC with extensive guidance available in product literature. In the event of uncertainty regarding the appropriateness of a fixing, consult the project engineer or fastener manufacturer for guidance.

Openings [3.4]

AAC is soft enough to be cut with hand tools. Niches can be carved into thicker walls and corners can be chamfered or curved for visual effect. Channels for pipes and wires are easily made with an electric router but with all carving and cutting care must be taken to use appropriate dust reduction strategies and appropriate personal protection equipment should be worn at all times.

Finishes [3.4]

AAC block work and panels can accept cement render, but the manufacturers recommend using a proprietary render mix compatible with the AAC material substrate. Site mixed cement renders have to be compatible with the AAC substrate, with the render having a lower strength than conventional renders.

All renders should be vapour permeable (but water-resistant) to achieve a healthy breathable construction. All external coating finishes should provide good UV resistance, be vapour permeable and be proven suitable for AAC. Consult the manufacturer's literature for further information on coatings.

5.3 Case Study and Cost Comparison

Case Study

East Hall, University of Indianapolis' (UIndy) new residence hall is the second in order in the university campus, after the Central Hall, where AAC been used as the main building unit. This four-storey structure with 154 single-occupant rooms, multiple lounges and two-storey atriums with balconies is constructed using AERCON's Autoclaved Areated Concrete (AAC) - a material produced in blocks,

lintels and panels. Central Hall of University of Indianapolis' is one of the first academic buildings in the US to use the AAC. Construction of the East Hall required 800 wall panels per floor and almost 500 floor panels per floor. The wall panels are 12 feet high and vary between 12 and 24 inches wide. The floor panels are about 2 feet wide and up to 19 feet 6 inches long. View 1 to View 3 shows some stages of the construction of the East Hall[3.7].

View 1: construction of AAC wall

View 2: prefab AAC units

View 3: painted exterior AAC wall

Cost Comparison

In 1980's, Siporex, a Swedish company manufacturing ACC, conducted two comparative building costs studies in Florida. The first study compares the cost of residential, office, warehouse and commercial buildings constructed with Siporex ACC and similar buildings built with commonly used construction materials. The second study compares precast / prestressed concrete and steel frame with Siporex material.

The first study indicates that, based on the cost per square foot of a wall surface, the cost of a traditional wall in a single-family / multi-family house was about $3.92. The fire resistance of the wall was two hours and the calculated R value was 5.5. In comparison, the cost per square foot for an eight inch thick Siporex panel was $3.48, having an R value of 9.1, without any added insulation, and a fire resistance of four hours. For cost comparison of roofs, traditional insulated steel roof on bar joist was estimated at $2.16 per square foot providing an R value of 4.0, and without fireproofing, the roof would offer no fire resistance. On the other hand, the eight inch Siporex roof panel would cost $2.98 per square foot and offer an R value of 10.0 and fire resistance of two hours.

The second study determined the total cost of the envelope of a light industrial building, including columns and beams and the enclosing shell. Equal spans of 70 feet were assumed between primary beams in the three alternatives selected – steel, precast concrete and Siporex. The estimated cost per square foot area was for steel framed structure $7.06, for precast concrete structure $8.58 and for Siporex structure $7.70. The insulation value provided by each of the ystems was similar, but the Siporex system provided higher fire resistance and savings on insurance costs. The study also indicated that where sprinkler systems are mandatory, savings can be realized in the size and type of sprinklers used[3.9].

The vice president – general manager of AERCON Florida LLC, mentioned that AAC is ideal for cost-conscious clients because of the reduced time and labor costs associated with installing the product. For AAC, no wall insulation is needed, and because the blocks are lighter the delivery costs are dramatically lower[3.7].

Section 6: Suitability of AAC for Canadian Environment and Market

6.1 Use of AAC in Canada

AAC is used for construction on every continent (more than 200 plants in 35 countries including Mexico), its use in United States and Canada has been rather limited. According to Professor Michael W. Grutzeck, the potential homeowners in U.S. and Canada make choices based on cost rather than longevity and durability. North America has always had abundant supply of wood.

AAC has an impressive use history in Europe and Japan; it is still a new construction material for those who live in North America.

At this point in time, building a home out of AAC house would cost more than a similarly sized wood framed house, because the AAC is not available locally, but on the other hand, an AAC house would be allegern free, maintenance free, water proof and last considerably longer[3.3]. At the same time there is only limited experience available on the use of AAC constantly below freezing point and at extremely low temperatures, below -50 °C. However, no *unfavourable* reports from the existing applications have been received [3.2].

Therefore, AAC can be recommended as a construction material in Canada. If AAC manufacturing plants are encouraged in Canada, the initial and maintenance costs will be less and at the same there might be saving in the cost of heat and power; bills will be less and resources will be conserved.

6.2 Concluding Remarks

In the US, AAC building construction costs approximately the same as the timber-framed construction when the AAC manufacturing plant is near by. Though the light weight of AAC reduces the shipping cost compared to other building materials, the use of AAC becomes less cost effective when the

construction site is far from the AAC manufacturing plant. Since no manufacturing plant of AAC is presently in operation in Canada, the initial cost of construction expected to be higher compared to other types of building construction. However, this high initial cost could be balanced through the savings due to the lower maintenance cost, durability of the structure, lower initial cost for the heating and cooling systems, and through possible reduction of insurance costs.

As tensile strength of AAC is higher than CMU, for non-load bearing walls with out-of-plane loading where usually the tensile strength governs the design, AAC can be a better variant. Also, due to limited compressive strength of AAC, it is useful for cladding and, for structural elements with low axial force such as low rise structures.

Existing U.S. codes and specifications could provide a guideline for the use of AAC in Canada but there are still a lot of experiments and research works required considering the local Canadian conditions. This is necessary for the proper development of codes and specifications for the production and use of AAC in Canada. AAC is being used in other cold countries like Scandinavian countries and Russia, so there's a good chance that AAC will be a perfectly suitable for the Canadian environment.

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ABBREVIATIONS & NOTATIONS

AAC	Autoclaved Aerated Concrete
ACC	Autoclaved Cellular Concrete
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASHRAE	American Society of Heating, Refrigeration and Air-Conditioning Engineers
ASTM	American Society for Testing and Materials
CEB	Euro-International Committee for Concrete or Comité Euro-International du Béton
CMU	concrete masonry unit
CSA	Canadian Standards Association
MSJC	Masonry standards Joint Committee
PAAC	Precast Autoclaved Aerated Concrete
p^H	potentiometric hydrogen ion concentration
TMS	The Masonry Society
UL	Underwriter's Laboratories®
A_n	net cross-sectional area of masonry
A_S	effective cross-sectional area of reinforcement
b	width of section
d	distance from extreme compression fiber to centroid of tension reinforcement
D_v	actual depth of masonry in direction of shear considered
E_s	modulus of elasticity of steel reinfocrement
f'_{AAC}	compressive strength of AAC
f_y	specified yield strength of steel for reinforcement
h	effective height of column, wall or pilaster
l_w	length of entire wall or of the segment of wall considered in the direction of shear force

M_u	factored moment
P_n	nominal axial strength
P_u	factored axial force
r	radius of gyration
T	nominal thickness of wall, or overall depth of member cross-section
V_n	Nominal shear strength
V_u	factored shear force
ε_{mu}	maximum masonry compressive strain
Φ	resistance factor
μ_{AAC}	coefficient of friction of AAC
D/ L/ L_r/ S/ R/ W/ E	Dead load/ Live load/ Roof live load / Snow load/ Rain load/ wind load/ Earthquake load

Prefabricated Masonry

L. Moulson, F. Habibza and S. Weinbee

Department of Civil and Environmental Engineering, University of Alberta

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ABSTRACT

For more than 100 years, prefabricated masonry has been used as a construction material all over the world. By the 1970's, engineers and researchers had a vision for prefabricated masonry to change the construction industry to an industry much like the precast concrete industry. This technical report gives a current view on the state of the industry and illustrates whether this goal was ever met.

The main topics of this report are application, manufacturing and design of prefabricated masonry. Each of these are discussed in detail with either case studies or examples. The first part of the report is designed to give the reader a background of the building material, its history, as well as, the advantages and disadvantages of its use. Then the manufacturing methods are described leading into the design of prefabricated masonry. A comprehensive design example is included in this paper; a valuable section for practicing engineers. The construction and installation of panels is discussed, including sequential images of the installation of a prefabricated fence, joints between panels and connections to building envelopes. Finally, the economic viability is discussed, giving insight on the wellbeing of the industry today.

INTRODUCTION

The need to minimize on site labour and reduce construction time has lead to the development of prefabricated masonry panels to be used as building components. Today it is used in various types of construction, including commercial, residential and institutional. Prefabricated masonry panels include shop fabricated assemblies made from concrete units, masonry, brick or clay tile. Panels are constructed away from the building and later assembled in place to increase time, efficiency, and improve overall quality of construction.

There are two types of manufacturing; mass production and architectural appeal, which provide different purpose and focus on either quantity or quality. There are several codes with covering various fundamentals that govern the design of prefabricated masonry panels in Canada along with papers recommending other design considerations. There are also considerations involved with the assembly of panels on site including material handling considerations, methods of installation and connections between panels and to the building envelope. The declined use of the product has invoked an economic feasibility study and analysis of the product in the industry.

History and Development

The use of some individual prefabricated masonry dates back more than 100 years, but was not widely used or developed. It wasn't until the 1950's after WWII where a shortage of housing caused prefabricated masonry panels to be used more broadly in Denmark, France and Switzerland. The use of prefabricated masonry panels started in North America in the 1960's and in more than 30 countries by the 1970's (Pasha, 1977). With the shortage of housing after the war, and an increased desire for a higher standard of living, masonry construction was in high demand. After the end of WWII there was a large shortage of skilled labour making it difficult for masonry contractors to meet demand. Therefore, it was thought that the construction of masonry panels off site could be performed by less skilled labour, which would expand the employment base. The brick laying process was mechanized by producing standard panels, however, this ended up being less productive and lead to an increased cost over conventional laid in place masonry construction. Later there was a movement towards using more skilled labour and other conventional masonry practices to increase the productivity (Pasha, 1977), reduce prefabrication costs and to make it more competitive. In the 1960's and 1970's there was intervention from the government and other organizations in the prefabrication industry to ensure stability of demand and to support research programs for further development of the product. There were efforts made to standardize the design, production, distribution and installation of prefabricated panels. These efforts contributed to improving in the quality of masonry units and mortars being used, and created a wide range of different types of systems and panels. Although great efforts were made to advance the prefabricated masonry industry, it has died off within the last 5-10 years due to economical reasons discussed later in this paper.

Advantages

The main motivations in the formulation of prefabricated masonry were skilled labour shortages and high demand for masonry construction. However, there are many more advantages than just these two. Prefabricated masonry reduces the need for skilled labour through the use of standardized machines which enable mass production that results in a larger economy of scale and a reduction in cost per panel (Chaya, 1979). With panels being manufactured in a plant, the work on the panel can be performed on several shifts to meet higher rates of demand if necessary.

The industrialization reduces variability in workmanship resulting in a more consistent quality of panel (Chaya, 1979). The consistent finished quality enables designers to focus on creating overall quality improvements to the panels. Plant managers can concentrate on improving manufacturing efficiency, mortar batching systems and curing conditions such as temperature and humidity which can be more tightly controlled. Through this improved quality control, high quality prefabricated panels can be produced. The industrialization of the manufacturing process also makes more efficient use of resources which reduces material waste and costs while increasing the demand of prefabricated panels.

There are also many benefits with panels being prefabricated off site. First, if the panels are fabricated indoors, it reduces weather delays. Panels can actually be manufactured year round without having to winterize the structure; thereby reducing heating and hording costs. Construction time can also be reduced if panels are prefabricated off site because other trades do not have to wait or work around the brick layers. With proper scheduling, the panels can be brought on site and installed without any delays to the construction schedule. A reduction in construction time can be extremely advantageous and is beneficial financially (Chaya, 1979). With panels not being delivered to the site until required, they do not need to be stored on site resulting in a less congested jobsite. Prefabricated panels can be installed using cranes eliminating the need for on-site scaffolding which can be costly. Panelization can also make the construction of irregular shapes and sizes easier. Forms or templates are created to achieve this and with repetitive use can lower cost.

Prefabricated masonry is more advantageous over conventional in-place masonry construction in seismic areas. The reason for this is the energy from the earthquake is absorbed through the connections and joints between panels as opposed to the mortar joints in conventional masonry. Therefore, seismic forces cause less damage to prefabricated structures (Holzkaemper, 2010).

Disadvantages

Although prefabricated masonry can be advantageous, there are many reasons why it should not be used for construction. It has not yet achieved the economy of construction and typically costs 1.5 times more than conventional laid in place masonry construction (see case study) due to transportation and handling costs. If not used in large scale the cost of production can be too high over conventional masonry construction. The use of prefabrication can also be eliminated because of availability and/or logistics. The location of manufacturing plants is a problem. Most plants are located far from sites resulting in high transportation costs. The size and type of prefabrication is limited by transportation and erection requirements as well as the architectural layout (Chaya, 1979). With prefabricated panels the connection detail requires more attention because they support the entire load and are prone to corrosion. One chief disadvantage of prefabricated masonry construction is that other trades' construction tolerances must be more stringent because with prefabricated panels adjustments can only be made at connections and joints between panels opposed to adjusting mortar thickness between individual bricks.

Applications

Various types of prefabricated masonry

There are different applications of prefabricated masonry panels including walls, fences and fireplaces. Structural brick veneers are the only current form of prefabricated masonry wall panels because they can only bear self weight and wind load due to economic feasibly.

Prefabricated masonry can be constructed off site or on site. Reasons for choosing one alternative over the other are site location, transportation costs, weather conditions, type of panels, and construction scheduling. The decision should be made based on each specific project and in the early stages of project planning.

Panels can be either reinforced or non-reinforced. Prestressed and post tensioned prefabricated masonry panels also exist, but are costly and rarely used.

Case Study

Prefabricated masonry is most commonly used when replacing the exterior façade of existing operational buildings. This was the case for an office building in Sudbury, Ontario. The existing exterior was deteriorated and the building's R-value was below standard. The prefabricated panels were backed with insulation enabling the walls' R-value to be increased. Since the building was required to remain open throughout the construction process the quick installation time provided minimal disturbance to occupants (Holzkaemper, 2010).

Figure 1: Sudbury Office Building (www.Pan-Brick.com)

A cost analysis comparing the use of prefabricated panels to laid in place for this project is provided in table 1. Although t.hey cost 1.55 times more, the prefabricated panels were chosen

because the construction process wouldn't disrupt the occupancy of the building.

Table 1: Cost Comparison Analysis (Adapted from Holzkaemper, 2010)

	Material Cost ($/m^2)	Construction Time (m^2/day)	Labour Cost ($/m^2)	Final Installed Cost
Laid in Place	6.7	225	55	$345,000
Prefabricated	13.4	19000	45	$535,000

Durability

Typically prefabricated masonry panels last longer than conventional masonry construction because they are more durable. However, the exact length of the life cycle depends greatly on the type of panel being used. The reason prefabricated masonry is more durable and longer lasting than conventional lay in place masonry is because more time and effort is put into design and there is increased quality control. Panelized construction results in fewer joints, which means there are fewer locations where water can penetrate the masonry. Therefore, there is a reduction in deterioration of building envelop due to water affects.

The durability of prefabricated masonry panels also varies quite widely based on application, location, manufacturer and materials of the unit. Panels can be manufactured for specific site conditions to increase their durability. Site specific manufacturing is an added expense to the project but can vastly increase the life span of the panels. In determining whether to design for site specific conditions the owner should do a financial analysis comparing initial design and construction costs to building life span and saved maintenance costs.

Manufacturing

The way prefabricated masonry is different from conventional types of masonry is that it is prefabricated away from the building face. This can be on the construction site which is not common, or at a manufacturing plant.

In a broad sense there are two types of manufacturing techniques. Each technique focuses on a different market. The first technique focuses on producing an end product that is aesthetically appealing. The second manufacturing technique focuses more on quantity of panel produced rather than quality.

Architectural Appeal

Entrepreneur William VetoVitz started the company Vet-O-Vitz Panel Systems based out of Ohio, which was a large company specializing in manufacturing prefabricated panels. The company specialized in complex shapes for architectural appeal. With complex panel shapes automation is limited, and the main automated piece of equipment is the scaffolding. This saves the workers time and reduces health issues related to strained muscles. Figure 2 shows the automated scaffolding. (VetoVitz, 2010).

Figure 2: Automated Scaffolding at the Vet-O-Vitz Manufacturing Plant (BIA, 2001)

For economic reasons it is important to use high early strength mortar. This allows for the prefabricated masonry to be moved the next morning by crane or fork lift into storage and away from the automated scaffolding. Manufacturing plants also had large turning tables for moving the panels and putting them on large flat bed trucks (VetoVitz, 2010). Figure 3 shows an architectural panel of an interesting shape that was created for the Columbia Center Tower in Troy, Michigan. This was a very impressive project where 700 panels were erected in 68 days.

Figure 3: Prefabricated Panels and Full Scale Illustration of the Columbia Center Tower (Building the Future, 2003)

The next figure shows the complex shape that can be formed by prefabricated masonry. This is an arch with brick orientated at a 90 degree angle.

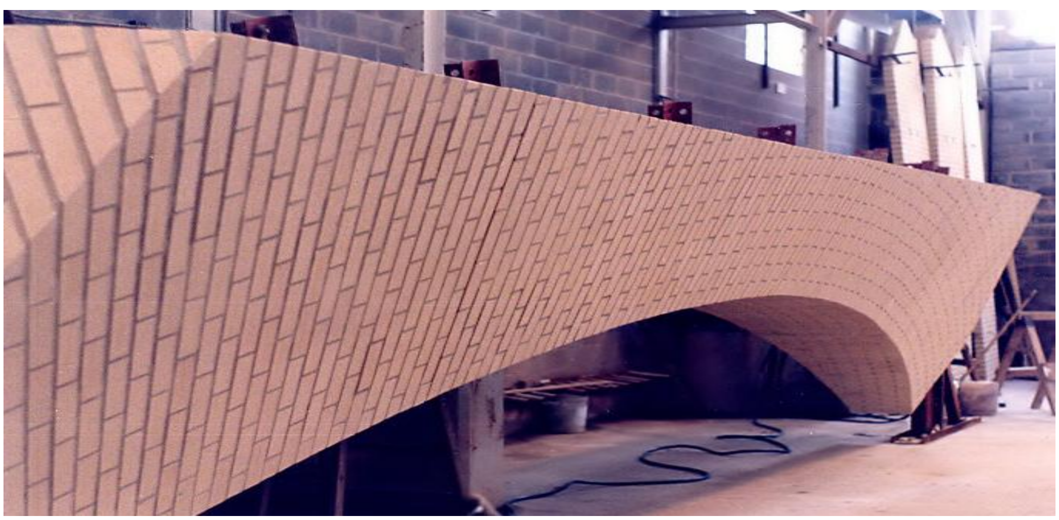

Figure4: Prefabricated Panel with a Complex Shape (Building the Future, 2003)

As illustrated in the figures above, very interesting projects can be built with prefabricated masonry and it is a building material very attractive to owners and architects. The downfall to this type of construction is that it is slow and requires skilled masons because laying the bricks is done by hand which is time consuming. Some companies have tried to automate as much of the process as possible, however in automation, there is a limit to the complexity of the shape that can be produced.

Mass Production

With mass production the panels are usually of regular shape and straightforward to build. In order to speed up manufacturing many manufacturers take advantage of automation. An inventor by the name of Harry J. Brandt filed a patent on a manufacturing system for prefabricated panels in 1972. In this system a panel is created without any skilled labour through the use of a forming box. First, brick is arranged in a pattern created by grooves in the bottom of the forming box. The mortar is than introduced through the creation of a vacuum resulting in a negative pressure facilitating the flow of mortar through the box and into void space between bricks. A positive pressure is than applied to the box filling any remaining voids with mortar. Once it has cured and reached sufficient strength, the panel is taken out of the forming box. Figure 5 depicts a typical forming box (Brandt, 1972).

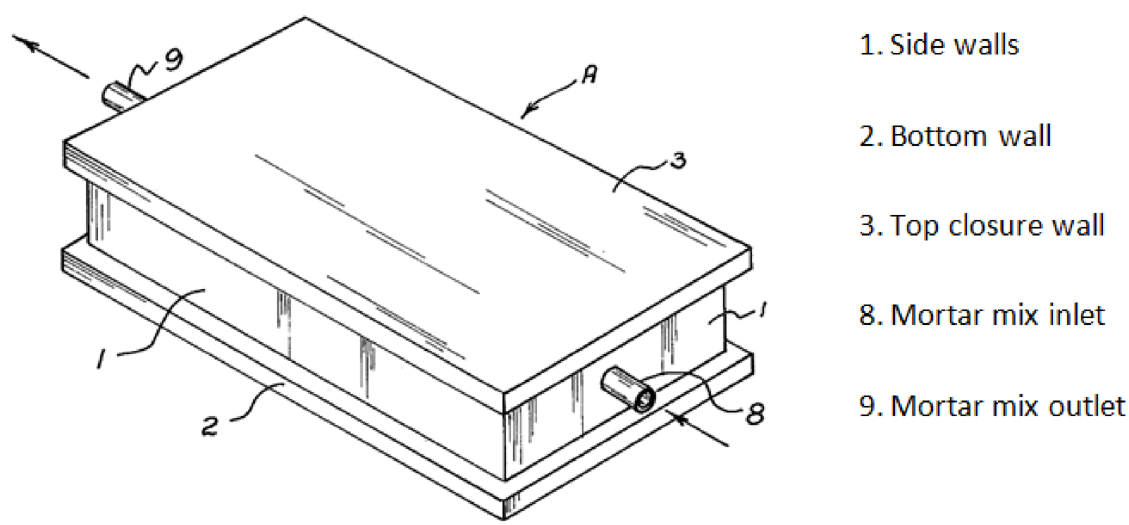

Figure 5: Forming Box for a Prefabricated Masonry Wall (Brandt, 1972)

Figure 6 illustrates the grooves on the bottom of the box to ensure proper brick spacing.

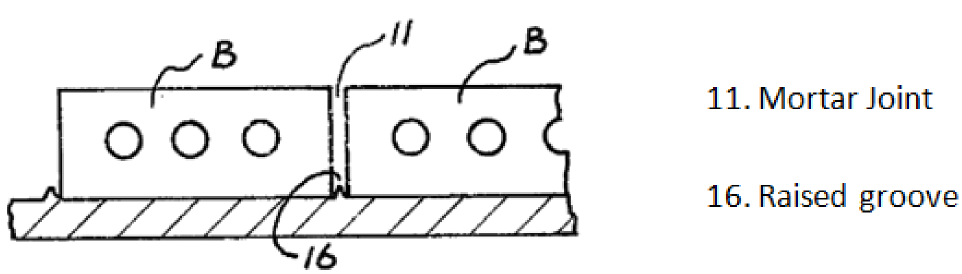

Figure 613: Illustration of Raised Grooves to Align Brick Inside Forming Box (Brandt, 1972)

Presently, research is working on developing new ways to automate this procedure and one day make it fully automated. The research done up to date is on stacking and storing of bricks. This is not for building the prefabricated masonry walls, but for moving the bricks to facilitate the building process. The machine currently being developed is designed to mimic human movement. Essentially, a brick robotic arm is used to lift and move bricks. In order for the robotic arm to move the brick, it needs to grip the brick, but because bricks are stacked close together the only way to grip the brick is through the cores. This requires state of the art sensing technology. As the robotic arm moves close to the brick, it finds a core in the brick and inserts a pneumatic brick gripper into the core. The robotic arm is illustrated in Figure 7 (Rihani, 2006).

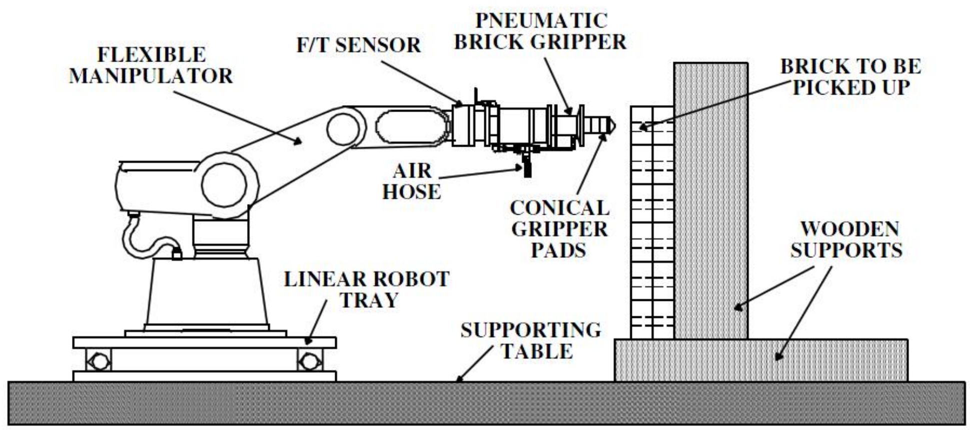

Figure 7: Robotic Masonry Arm Used to Stack and Move Brick (Rihani, 2006)

The robotic arm is a very valuable concept but is still a new technology that requires more research before it can be implemented to build a prefabricated masonry element.

Design

Standards and Specifications

Prefabricated panels must be stronger than the traditional laid in place masonry because panels must be strong enough to sustain the loads acting on it during transportation, storage and placing. It is for this reason that standards have been implemented to make sure these requirements are met. The three documents that govern the design of prefabricated masonry panels are: CSA S304.1 - Design of Masonry Structures, ASTM C901 – Standards and Specifications for Prefabricated Masonry and ACI 530.1 - Specifications for Masonry Structures. ACI 530.1 only directs designers to ASTM C901 hardly qualifying it as an actual design standard for prefabricated masonry. CSA S304.1 provides the designer with some information on prefabricated panels but is not nearly as comprehensive or informative as ASTM C901.

CSA S304.1 - Design of Masonry Structures

CSA S304.1 requires that in addition to meeting the requirement of a traditional masonry structure, prefabricated masonry structural components must resist the additional loads during transportation, storage and lifting. It also requires the engineer to design the "joints and bearings for dimensional changes that could occur due to shrinkage, elastic deformation, creep and temperature" (CSA S304.1, 2004).

ASTM C901 – Standards and Specifications for Prefabricated Masonry

C901 has the following categories specifying the requirements for all prefabricated masonry panels:

Materials and Manufacturing
Structural Design
Dimensions and Permissible Variations
Workmanship Finish and Appearance
Quality Control
Shop drawings
Handling, Storage, and Transportation

Materials and Manufacturing Design

The individual modules (bricks) of prefabricated masonry panels must meet the local building code specifications, and satisfy the ASTM C901 requirements. ASTM C901 requires that all reinforcing material used in a panel be coated to avoid corrosion as well as all other corrosive materials, such as ties and anchors. In addition, "the mortars and the grout used in constructing the masonry panel must meet the specification provided in ASTM C 270 and ASTM C 476 respectively" (ASTM C901).

Structural Design

When designing prefabricated masonry panels, all the loads that act on the panel from the time of casting to transportation and lifting must be considered, including in situ service loads. The design also has to satisfy the requirements of regional building codes. ASTM requires that differential movement between dissimilar materials must be considered. The lifting capacity of the machine used in erecting the prefabricated masonry panels must be four times the dead weight of the panel.

Dimensions and Permissible Variations

The size of panel depends on the standard nominal size of the modules being used in the manufacturing of the masonry panel. ASTM specifies that the nominal dimension can be larger than the specified dimension by the thickness of one mortar or less then 13mm. Individual modules used to build prefabricated panels can vary in shape and size as shown in Figure 8.

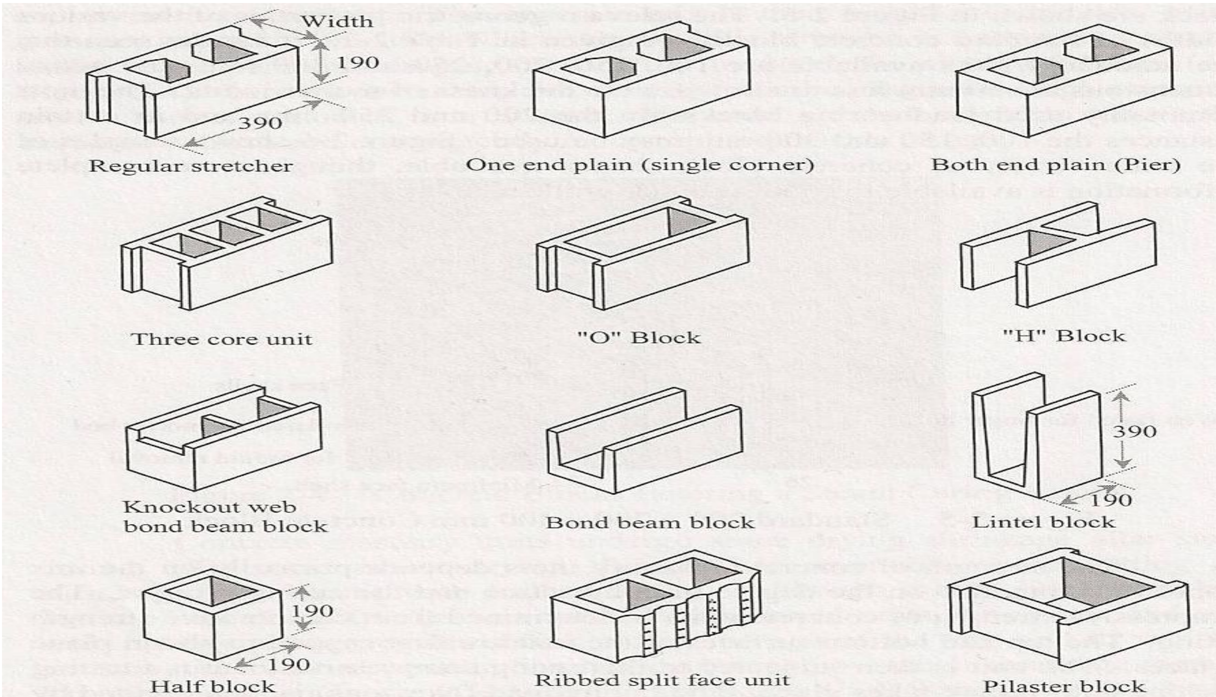

Figure8: Examples of Concrete Modules (Korany & Hatzinikolas, 2005)

In addition, ASTM limits the maximum variance of the masonry dimensions from specified values. For instance, the maximum permissible variation cannot be greater than 6.4mm.

Workmanship Finish and Appearance

In order to ensure that the panels have been installed within the acceptable tolerances, ASTM requires that a sample panel be provided for comparison. In addition, the location of all anchors, inserts and fittings cannot vary from specified shop drawings by more than 9.5mm. The out of plane warpage is limited to 3mm for every 1.8m of panel height or width.

Quality Control

To insure the consistency of the final product, ASTM requires that at least ten samples of masonry units out of a batch of 50,000 should be tested to verify the compressive strength and initial absorption rate (IRA). For every 465m^2 of panel assemblage or one storey height, one sample panel must be tested for compressive strength and for each day's work one panel must be tested for flexural strength.

Shop Drawings

A smooth transition from fabrication of panels to their final intended function requires good fabrication and placement drawings. ASTM C901 requires that the fabrication drawings must consist of detailed drawings which indicate the location of all reinforcements, inserts, anchors, bearing seats, lifting inserts, coursing, size and shape of openings, and the panel size and configuration. It also requires placement drawings which "show the panel identification and location, reference dimension, panel dimensions, dimension of joints between panels and connection details" (ASTM C901). In addition, it requires that every panel must be marked appropriately to avoid any confusion and misplacement of panels.

Transportation and Handling Design

The primary mode used to transport prefabricated masonry panels to site is trucking. The panels must have enough resistance to withstand the loading it endures during transportation. In addition, it is essential that the transportation of panels to site meet the local traffic regulations and bylaws.

Anchors and reinforcement have to be designed to resist the loads imposed by erecting, lifting and installing panels into place. Figures 9 and 10 show lifting techniques adopted from the erection of precise concrete panels that could be used to erect and install prefabricated masonry panels.

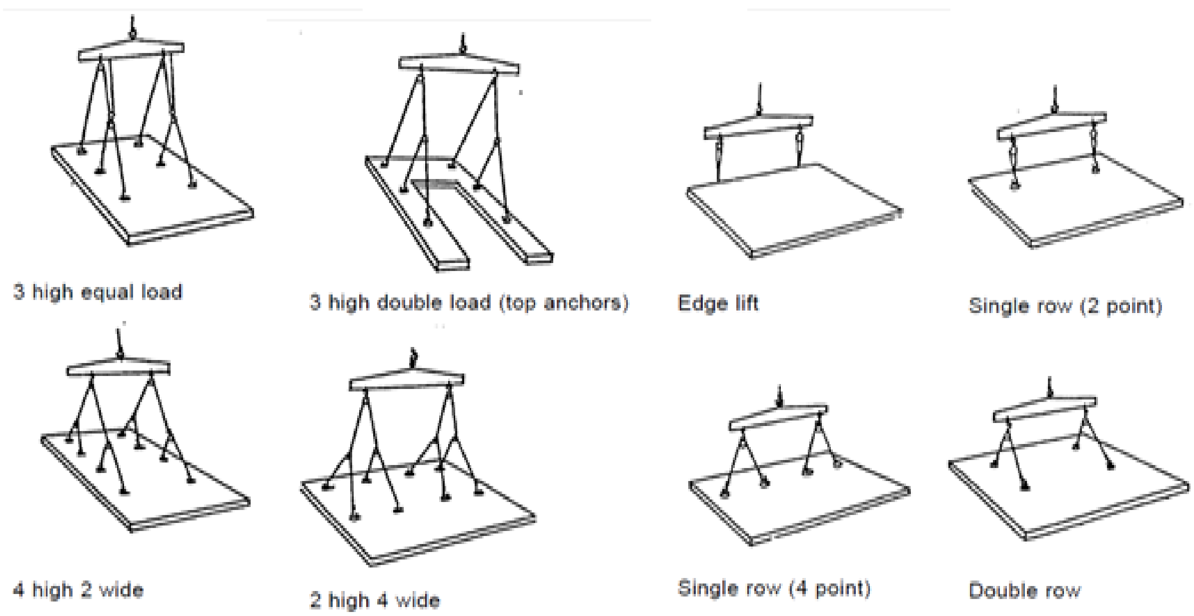

Figure 9: Common Methods of Lifting Prefabricated Panels (OSHS,2002)

In order to emphasize the strength requirements of prefabricated masonry panels, a design example indicating the transportation and handling loads to be considered during the design is given further in this paper.

Additional Design Considerations

The design of prefabricated masonry structures depend on its intended purpose. The size, reinforcement and dimension of prefabricated panels used in construction and assembly of the final masonry structure depend on the load combination it has to resist during transportation and handling and during its service life. The loads that must be considered in design of prefabricated masonry structure are:

Dead Loads
Live Loads
Transportation and Handling Loads
Storage Loads
Wind Loads
Rain and Snow Loads
Seismic Loads
Hydrostatic Loads

Masonry panels must be designed to resist the most unfavorable load combination and placement. Two methods of design have been specified in the National Building Code of Canada (NBC), namely Working Stress and Strength Design and Limit State Design. Both of these methods, although fundamentally different, require that the critical load combination must not exceed the factored resistance of the structure. The following loading combinations have been specified by CSA S304.1 for determining design loads.

$$U=1.4D \quad \text{..........} \quad (1)$$

$$U=1.25D+1.5L+ (0.5S \text{ or } 0.4W) \quad \text{..........} \quad (2)$$

$$U=1.25D+1.5S+ (0.5L \text{ or } 0.4W \quad \text{..........} \quad (3)$$

$$U=1.25D+1.4W+ (0.5L \text{ or } 0.5S) \quad \text{..........} \quad (4)$$

$$U=1.00D+1.0E+ (0.5L +0.25S) \quad \text{..........} \quad (5)$$

It is recommended by the authors that transportation and handling load must also be considered in determining the critical design load combination for prefabricated masonry. In addition, since the panels are fabricated off site, an impact load factor of 2.0 for straight panels and 1.25 for a 15 degree tilted panel is also recommended by Hatzinikolas & Pacholok.

Design Example – Element of a Sound Barrier Wall

Design of 2400mm height, 2700mm wide, 100mm thick concrete block wall. The panels are to be designed for service load and for the loads it incurs during handling and storage. Furthermore, the panels must incorporate sufficient reinforcement that would resist the vertical loads acting on the panel.

Prior to beginning design the following assumptions are made:

- Normal Weight Density $\rho = 2100$ kg/m^3
- the panel is fully grouted and constructed with type S-Mortar
- Compressive strength normal to bed joint $f'm$ = 9.8 MPa
- Yield strength of Steel Reinforcement fy = 400 MPa

- Modulus of elasticity for concrete & steel are Em = 8500 $f'm$ and Es= 200000 MPA respectively

- For primary wind load design the location is assumed to be Edmonton, Alberta

While designing the prefabricated panel for handling and installation load, it is recommended by the authors to take into consideration the construction procedures followed by the manufactures in fabricating the panels. For instance, some manufactures fabricate the panels in bottom up construction with main reinforcement installed vertically and grouted while others may follow a different method of construction. In either case, the panels must be designed to allow for rotation of the panels to preferred orientation. In this example, it is assumed that the construction is from bottom up and in plane lifting reinforcement is provided in both directions to allow for lifting and rotating the prefabricated panels and the main reinforcing bars are horizontal to resist the flexural wind load. The final design concept is illustrated at the end of the example in Figure 12.

1) ***Lateral Wind Load***

Wind load depends on location and height of the building in addition to wind intensity. The intensity of the wind load recorded in NBC Table C-2 for Edmonton is 0.45 kPa. This pressure will be used to analyze the prefabricated masonry panel.

$P = I_w \, q \, C_e \, C_g \, C_p$ $\qquad\qquad$ $C_e = \left(\frac{h}{20}\right)^{0.2} = 0.65$

$I_w = 0.8$ (the sound barrier wall considered is of low importance)

$C_g = 2$ $\qquad$ $C_p = 0.8$ $\qquad$ $C_e = 0.65$

P= (0.8) (0.45) (0.65) (2) (0.8) = 0.38 kPa (Pressure on windward face)

P= (0.8) (0.45) (0.65) (2) (-0.5) = -0.24 (Pressure on leeward face)

$P_{total} = 0.62$ kPa

Assuming the reinforcement is at the center of the panel thickness, the flexural analysis is carried out for the prefabricated masonry panel. The orientation of the panel is such that the flexural stresses are acting perpendicular to the bed joints, and a flexural tensile stress of f_t' = 0.65 MPa is used in analysis. A linear elastic analysis is performed on the prefabricated panel since compressive stresses under transverse loading is predicted to be within linear elastic range. In addition, the wall panel is supported between the pilasters, and bending of the panel would be around the x-axis which is taken to be along the height of the panel. The axial load or the self weight of the panel is acting parallel to the direction of the bending moment. Therefore, the self weight of the panel is also considered in the design.

$f_{compression} = Pf/Ae + Mf/S \leq \phi_m f'm$

$f_{tension} = -Pf/Ae + Mf/S \leq \phi_m f't$

Where,

Pf = 1.4 (2100 kg/m^3)(1.0m)(0.9m) (9.8m/s2) = 25931 N/m For compression

Pf = 0.9 (2100 kg/m^3)(1.0m)(0.9m) (9.8m/s2) =16670 N/m For Tension

Mx = Out of plane bending moment (x is taken to be along the height of the panel)

Sx = Section elastic modulus

W_f = (1.4) (0.62 kPa) (1.0 m) = 0.87 kN/m

Mf = (W_f L^2)/8 =0.87 (2.7)^2 /(8) = 0.90 kN-m/m

Ix = b h3/12 = 1000 (90)^3/12 = 6.08E7 mm^4/m

Sx = Ix / (45) = 1.35E6 mm^3/m

A_e = 90000 mm^2 /m

Based on the above applied moment and cross sectional properties, the applied stresses are calculated as follows:

$f_{compression}$ = 25930/90000 + 0.90E6 /1.35E6 ≤ (0.6) (9.8)

= -0.96 MPa ≤ 5.88 MPa Okay

$f_{tension}$ = -16670/90000 + 0.90E6 /1.35E6 ≤ (0.6) (0.65)

= 0.49 MPa ≤ 0.39 MPa Not Okay

In order to improve the tensile capacity of the panel, steel reinforcement is used to resist the tensile stresses.

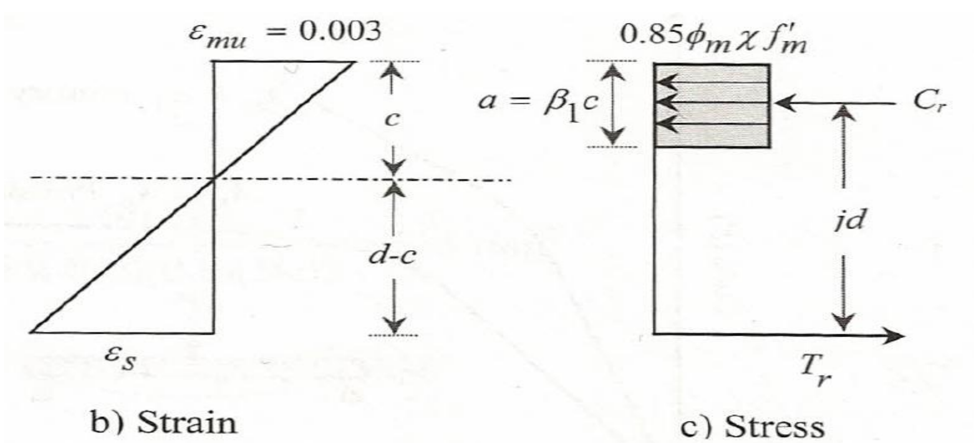

Figure 10: Stress and Strain Distribution Along the Thickness of the Panel (Korany & Hatzinikolas, 2005)

The moment arm is taken to be 0.8d, since the effective depth is 45 mm the area of steel required is calculated as below.

$As = Mf / (0.8\ d\ \phi_s\ fy) = 0.90E6 / (0.85(0.8)\ (45)\ (400)) = 74\ mm^2/m$

Although the area of steel required to resist flexural stresses is 178 mm^2, the panel is provided with As=300 mm^2, this area of steel is required so that reinforcement can be well distributed, and also to take into account the storage and transportation loads acting on it.

Since the load is bent in out of plan fashion, the flexural stress acts perpendicular to the bend joint. In this case, the flexural tensile strength of the masonry panel is lower compared to flexural stresses acting parallel to the bed joint. Therefore, in the above calculation $f_t' = 0.65$ is used.

$Cm = Tr$

$a = (\phi_s\ fy\ As) / (0.85\ \phi_m\ \chi\ f'm\ h_w) = 6.2\ mm$ Where $As = 91\ mm^2/m$

$Cm = 0.85\ \phi_m\ \chi\ f'm\ \beta c\ h_w = (0.85)(0.6)(1)(9.8)(19)(1000) = 30987\ N/m$

$Mr = Cm\ (45 - 6.2/2) = 1.3\ kN\text{-}m/m \geq 0.90\ kN\text{-}m/m$ Okay

$Sx = 1.3E6/(0.6\ (0.85)) = 2.55E6\ mm^3/m$

Now the above section modulus can be used to satisfy the flexural tensile stresses induced by the wind load.

$f_{tension} = 0.17\ MPa \leq 0.0.39\ MPa$ Okay

Therefore, choose 3-10M bars spaced at 1100 mm on centre.

In addition to considering the flexural requirement, it is necessary to consider the bearing forced transferred to pilaster at each end of the panel, and the maximum shear resistance of the panel. These two requirements are calculated as follows:

$f = W_f A_m / 2 = (0.87)\ (2.4)\ (2.7)/2 = 2.82\ kN$

$Vf = 1.04\ kN/m$

Out of Plane Shear

$Vr = \phi_m\ [Vm\ b\ d + 0.25P_d] \leq 0.4\ \phi_m\ \sqrt{f'_m}\ (bd)$

$Vm = 0.16\ (2 - Mf/ (Vf\ d))\ \sqrt{f'_m}$ where $0.25 \leq Mf/ (Vf\ d)) \leq 1.0$, and $d \leq 4t$

Mf= 0.0 at Vf, right and left of the panel; therefore, choose $Mf/ (Vf\ d)) = 0.25$

$Vm = 0.16\ (2-0.25)\ \sqrt{9.8} = 0.16\ (2-0.25)\ \sqrt{9.8} = 0.88\ MPa$

$Vr = 0.6\ [0.88\ (45)\ (360) + 0.25\ (0.9\ (168))] = 33\ kN/m \geq Vf = 1.04\ kN/m$ Okay

If we don't include the self weight, then we would have the following shear resistance.

$Vr = 0.6\ [0.88\ (45)\ (360) + 0.0] = 8.6\ kN/m \geq Vf = 1.04\ kN/m$ Okay

$Vr < 0.4\ \phi_m\ \sqrt{f'_m}\ (b\ d) = 12.2\ kN/m$

Therefore, the out of plane shear resistance is 12.2 kN/m if self weight of the panel is taken into account, or it is 8.6 kN/m if the self weight is not taken into account.

Sliding Shear

In addition to the out of plane shear resistance of the panel, it must also resist the sliding shear, which can be checked according to following:

$Vr = \phi_m \ \mu C$

where C = Compressive force normal to the bed joint = 0.9 Pd + Tr

$C = 0.9\ (0) + 0.85\ (91\ mm^2/m)\ (400) = 30.94$ kN/m

$Vr = 0.6\ (0.7)\ (30.94) = 13$ kN/m $\geq Vf = 1.04$ kN/m   Resistance without self weight Okay

$C = 0.9\ (44.5$ kN/m$) + 0.85\ (91\ mm^2/m)\ (400) = 71$ kN/m

$Vr = 0.6\ (0.7)\ (71) = 30$ kN/m $\geq Vf = 1.04$ kN/m   Resistance with self weight Okay

Therefore, the resistance to sliding shear not including self weight is 13 kN/m and including self weight is 30 kN/m both of which are greater than 1.04 kN/m, satisfying the conditions for sliding shear resistance.

Bearing force = 2.82 kN

Bearing area Required = $2.82E3/(\sqrt{9.8}) = 900\ mm^2$

Bearing width required = 900/2400 =0.38 mm

Although only a bearing area 0.38 mm is required, it is recommended to provide a minimum bearing area of 25 mm at each end of support such that the load can be transferred to the pilasters safely. This design concept is adopted from the paper written by Hatzinikolas & Pacholok.

2) ***Dead Load Design***

The panels must be designed to carry its self-weight during its intended function. This analysis is performed to ensure that flexural strength of the panel under its factored service load is not exceeded. In order to perform the analysis, it is assumed that the reinforcement at the bottom of the panel is effective in resisting the flexural tensile stresses. Therefore, the effective depth of the panel is 2300 mm and the bending moment resistance is calculated as following:

Dead Load= $D = (2100)\ (0.90)\ (9.81) = 18.54$ KN/ m^2

$Df = 1.4\ D = 26$ kN/m^2

$Cm = (0.85\ \phi_m\ \chi\ f'm\ b_w\ a) = 450a$

Assume $As = 100\ mm^2$ is placed at the bottom core of the wall, than the resistance is:

$Tr = (\phi_s\ fy\ As) = (0.85)(400)(100) = 34$ kN

$a = (\phi_s\ fy\ As)\ /(0.85\ \phi_m\ \chi\ f'm\ b_w) = 75.6$ mm

$Mr = Cm\ (1200-76/2) + Tr\ (1200-100) = 77$ kN-m $\geq$ $Mf = Df\ L^2/8 = 57$ kN-m Okay

Therefore, provide 1-10M bar at the bottom core of the panel to resist the flexural force acting on the panel. In addition, the factored in plane shear resistance of the panels is compared with the factored in plane shear force applied to the panel.

$Vf = 26\ kN/m^2\ (2.4)\ (1.0)/2 = 31.2\ kN/m$

Factored In Plane Shear Resistance of the Panel

Assuming the self weight of the panel is included in the design, the shear resistance is:

$Vr = \phi_m\ [Vm\ bw\ dv + 0.25P_d] \leq 0.4\ \phi_m\ \sqrt{f'_m}\ (bw\ dv)\ dv \geq 0.8\ Lw \qquad dv=2300$

$Vr = 0.6\ [0.88\ (90)\ (2300) + 0.25(0.9)\ (62400)\ ] = 118\ kN/m \geq\ Vf = 31.2\ kN/m$ Okay

Although, the shear strength of the panel is greater than the applied shear force, the panel is provided with 100 mm^2 of shear reinforcement due to lifting and rotation of the panel.

The shear reinforcement required in the service load condition functions as flexural reinforcement during handling of the panel, therefore, it is recommended that a well distributed ladder type joint reinforcement be provided for the prefabricated panel of No.9 (3.7mm dia.) at 400 mm spacing. The reinforcement distribution and detail is provided in Figure 12.

3) ***Handling Load***

The design of the prefabricated panel for handling load is considered in the same way as it was designed for the service load acting on it during its life time; however, in this case the panel is designed to resist the impact load that it incurs during handling and storage.

Since the prefabricated concrete masonry panel is lifted from the ground by means of embedded or external reinforcing bars, the design must consider the area of reinforcement required to lift the panel without causing any damage to it. The analysis and design has been carried out as follows:

Impact factor for vertical load (I_f) = 2 (adopted from research paper written by Hatzinikolas & Pacholok)

$P1=P2= P \qquad P= I_f\ (Df/2) = Df = 35.51\ KN$

$As_{required} = 2\ (35510/\ (0.85\ (400)) = 208\ mm^2$ Choose 2 – 10M @ 500 mm from each end

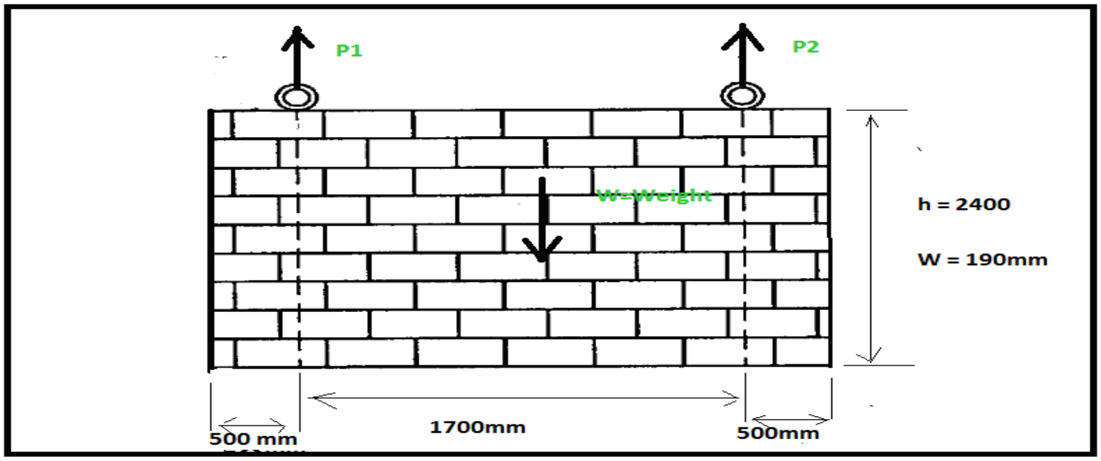

Figure 11: Hoisting Loads Acting on the Masonry Panel (Sutter, 2001)

A total area of 200 mm^2 is chosen because it will be sufficient for the design of this particular panel since the dead load is already factored by 1.4 in addition to the impact factor of 2.0 which has been used in the analysis. Therefore, the total reinforcement for the prefabricated panel is chosen to be 200 mm^2 spaced at 200 mm from each end.

Since the masonry panel is constructed in the traditional way of bottom up construction, it needs to be rotated by a 90 degree angle to make sure that the flexural reinforcement installed in the prefabricated fence panel is resisting the flexural force induced by the lateral wind load. Therefore, to ensure that the panel can be rotated safely, it is required that 200 mm^2 temporary steel reinforcement be provided in each direction to resist the handling loads acting on the panel. As the panel is rotated and lifted in the longitudinal direction, the shear reinforcement along with lifting reinforcement acts as flexure resistance. Assuming that only two of the No.9 joint reinforcement bars and one longitudinal-lifting-reinforcement bar at the bottom of the panel are active in resisting the flexural tensile stresses, the panel in this position has been analyzed as before and it is found that the strength of the panel is sufficient. Namely, the bending moment resistance of the panel was found to be greater than the applied moment. The calculation is carried out as follows:

$Df = 1.4\ D = 62300$ KN/m

$Cm = (0.85\ \phi_m\ \chi\ f'm\ b_w\ a) = 450a$

Assume As = 100 mm^2 is placed at the bottom core of the wall, than the resistance is:

Joint Reinforcement

$Tr1= (\phi_s\ fy\ As) = 2\ (0.85)(400)(11) = 3.74$ kN

$Tr2= (\phi_s\ fy\ As) = 2\ (0.85)(400)(11) = 3.74$ kN

Lifting Reinforcement

$Tr= (\phi_s\ fy\ As) = (0.85)(400)(100) = 34$ kN

$a = (\phi_s\, fy\, As) / (0.85\, \phi_m\, \chi\, f'm\, b_w) = 92$ mm

$Mr = Cm\ (1350\text{-}92/2) + Tr1\ (2(1350) - 3(400)) = 54$ kN-m $\geq$ $Mf = Df\ L^2/8 = 50.5$ kN-m Okay

In Plane Shear Resistance

$Vr = \phi_m\ [Vm\ bw\ dv + 0.25P_d] \leq 0.4\ \phi_m\ \sqrt{f'_m}\,(bw\ dv)$ $dv \geq 0.8$ Lw dv=2300

$Vm = 0.16\ (2-0.25)\sqrt{f'_m} = 0.88$ MPa where $0.25 \leq Mf/(Vf\ d)) \leq 1.0$ and $d \leq 4t$

Vm = 0.88 MPa

$Vr = 118$ kN/m $\geq Vf = 35.1$ kN/m Okay

Sliding Shear

$Vr = \phi_m\ \mu\ C = 0.6\ (1.0)(192.68) = 48.17$ kN/m $\geq Vf = 35.1$ kN/m

Figure 12 summarizes the overall design of 2400X2700x100 prefabricated concrete masonry panel. The design of the panel considered in this report could be improved by selecting a lighter panel such that transportation and handling costs can be reduced.

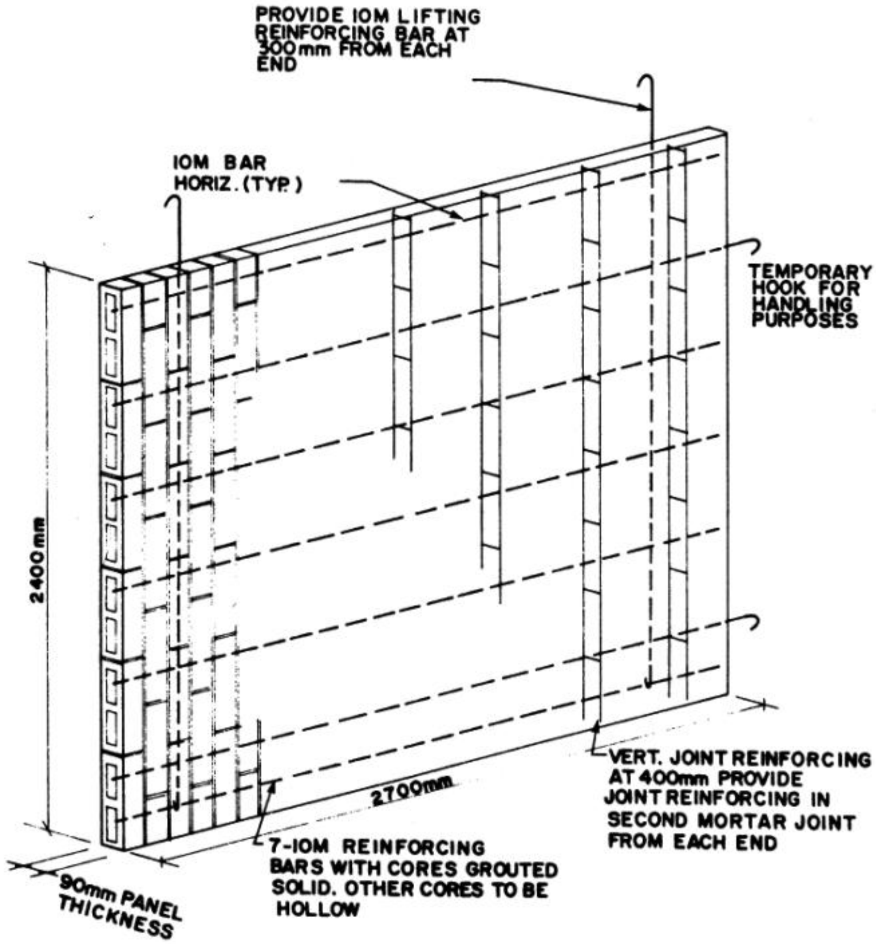

Figure 12: Reinforcing Detail for Masonry Panel (Adapted from Hatzinikolas & Pacholok)

4) ***Storage Load***

The prefabricated panels can be stored in different ways by manufacturers. Some manufacturers queue the panels in rows while others may stack panels on top of each other to reduce the storage area. In this example, a check is performed to verify stacking of panels. It is assumed that the masonry panels are stacked on top of each other in the configuration shown in Figure 13, and then the capacity of the panels are checked against the applied load. The number of panels that can be stacked on each other depends on the self weight of the panel. For an ACP Concrete™ panel it is specified that the maximum number of panels to be stacked on top of each other is six.

Figure 13 shows 6 panels that are stacked on top of each other and are symmetrically placed so the entire gravity load is distributed between the timber supports. It is obvious that the panels at the bottom would have to be stronger than the ones near the top. Therefore, it is required that the crushing strength of the masonry panel be checked at support. For this example the bottom panel governs the design. This temporary strength requirement is needed to facilitate the flow of load to the ground. A factor of 2 is used for impact loading condition, since the panel is under temporary load and could be subject to impact loading during stacking and un-stacking processes.

Panel 1 (governs the design)

Load= 6 I_f (P1+P2) = 6 (2) (2.7) (0.90) (9.81) (2100) = 600 kN/m

P1=P2=P =300kN/m

Check Crushing Capacity

P = 300kN/m (Line load is applied at each support)

Assuming bearing area of timber to be 200 X 2400

$Pr = 0.8\ [0.85\ \phi_m\ f'm\ A_b]$

Pr=0.8[(0.85)(0.6)(9.8)(200000)]=799.7 kN /m >> P = 300kN/m (okay)

Check Flexural Capacity

w = (2) (26 kN/m^2) =52 kN/ m^2 Df = 124.8 kN/ m^2

Mf = (w L^2)/8 = (124.8) (2.7$^{2)}$/8 = 114 kN-m/m

Mr = Cm (45-3.4) = 1.41 kN-m/m << Mf not good

Therefore, the panels cannot be stacked horizontally and must be stored horizontally in a queue. If stacking is the only method of storage, then flexural reinforcement must be provided to increase the strength of the panels or a lighter weight panel must be used.

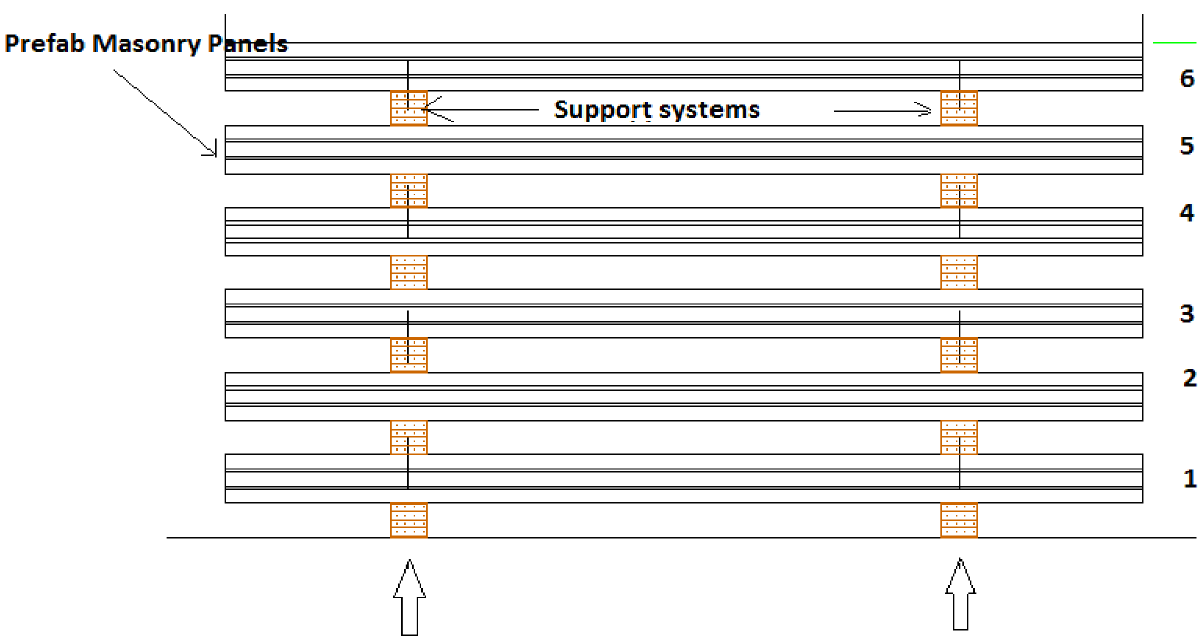

Figure 13: Stacking of Prefabricated Masonry Panels (Adapted from ACP,2002)

Construction

The construction productivity is increased because prefabricated panels are cured offsite and only brought to site when ready to install in place. The only onsite work is the assembly and connection of the individual panels, which can be efficiently accomplished with unskilled labour. The construction time can be reduced to as much as 85% when compared to conventional laid in place masonry. This also results in less interference between other trades.

Joint Connections

For the purpose of this report, the construction procedure and installation of prefabricated fence panel is considered. The construction of the prefabricated panel starts with installation of channeled masonry pilasters as shown in Figure 15. The installation must meet the local building code requirements in addition to the site condition restraints and bylaws. The prefabricated panels are then slid into the pilaster's channel between pilasters. It is recommended that a minimum of 25 mm bearing length be provided at each end of support to transfer the lateral wind load and self-weight of the panel to the pilasters (Hatzinikolas & Packolok). This construction procedure and installation has been summarized in Figures 14-16 and the final finished prefabricated structure is shown in Figure 17.

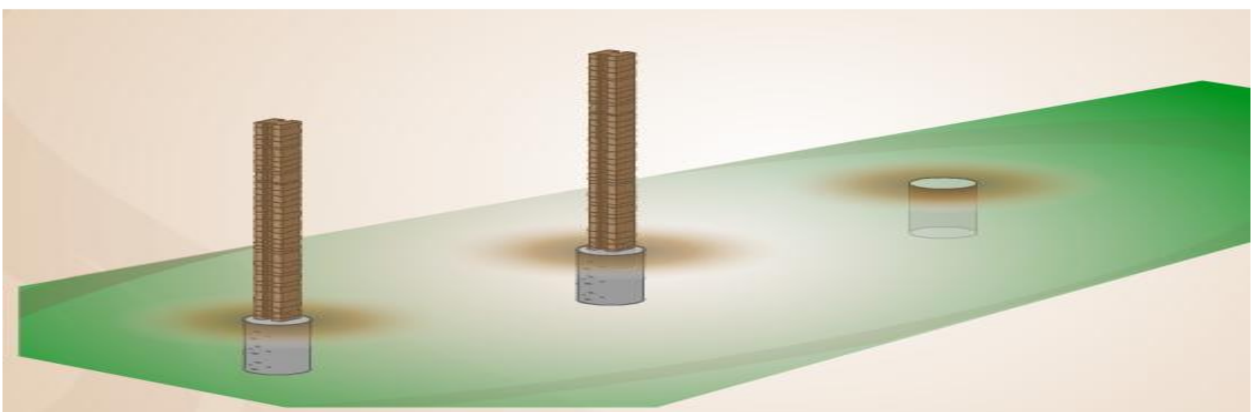

Figure 14: Installation of Pilasters or Support Columns (Artisan, 2010)

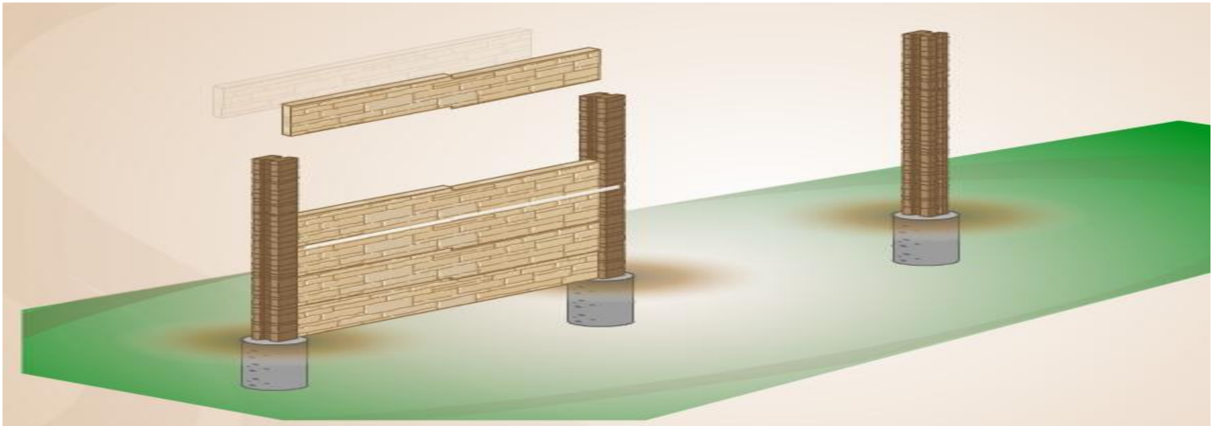

Figure 15: Installation of Prefabricated Panels Between Pilasters (Artisan, 2010)

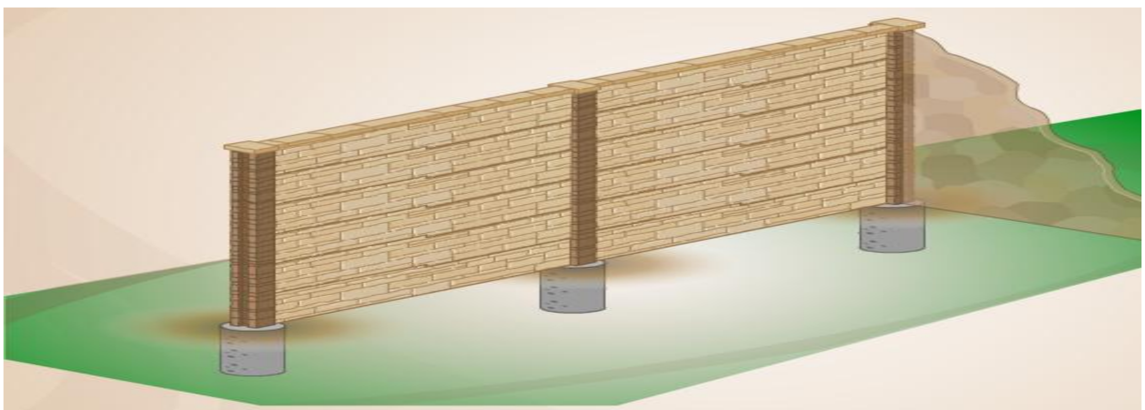

Figure 16: Installation of Post Caps (Artisan, 2010)

Figure 17: Prefabricated Sound Barrier Wall (AFTEC, 2010)

There are various post or pilaster designs to resist the lateral loads acting on the fence panel. The design discussed below is patented and is presented in this report for educational purposes only.

Figure 18: Cross Sectional View of Light Weight Steel Gauge With and Without Torsion Sleeve (Boot, 1994)

The post system used in this construction has a thickness of 1 mm to 5 mm, making it very flexible and highly susceptible to buckling and out of plane torsion due to wind loading. A torsion sleeve, shown in Figure 18b, is used to provide additional stiffness and rigidity and should be installed at points of high stress concentration. This occurs at the base of the post due to the cantilever action of the post. The width of the torsion sleeve depends on the forces acting on the sleeve and can vary from 30 mm-800 mm depending on the design requirements.

It is important that the joint connection between the panels be sufficiently strong to withstand the loads acting on it. Figure 19 shows the panel to panel connection of the prefabricated panel to the pilaster to assist the system in resisting the service loads acting on the structure. An angle bracket and steel plate connection at the panel – post juncture is also provided to stiffen and connect the panels to the post.

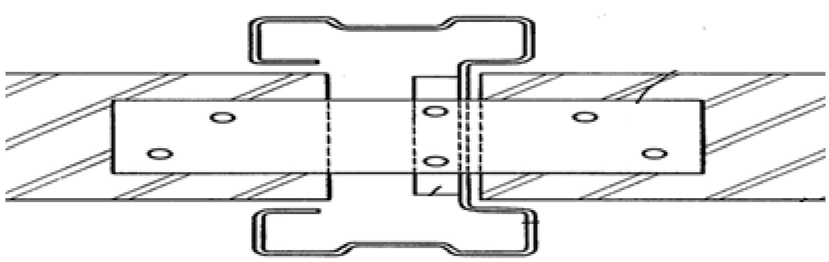

Figure 19: Prefabricate Masonry Connected to a Fence Post by Means of Steel Plate (Boot, 1994)

This post design system is optimal for prefabricated panels with a thickness between 50-100 mm, height between 1500-2000 mm and length between 3000-4000 mm. By incorporating the sleeve torsion, angle bracket, and steel plate a light structure like the post can be given sufficient rigidity to withstand the loads acting on it during its service life.

Connection to Building Envelope

The connection of a prefabricated panel to the building envelope is very important because it is part of a system. Connections to the building envelope need to hold panel self weight and resist wind load. The advantage of prefabricated walls is that movement can be introduced directly in to the connection and not in the brick panel. Hence, the panels need to be isolated from the rest of the building. The designer needs to consider differential movement due to external influences and also influences of the building envelope. All three directions (perpendicular, parallel and vertical) of the wall need to be considered when designing connections because they are static connections that also need to allow for some differential movement. The problem of differential movement is solved by introducing soft joints between panels (Tawresey, 2004). Soft joints are introduced between panels in the vertical direction and in the horizontal direction, this prevents water from infiltrating but allows panels to expand and contract. In most cases double caulked joints are used. The caulking is dependent on the type of brick and what is called a compatibility test which needs to be performed for every job (VetoVitz, 2010).

The material in connections is made up of miscellaneous steel, tee's, steel angles, plates and rods which are most commonly used. It is also noted that the steel will be exposed to weathering, and it is usually shop painted. The connector is required to employ multiple bricks to prevent failure of the connection due to cracking of a single brick. In seismic regions, it is required for the connection to the masonry to be wrapped around the reinforcement to allow for transfer of loading directly to the reinforcement (Tawresey, 2004).

Typically there are two gravity connections; one at each end of the wall. These gravity connections need to bear the dead load and also resist moment in a direction that is perpendicular to the plane of the wall. Figure 20 illustrates a connection that is used in many panel systems. This is a connection to a concrete slab where an angle is embedded into the masonry wall when fabricated which is shop welded to a plate. This assembly will then be welded to an embed plate in the slab. This connection needs to be designed for the forces in the direction of the arrows shown in figures 20 and 21.

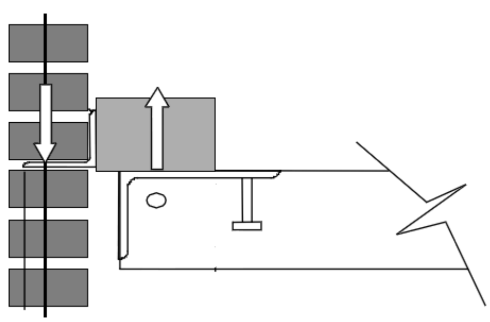

Figure 20: Gravity Connection Illustrating Vertical Forces (Tawresey, 2004)

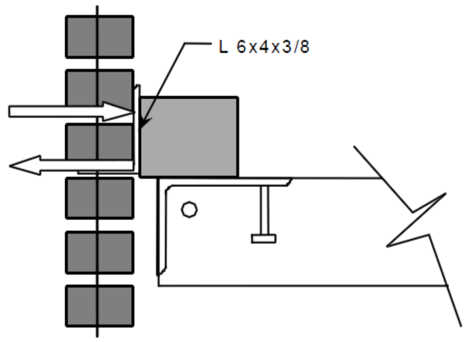

Figure 21: Gravity Connection Illustrating Forces Perpendicular to Plane of Wall (Tawresey, 2004)

In addition to the gravity connections, a wind slip connection must be installed every 10 ft. These connections need to be stiff in the direction perpendicular to the plane of the wall but need to be able to move parallel to the plane of the wall. External influences such as temperature change and seismic loading is of most importance when designing for differential movement of these connections. In figure 22 a typical lateral connection between panel and structural envelope is illustrated. This connection acts to support the wind load in the direction perpendicular to the plane of the wall. Here a T section installed during prefabrication is welded a coupler clasping a rod. The end of the rod is threaded and the coupler, which acts like a nut, will hold the rod in place. The coupler allows for some construction tolerance when erecting the panel because if the panel is slightly out of place in the direction perpendicular to the plane of the wall, the rod can be threaded deeper into the coupler (Tawresey, 2004).

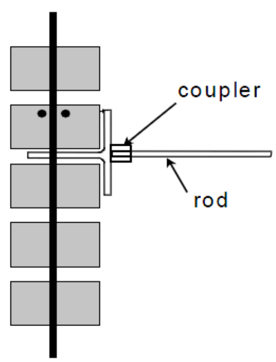

Figure 22: Lateral Connection for Prefabricated Panels (Tawresey, 2004)

Prefabricated masonry connections are manufacturer dependant and the above only presents typical building envelope connections. It is extremely important for the designer to work in conjunction with the panel manufacturer when designing connections.

Economic Feasibility

During the 1970's and 1980's researchers and practicing engineers all over North America saw prefabricated masonry as a viable construction material. Their goal was to justify why prefabricated masonry should be used over popular materials such as concrete, wood and steel. The engineering advancements were not in the material itself but the manufacturing methods with the underlying challenge to build masonry elements more efficiently.

By the early 2000's North American prefabricated masonry took place predominantly in the Pacific Northwest region of the United States. The following companies were major contributors to the prefabricated masonry industry.

- KPFF Consulting Engineering (Western United States)
- Barkshire Panel Systems (Washington)
- L.C. Pardue (Oregon)
- Vet-O-Vitz Panel Systems(Ohio)

KPFF Consulting Engineering is a large design firm spread across the western United States. Barkshire Panel Systems, L.C. Pardue and Vet-O-Vitz panel systems were companies that manufactured and installed prefabricated masonry panels. There were other smaller companies scattered around North America, but were prevalent industry players.

In conducting the literature review, it was found that there is no new information on any of these companies regarding prefabricated masonry. To obtain current information interviews with practicing engineers in the prefabricated masonry field were conducted. An expert in prefabricated masonry at KPFF, Steve Dill, was contacted and asked to explain the current situation of prefabricated masonry in the United States. He made it clear that that prefabricated masonry is no longer widely used and the three companies Barkshire Panel Systems, L.C. Pardue and Vet-O-Vitz Panel Systems are no longer involved with prefabricated masonry. On top of which, Barkshire Panel Systems and L.C. Pardue have gone out of business. The Vet-O-Vitz Panel System was bought out and is now called Advanced Masonry Technology but is no longer manufacturing prefabricated masonry panels (Dill, 2010).

There are several reasons why the prefabricated masonry industry died off. Executive Director of The Masonry Society, Phillip Samblanet, gave a few economic reasons why prefabricated masonry is not being used. He explained that in order to save money on brick panels, a multistory building that has very regular elements needs to be in high demand. Currently, with the economy the way it is in North America, the market is dominated by smaller storey projects. From an economic point lower storey buildings are more efficiently built with laid in place masonry (Samblanet, 2010).

Another major contributor to the masonry industry is Jon Chrystler from the Western States Clay Product Association. He stated that contractors prefer laid in place masonry because of the flexibility involved. When building prefabricated masonry, the dimensions are predetermined and more stringent tolerances are required. A building is made of many different components all from different trades. The stringent construction tolerances of prefabricated masonry affect these other trades (Chrysler, 2010).

William VetoVitz stated some different reasons. He was very optimistic about the industry and sees much potential. He said it was a problem with marketing. It is a proven fact that prefabricated masonry panels are not less expensive than laid in place masonry panels; therefore, they do not seem attractive unless they are marketed with an architectural appeal. The prefabricated panels are more of a specialty building material and should be used on impressive structures where owners are willing to spend the money on a building for its image. Since complex shapes cannot be constructed using conventional hand laying techniques on site, there is a need for prefabricated masonry (VetoVitz, 2010).

Another reason why prefabricated panels are not as popular today as they were back in the 1970s and 1980s could be because of doubts lingering in some engineers due to the Sarabond panel fiasco in the 1990's. There was a large lawsuit on the mortar used in these panels against the Dow chemical plant. The problem stemmed from the additive in the mortar which ended up cracking the panels and causing major damage to the building envelope. There aren't any findings to prove that this was the cause of a decline on popularity among panels, but especially for people not familiar with the industry, this could cause a negative effect (SC Judicial Department, 1997).

It also can be inferred by the literature that companies are not willing to spend the money on the capital equipment required to build prefabricated masonry. The manufacturing of the panels is where most of the cost is incurred and fronted by the company. There is a significant amount of money that goes into prefabricated masonry before a panel is made. The large initial costs for a business entering a new market make it very risky. Referring back to the manufacturing section of this report, in order to be efficient in laying bricks, apart from the warehouse itself, at a minimum cranes and fork lift's need to be purchased for the plant. Usually masons will want automated scaffolding as well. In addition, money needs to be spent on large transporting trucks and hauling equipment. What if after all of this, the site is inaccessible by large trucks? This will limit the size of the panel that can be fabricated. What if the construction site is far from the manufacturing plant? Then transportation costs will skyrocket. For all of these reasons, it is understandable for a company to be hesitant to enter the prefabricated industry, especially, with other viable alternatives to prefabricated masonry.

Conclusion

Prefabricated masonry is a growing industry which will continue to gain a more wide scale acceptance. With continued research there will be more advanced techniques, requirements and applications for prefabricated masonry. The various types of manufacturing methods and materials have made it more adaptable to a wider variety of applications, and with improvements in cost it is possible to make it a more viable alternative to conventional masonry techniques. After a thorough literature review of prefabricated masonry, the authors have come up with the following concluding points:

- Prefabricated masonry panels can be installed with less skilled labour and since the panels are already finished and cured on-site construction can be significantly reduced.
- Lack of resources such as literature and research on the topic has made it hard for the product to be considered in design by engineers.
- Presently, most prefabricated panels produced by the manufactures are non-load bearing walls, fences, fireplaces and veneers.
- Even though there is a cost savings in prefabricating the wall in a controlled environment like a shop, the transportation and storing costs do not allow prefabricated masonry to be cheaper then laid in place masonry.

- The cost of the prefabricated masonry can be reduced by introducing an automation manufacturing system and mass production which is a topic that still requires research.

- In order to improve success in the industry, focus should be made on the architectural niche market.

The objective of this paper is to inform the reader about the manufacturing, application, and design of prefabricated masonry. This objective was achieved and used to present to the current state of the industry.

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All have provided guidance and encouragement to the authors of this paper and immensely contributed to the quality of information provided in this technical report.

List of Symbols

A_e = effective cross-sectional area of masonry, mm^2

a = depth of the equivalent rectangular stress block, mm

A_v = cross-sectional area of shear reinforcement, mm^2

b_w = overall web width, mm

b = effective width of rectangular member, or flange for T and I sections or webs as defined for each case, mm

d = distance from extreme compression fibre to centroid of tension reinforcement, mm

d_v = effective depth for shear calculations, which need not be taken as less than 0.8_w for walls, mm

E_m = modulus of elasticity of masonry, MPa

E_s = modulus of elasticity of steel, MPa

f_t = flexural tensile strength of masonry, MPa

f_y = yield strength of reinforcement, MPa

h = unsupported height of a wall or column, mm

h_w = total wall height, mm

I = the moment of inertia of wall section for out-of-plane bending, mm^4

L = length of flexural wall panel or length of masonry infill shear wall, mm

M_f = factored moment, kN•m

M_r = factored moment resistance, kN•m

P_{cr} = critical axial compressive load, kN

v_m = shear strength of masonry, MPa

V_f = shear under factored loads, N

V_m = factored shear resistance of masonry members provided by the masonry, N

V_r = factored shear resistance, N

V_s = factored shear resistance provided by shear reinforcement, N

w = diagonal strut width, mm

w_f = factored uniform lateral wind or seismic load on the wall, N/mm

ρ = density of the masonry

Application of Advanced Composite Material to Masonry Structures: Field Applications and Design Methods

A. Jamshidi, M.S. Gorji and M.A. Nazief

Department of Civil and Environmental Engineering, University of Alberta

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ABSTRACT

Advanced composite materials such as Fibre Reinforced Polymers (FRP) are used widely in masonry construction nowadays. Strengthening and retrofitting of existing members, application of composite materials to enhance the capacity of masonry structures is of most interest to the field. This paper briefly presents the characteristics of the materials and the design methodologies of FRP-strengthened masonry walls from the structural standpoint.
Key words: Masonry Structures, Fibre Reinforced Polymers, Walls, in-plane behaviour, out-of-plane behaviour.

Introduction

Masonry is considered one of the oldest materials used in the construction industry. A lot of evidence about its use can be seen from ancient buildings such as the Giza pyramids in Egypt. In the middle centuries (17^{th} century) masonry was used extensively in the construction industry. An example of a building that was constructed in this period is the Church of Our Lady in Dresden-Germany.

Advance composite materials (Fibre Reinforcement Polymers (FRP)) are used to improve the performance of masonry structures. The FRP can be used in many applications. It can be used to increase the mechanical (Compressive, Flexural...etc) strength of the masonry elements. It can also be used to retrofit different elements instead of the use of concrete or steel elements (Aiello & Sciolti (2006)).

Fibre Reinforced Polymers

FRP's are high strength fibres embedded in a resin matrix. There are several types of fibres that are present in the market. The most common used types are Carbon fibres (CFRP's), Glass fibres (GFRP) and Aramid fibres (AFRP's) (refer to Fig. (1)). The mechanical properties vary a lot from one type to another (for more details refer to Shrive (2006)).

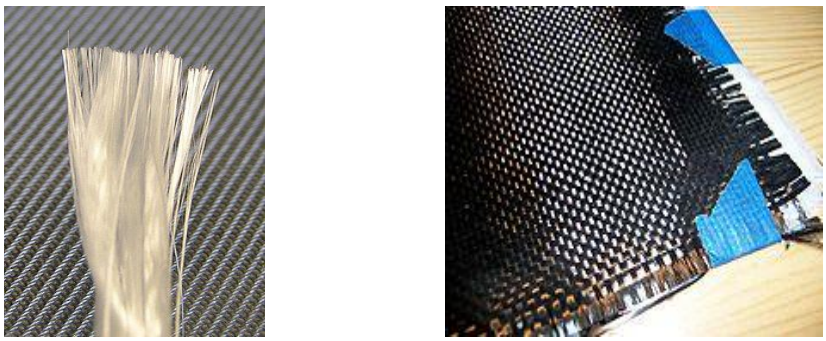
The use of the FRP has advantages and disadvantages. A lot of researchers dealt with the advantage of the use of these materials (Triantafillou (1998), Arduini and Nanni (1997), Fam et al (1997), Hamid (1996), Galati et al. (2006)):

a. It has a good corrosion resistance.
b. Its application to already existing structures is not that difficult compared to the traditional retrofit method (the use of concrete and steel).
c. It increases the strength of the element without the need to increase its dimension.
d. The repair is unobtrusive. Therefore they are useful in the repair of historical buildings.

e. It increases the strength without increasing the mass. Hence, improve the dynamic resistance of the building.

However, there are some disadvantages that should be pointed out (Kolsch (1998), Shrive (2006)):

a. There is no evidence on the long term behaviour of these materials.
b. Lack of fire resistance.
c. The resin used is not environmental friendly. They are flammable and give toxic vapours when burned.
d. The resin is sensitive to light. It becomes brittle when they are subjected to direct sunlight. Therefore it must be protected from direct sunlight.



a) Glass Fibre Polymers b) Carbon Fibre Polymers

Figure (1): Examples of the most common used Fibres (Wikipedia (2010))

FRP are usually provided in the form of sheets, strips or tendons for pre-stressing application, bars and meshes. The fibres can be arranged in one direction. The strength will be greater in that direction than in the transverse one. If the fibres are arranged multidirectional, this will provide orthotropic properties and hence the behaviour in both orthogonal and transverse direction is not significantly different (Ibrahim et al. (2000)).

The effect of using various material forms for the FRP and methods of installation on the in-plane and out-of-plane behaviour have been examined (Nanni and Tumialan (2003)). Also, failure mode for the in-plane behaviour of masonry walls was investigated by lots of researchers (Moon et al. (2002), Badoux et al. (2002), Fam et al. (2002)).

Tensile strength of FRP depends on several factors. The type of fibres, its orientation with respect to the load, the quality of that type of fibres and the storage time for these fibres will affect its strength (ACI 440-2R-02). The tensile properties of the fibres are provided by the manufacturer. FRP should not be used to resist compressive stresses. The compressive strength of the fibres is so small when compared to the tensile strength. When the fibres are subjected to compressive force they usually buckle which will result in its failure. Ehsani (1993) stated that the compressive modulus of elasticity is approximately 80% of the tensile modulus of elasticity for GFRP, 80% for CFRP and 100% for AFRP.

FRPs have been used extensively in the repair of many structures. Most of these structures are of a historical nature. An example of these applications is the San Francisco City Hall Building. This building was undergoing seismic retrofit in the mid-1990s. Another example is retrofitting medieval bell tower in Serra San Quirico, Ancona (Italy)

Methods of Installation of FRP to Masonry

There are several methods that can be used to install this material to the masonry wall. These methods are Near Surface Mounted (NSM), Surface mounted Reinforcement and Surface mounted laminate. In order to guarantee the success of the installation of these techniques, the surface of the wall must be prepared. Unfilled cracks or irregularities in the wall surface can easily cause debonding.

Near Surface Mounted (NSM)

NSM is the most common technique that is used for mounting the FRP. The FRP rod or strip is embedded in groove that was cut in the surface of the masonry. These grooves are cleaned using pressurized air jet to remove any fine materials present after cutting it. Epoxy coating is then applied to the grove to cover part it. Afterwards, the FRP is mounted and the grooves are closed using epoxy (refer to Figure 2). This is done to help in developing the composite action between the masonry and the used FRP.

The main advantage of this technique is that it requires the minimum surface preparation. Moreover, it is cost effective. The drawback of this technique is that it cannot be used where the wall is subjected to high gravity load or when the wall is severely deteriorated. Also, it is not effective when the moisture content of the wall is large such as basement walls (Hatzinikolas and Korany (2005)).

Surface Mounted Reinforcement and Laminates

In this technique, the FRP strips are mounted on the surface of the wall directly (refer to Figure 2). The surface of the wall should be free of loose materials. The surface should be levelled using an epoxy mortar. An inverted V-grooved spatula is used to apply a thin layer of 2-3 mm of epoxy adhesive to FRP strips. FRP strips are placed on the wall and pressed against it using a rubber roller until the adhesive material is squeezed out from both directions (Hatzinikolas and Korany (2005)).

This method of mounting the FRP is so fast when compared to NSM technique. Also, it allows better accessibility to bonding area and better quality control (Hatzinikolas and Korany (2005)).

a) NSM b) Surface mounted c) Overlay Reinforcement

Figure (2): Different technique of installation of FRP

Overlay Reinforcement

This is an old technique where traditional reinforcement was used to enhance the in-plane and out-of-plane behaviour of the wall (refer to Figure 2). Shotcrete and ferrocement is then applied to the whole surface of the wall. Nowadays, FRP wraps are used to replace the reinforcement. Unlike the traditional reinforcement, the FRP wraps are light in weight. They are not going to add much weight to the structure, hence no need for further repair for the carrying elements. This technique helps with saving of the used material and provides an optimum design for the required elements (Hatzinikolas and Korany (2005)).

In this technique a layer of epoxy sealant is applied to the surface of the wall, then overlaying the FRP layer to the wall surface, and finally finishing with a layer of epoxy saturant. This technique was applied to a vaulted masonry Ceiling of the Christ Church Cathedral in B.C.

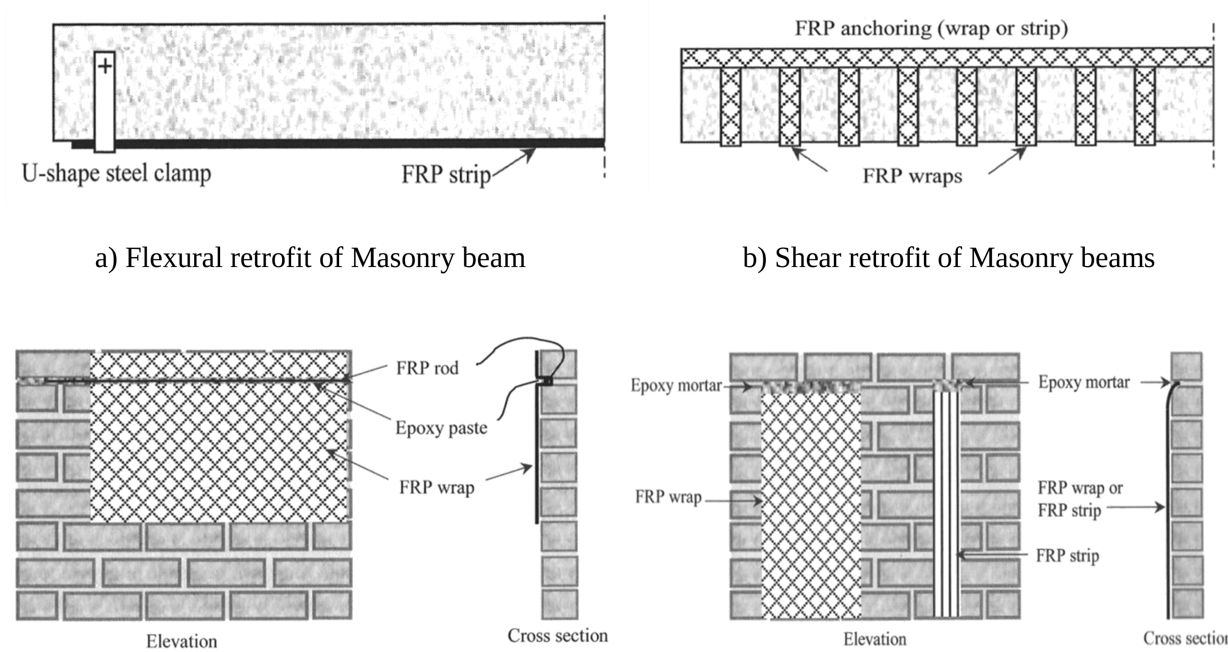
Bond and Anchorage

The objective of the bonded FRP technique is achieved through the perfect adhesion between FRP and the structural member, so the bond quality is a determinant factor in this matter. The success of FRP application is greatly affected by ambient condition. The temperature and humidity issues should be carefully considered according to manufacturer's recommendation. The high humidity at the surface negatively affects the mechanical interlocking and results in low bond quality. It is worth mentioning that the method should be applied at the optimum operational temperature of the resin. Surface preparation is another important factor that affects the bond quality significantly. Sand blasting and levelling are applied to very smooth surfaces and to very rough surfaces respectively. It is not recommended to apply FRP strips and wraps to concave surfaces to prevent the premature debonding. One of the most important issues in FRP retrofit methods is peeling off due to inadequate development length and low tensile strength of masonry to carry out the concentrated stresses below the wrap and strip ends. Several methods have been introduced to prevent this problem, such as mechanical anchorage, FRP anchorage, and embedded inside masonry. U-shape steel clamps can be used to anchor FRP bending reinforcement to prevent peeling off (Figure 3-a) (Hatzinikolas and Korany (2005)).

FRP wraps and strips can be used in shear retrofitting to anchor the FRP stirrups (Figure 3-b). The FRP rod is also used to anchor FRP wraps applied to masonry. The method is shown in figure (3-c). Imbedding the FRP inside masonry is an alternative to improve the anchorage and prevent peeling off (Figure 3-d) (Hatzinikolas and Korany (2005)).

Fire Resistance of FRP

The performance of FRP in high temperature and fire situation is not well recognized. The number of researches conducted in this area is limited. The material characteristics of FRP composites are stable below the glass transition temperature (Tg) of its resin. However, significant changes in mechanical properties of the material occur above this temperature. The resin starts decomposing when the temperature reaches the decomposition temperature (Td). Hence, it produces smoke, liquids, incombustible and combustible gases (Bai et al. (2008)). Due to lack of fire resistance of FRP composites, it is not reasonable to apply these materials where the fire safety is critical in structural design. Some works have been done to improve the fire resistance of FRP specially using flame-retardant intumescent coating (Hörold (1999), Porter et al. (2000)) or a liquid-cooling system (Keller et al. (2005), Keller et al. (2006)). One of the problems is the Coefficient of thermal expansion of the FRP. For some types of FRP, it is not constant in the longitudinal and transverse direction. Some fibres have a negative value for the coefficient of thermal expansion in the longitudinal direction and a positive one in the transverse. For these fibres when it is subjected to heat, it shrinks in one direction (longitudinal) and expands in the other direction (ACI 440-2R-02).



a) Flexural retrofit of Masonry beam

b) Shear retrofit of Masonry beams

c) Using FRP Rods

d) Using embedment

Figure (3) Anchorage system for external FRP Reinforcement. (Hatzinikolas and Korany, 2005)

The application of these materials are relatively new, more experimental and analytical approaches are required to predict and clarify the performance of FRPs under high temperature and fire condition to develop a comprehensive fire resistance design guidelines for FRP-strengthened structural elements.

Quality Assurance

According to the existing guidelines which address to the design of FRP strengthened structures, quality assurance of the applied technique should be maintained. Quality assurance is achieved through a set of inspections and applicable tests to document the acceptability of the installation (ACI 440-2R-02). The objective of the bonded FRP technique is achieved through the perfect adhesion between FRP and the structural member, so bond quality assessment of the applied FRP should be carried out. Non-destructive testing is one of the main procedures to identify and verify the quality of the FRP technique used. In the ICBO standards, defects in the bonding are allowed if less than 13 cm^2 for a maximum of 10 delamination per about 1 m^2 (ICBO (2001)).

The most common non-destructive evaluation techniques that have been applied to masonry structures especially historic buildings can be mentioned as follows: impulse radar, impact echo, ultrasonic pulse velocity, spectral analysis of surface waves, electromagnetic detection, infrared thermography and fibre optics. Advanced materials and equipments are used under the supervision of well trained inspectors to assure the quality of the Non-destructive testing evaluations. It is worth noting that the environmental effects on the evaluation results should be taken into account.

Durability of the FRP System

There are many factors that affect the durability of the FRP system. The durability is affected by the high temperature, humidity and chemical exposure to the surrounding media (Karbhari (2007)). To increase the durability of the system, FRP must be coated to protect it from any potential environmental damage that it could be subjected to. These Harmful environments could be (ACI 440-2R-02):

a. Ultraviolet light protection: Coating must be applied to prevent degradation and change in the mechanical properties (tensile and compressive strength).
b. Vandalism: Protection against vandalism must be hard and durable. Polyurethane coating can offer protection against cutting and scraping. Overlaying of cementitous material could provide more protection.
c. Impact, Abrasion and wear: the types of coating used for protection against these conditions are similar to those for the vandalism. These coatings should be very durable, since it does not occur once.
d. Aesthetics: Acrylic latex coatings that match the color of the masonry are used to conceal the FRP.
e. Chemical Resistance: Exposure to harsh chemical must be prevented. These chemical will greatly affect the behaviour of the applied FRP. Using chemical resistance coating such as urethanes and novolac epoxies will give the required resistance.
f. Fire: Fire affects the FRP greatly. When the temperature reaches the glass transition temperature of the resin, the tensile strength, stiffness and bond properties of the FRP will be affected greatly specially if the FRP is applied externally. Therefore, it is a must to use coatings that are capable of isolating the FRP so that it will not lose its strength.

It should also be stated that the proper storing and handling of these materials will lead to increase the life time, hence its durability.

Design Methodology

This section presents the design methodology that is presented in different standards. The most common standards are the ACI 440-2R-02, CSA S806, and the ISIS Canadian manual. Design of Un-reinforced masonry (URM) walls for the out-of-plane and in-plane will be addressed in this section.

ACI 440-2R-02

The design methodology of the ACI 440 is based on the limit state design principle. The ACI 440 stated some assumption for the design of unreinforced masonry (URM) walls. These assumptions are listed as follows:

a. Plane section before bending remains plane after bending. This will lead to the fact that the strain in the FRP and the masonry are directly proportional to their distance from the neutral axis.
b. The maximum usable compressive strain in concrete masonry blocks is 0.0025, and for clay and natural stone masonry is 0.0035.
c. The tensile stress in the FRP is linear up to failure.
d. Neglect the masonry in tension and the FRP in compression.
e. No relative slip between the masonry and FRP until de-bonding occurs.

The behaviour of the wall arching mechanism is neglected. That means that the wall will act as simply supported element, or, at least close to that.

FRP is mainly used to reinforce URM walls subjected to earthquake, wind, hurricane, blast loads and earth pressure. The ACI 440 provides a limitation for the maximum tensile strength and the corresponding strain that can be resisted by the fibres as well as the modulus of elasticity (refer to equation (1), (2) and (3))

$$f_{fu} = C_E f_{fu}^* \tag{1}$$

$$\varepsilon_{fu} = C_E \varepsilon_{fu}^* \tag{2}$$

$$E_f = \frac{f_{fu}}{\varepsilon_{fu}} \tag{3}$$

Where: f_{fu}= Design ultimate tensile strength of the FRP.

ε_{fu} = Design ultimate tensile strain in the FRP.

f_{fu}^* = Ultimate tensile strength for the FRP as reported by the manufacturer.

ε_{fu}^* =Ultimate tensile strain for the FRP as reported by the manufacturer.

C_E = Environmental reduction factor (Refer to table 8.1 in the ACI 440M).

E_f = Modulus of Elasticity for FRP.

The value of C_E is close to unity for FRP located indoors. Its value will decrease significantly as the FRP are exposed to severe conditions. The ACI 440 limits the maximum tensile strength carried by the FRP taking into account the bond between the FRP and the masonry wall. Equations (4) and (5) show the effective strain and stresses in the FRP.

$$\varepsilon_{fe} = K_m \varepsilon_{fu}^* \leq \varepsilon_{fu} \tag{4}$$

$$f_{fe} = E_f \varepsilon_{fe} \tag{5}$$

Where K_m is the bond reduction coefficient? Its value is equal to 0.45 for FRP laminate and 0.35 for NSM FRP.

The effective strain and stress used to design for the in-plane shear using FRP is given by equation (6) and (7).

$$\varepsilon_{fe} = K_v \varepsilon_{fu}^* \leq \varepsilon_{fu} \tag{6}$$

$$f_{fe} = E_f \varepsilon_{fe} \tag{7}$$

Where K_v is the bond reduction coefficient for shear controlled failure mode. It is dependent on the FRP reinforcement index, ω_f , defined in equation (8)

$$\omega_f = \frac{1}{85} \frac{A_f E_f}{A_n \sqrt{f_m^`}} \tag{8}$$

Where A_f= The cross-sectional area of the FRP reinforcement.

A_n= The Area of the net mortared/grouted section.

$f_m^`$= The masonry compressive strength.

The value of K_v is based on experimental data that was conducted by several researchers (Tinazzi and Nanni (2000), Tumialan et al. (2001), Morbin and Nanni (2002), Vallunzzi et al. (2002), Grando et al. (2003), Zhao et al. (2003), Santa Maria et al. (2004), Senescu and Mosalam (2004), Stratford et al. (2004) and Santa Maria et al. (2006)). Its value is given as equation (9)

$$K_v = \begin{cases} 0.40 & for\ \omega_f \leq 0.20 \\ 0.64 - 1.2\omega_f & for\ 0.20 < \omega_f \leq 0.45 \\ 0.1 & for\ \omega_f > 0.45 \end{cases} \qquad (9)$$

Out-of-Plane behaviour of URM walls

When the FRP is used to increase the out-of-plane flexural strength of a masonry wall, the wall should be checked to have the adequate shear strength. If it was not adequate to resist this shearing force, shear strengthening must be done. Grouting of the hollow units near the supports as well as reducing the member span by providing bracing would improve the shear resistance for such walls.

The flexural strength of the URM wall should be designed so that its capacity is greater than the applied forces (Refer to Equation (10)).

$$\emptyset M_n \geq M_u \qquad (10)$$

Where $\emptyset$ is the strength reduction factor and its value is taken 0.60, M_n is the nominal strength of the section with the FRP and M_u is the factored load applied on the wall. Taking the moment about the centroid of the compression block, the value of M_n will be as in equation (11)

$$M_n = A_f f_{fe}\left(d_f - \frac{\beta_1 c}{2}\right) + P_u\left(\frac{t}{2} - \frac{\beta_1 c}{2}\right) \qquad (11)$$

Where A_f= The cross-sectional area of the FRP used.

f_{fe}= The stress in the FRP.

d_f= The depth till the FRP centroid.

$\beta_1 c$= The depth of the masonry compression block.

t= The depth of the wall.

The failure criteria that the wall may fail in are either i) crushing in the masonry blocks or ii) De-bounding of the FRP system. Crushing in the concrete is an acceptable mode of failure provided that the serviceability requirement is satisfied. The most common type of failure that is expected to occur is the de-bounding of the FRP in case of the absence of any mechanical anchorage. To prevent the de-bonding of the FRP, the strain in the FRP must be limited to its effective strain given by equation (4).

In-Plane behaviour of URM walls:

The behaviour of the URM walls under in-plane loading depends on several factors such as: height of the wall, its thickness, slenderness, and bond pattern. Strength of the mortar and the block unit also are important factors. Also, the loading and support conditions affects the design. The most common types of failure are: i) Joint sliding, ii) Diagonal tension and iii) Toe crushing. Both the joint sliding and the diagonal tension are shear failures. Generally, it is not preferable to have diagonal tension failure because it is a brittle failure. Toe crushing is a flexural controlled failure. The nominal capacity of the wall for in-plane loading is computed as follows:

$$V_n^{URM} = \min\left(V_{bjs}, V_{dt}, V_{tc}\right) \tag{12}$$

Where V_{bjs} is the nominal lateral strength corresponding to joint sliding, V_{dt} is the nominal lateral strength corresponding to the diagonal tension and V_{tc} is the nominal lateral strength corresponding to toe crushing.

The wall has to be investigated first to see whether it needs to be strengthened or not.

For shear reinforced masonry the factored loads must not exceed the nominal capacity as given in equation (13):

$$\emptyset V_n \geq V_u \tag{13}$$

Where the value of $\emptyset$is taken to be 0.8.

The ACI 440 has some limitation for the use of the FRP as reinforcement for the wall. Table 1 summarizes these limitations (ACI 440-02).

Table (1): Limitation on the use of the FRP (ACI 440)

Masonry Type	Wall Construction	FRP Strengthening Layout
Hollow Unit Masonry Walls	t=200 mm or less Ungrouted walls or Partially-grouted walls with grouted cell spaced greater than 1.20 m	FRP on one face of the wall is acceptable
	t=200 mm or less Fully-grouted walls or Partially-grouted walls with grouted cell spaced 1.20 m or less	FRP on two faces of the wall is required
	t=250-300 mm Ungrouted walls or Partially-grouted walls with grouted cell spaced greater than 1.50 m	FRP on two faces of the wall is required
	t=250-300 mm Fully-grouted walls or Partially-grouted walls with grouted cell spaced 1.50 m or less	Use of the FRP is not recommended
	t is greater than 300 mm Ungrouted or Grouted	Use of the FRP is not recommended
Solid Unit Masonry Walls	Single wythe walls with t=100 mm or less	FRP on one face of the wall is acceptable
	Double wythe walls with t=200 mm or less	FRP on two faces of the wall is required
	Multi-wythe walls with t greater than 200 mm or less	Use of the FRP is not recommended

The nominal value of $V_n$is computed using equation (14):

$$V_n = V_n^{URM} + V_f \tag{14}$$

Where V_f is the nominal shear strength of FRP and it is given by equation (15):

$$V_f = \begin{cases} p_{fv} w_f \frac{d_v}{s_f} & \text{for FRP Laminate} \\ p_{fv} \frac{d_v}{s_f} & \text{for NSM FRP} \end{cases} \tag{15}$$

Where w_f is the width of the FRP laminate, d_v is the effective masonry depth and taken as the minimum of the length or height of the wall, s_f is the center to center spacing between each strip, p_{fv} is given by equation (16):

$$p_{fv} = \begin{cases} n t_f f_{fe} \leq 260 \frac{N}{mm} & \text{For FRP Laminate} \\ A_{f,bar} f_{fe} \leq 44500 \frac{N}{mm} & \text{for NSM FRP} \end{cases} \tag{16}$$

Where, n is the number of piles in the FRP laminate, t_f is the nominal thickness of one ply of FRP.

For the flexural strength of the FRP, Ø is taken to be 0.6. Similar to the out-of-plane behaviour, the factored moment should not exceed nominal capacity for the section multiplied by the reduction factor (Refer to Eq. (10)).

Figure 4 shows a typical wall reinforced with Fibres. Assuming that the value of the axial force acting at the centroid of the wall (at L/2, where L is the length of the wall), the value of M_n can be computed as shown in equation (17).

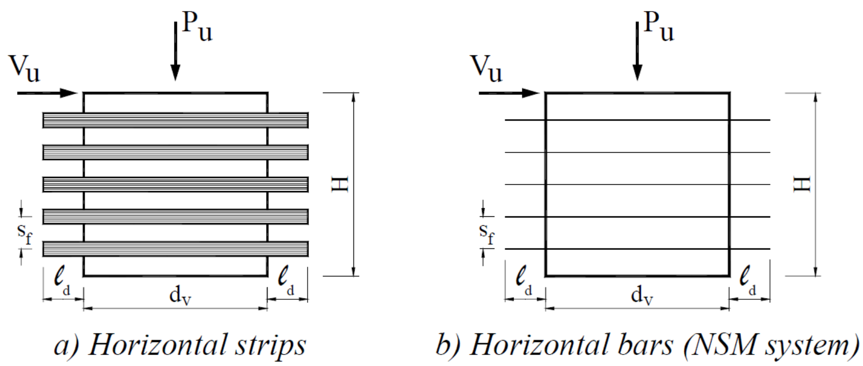


Figure (4): FRP strengthening for shear controlled walls

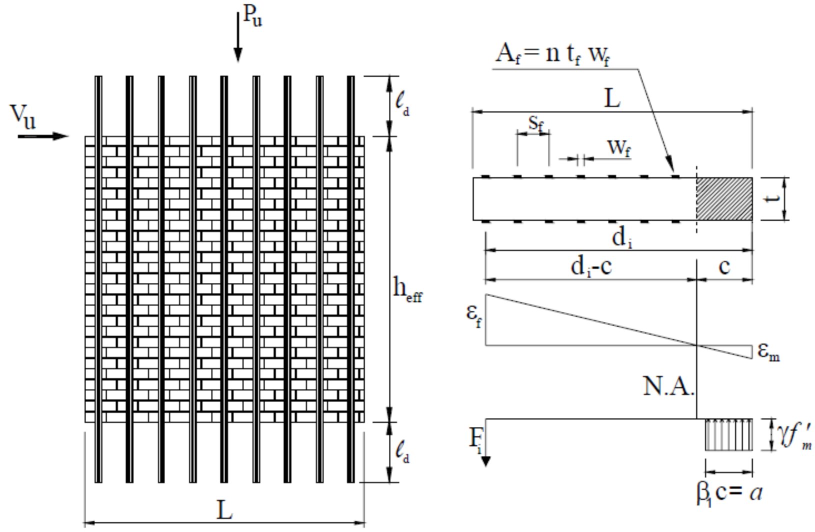


Figure (5): FRP strengthening for flexural

$$M_n = \sum_i F_i \left(d_i - \frac{\beta_1 c}{2}\right) + P_u \left(\frac{L}{2} - \frac{\beta_1 c}{2}\right) \tag{17}$$

Where F_i is the force acting on the i-th FRP strip located at distance d_i from the extreme compressive face (refer to Figure 5). In this case the value of V_n can be computed from the following equation (Eq. (18)).

$$V_n = \frac{M_n}{kh_{eff}} \tag{18}$$

Where k is the coefficient that reflects the end conditions of the wall (equal to 0.5 for fixed-fixed condition and 1 for fixed free wall), h_{eff} is the effective wall height.

CSA S806-02, ISSI-08 and CSA S304.1-04 Design Approach

These standards are based on limit states design principles and are consistent with The National Building Code of Canada (NBCC). Building components should be designed in such a way that factored resistance is greater than factored loads which are in accordance with NBCC.

The mechanical properties of FRPs are dependent on such factors as fibres types, fibre volume fraction, type of polymer, and manufacturing process. FRP characteristics, in turn, are product-dependent and should be obtained from manufacturer data sheets or by testing which the instruction is provided in S806. If the information was not available, clause 7 provides properties of FRP components. Due to the environmental exposure and variability in material properties, the reduction factor is applied to the material. Clause 7.1.6.2 assigns the factor, ϕ_{frp}, of 0.75 for both CFRP and AFRP reinforcement and 0.3 for GFRP because of stress corrosion.

Strength and stiffness of FRP in compression is lower than the tensile properties. The compressive strength of CFRP, GFRP, and AFRP might be taken as 50%, 30%, and 10% of their tensile strength, respectively. However, available design guidelines on FRP-reinforced concrete strongly recommend that the use of FRPs as compression reinforcement should be avoided. The main difference between steel and FRPs in behaviour is that steel has a plateau which exhibits plastic deformation while the response of FRPs is linear up to failure. This major disparity has a significant impact on the flexural and shear design.

Masonry code (S304.1-04) adopted the under-reinforced section design philosophy to ensure that steel yield before masonry crushes. The linear elastic response of FRPs leads to sudden failure in both tension and compression. However, due to plastic deformation of masonry concrete in compression, it provides some warning which is more desirable. Since the modulus of elasticity of FRPs is typically lower than that of steel, serviceability is usually the major requirement to control the design factor.

Flexural Resistance:

When masonry and FRP reach their ultimate strains at the same time, the balance condition happens. In this regards, the balanced failure reinforcement ratio can be evaluated from strain compatibility and equilibrium of forces as follows:

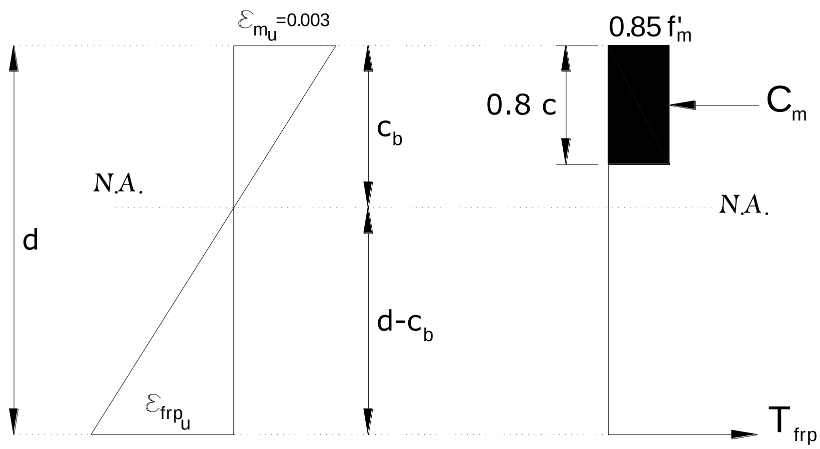


Figure (6): Strain distribution and stress distribution on the wall

$$c_b = d\left(\frac{\varepsilon_{m_u}}{\varepsilon_{m_u}+\varepsilon_{frp_u}}\right) \tag{19}$$

$$C_m = T_{frp} \tag{20}$$

$$0.85\phi_m \chi f_m' \beta_1 c_b b = \phi_{frp} A_{frp_b} f_{frp_u} \tag{21}$$

$$\rho_{frp_b} = \frac{A_{frp_b}}{bd} = 0.85\beta_1 \chi \frac{\phi_m f_m'}{\phi_{frp} f_{frp_u}}\left(\frac{\varepsilon_{m_u}}{\varepsilon_{m_u}+\varepsilon_{frp_u}}\right) \tag{22}$$

When the reinforcement ratio is lower than balanced, failure happens by rupture of FRP otherwise under-reinforced ratio will fail by masonry crushing.

$$\rho_{frp_{min}} \rightarrow \; M_r > 1.5M_{cr} \tag{23}$$

$$\rho_{frp_{max}} = \; 0.2\frac{0.85 f_m'}{f_{frp_s}} \tag{24}$$

$$f_{frp_s} = E_{frp}\varepsilon_{frp_s} \tag{25}$$

$$\varepsilon_{frp_s} = 0.002 \tag{26}$$

Shear Resistance:

There are two possible shear failure modes: rupture of FRPs due to reaching their tensile capacity (shear-tension) and crushing of masonry (shear-compression). Mode of failure changes from shear tension to shear compression by increasing the shear reinforcement which results in large deflections and is more desirable.

Similar to steel reinforcement, the factored shear resistance of FRP-strengthened masonry beams is given by

$$V_r = V_m + V_{frp} \tag{27}$$

Where, V_m = factored shear resistance of masonry as defined in S304.1

V_{frp} = factored shear resistance of FRP as given in S806-02.

$$V_{frp} = 0.4\left(\frac{\phi_{frp}A_{frp_V}f_{frp_u}d}{s}\right) < 0.36\phi_m\sqrt{f_m'}b_w d \tag{28}$$

$$A_{frp_{min}} = \frac{0.35b_w s}{0.4f_{frp_u}} \tag{29}$$

Sample Example

Simple examples are used to illustrate the design of URM with FRP using different ways of mounting the Fibres (Laminate or NSM).

Example 1

An URM wall of 4.0 m height, 4.0 m long and 190 mm thick, full grouted. This wall is constructed using 20 MPa concrete units and type S mortar. The applied loads are 200 kN/m dead load, 50 kN/m live load. The wall is also subjected to wind pressure of 3 kPa. It is required to design this wall using FRP. In this example, the analysis was carried out using the CSA S806-02 and ACI 440-02. For the sake of comparison, an analysis was also carried out using ordinary steel.

Check slenderness:

Although the eccentricity is zero, S304.1 requires that a minimum values of 0.1t=19 mm be used. For hinged-hinged condition, k=1.

$$\frac{kh}{t} = \frac{1.0(4000)}{190} = 21.1 > \left(10 - \frac{3.5e_1}{e_2}\right) = 6.5 < 30$$

Therefore, slenderness effects should be considered. Here, the moment magnifier method has been used. There are three types of load applied to this wall, namely, dead, live and wind loads. The various combinations have to be considered.

a) Principal load: dead plus live

U=1.25D+1.25L+0.4W

The factored axial and wind loads are

$P_f = 1.25D + 1.5L = 1.25(200) + 1.5(50) = 325\ \frac{kN}{m}$

$w_f = 0.4\ (3) = 1.2\ \frac{kn}{m^2}$

And the primary moment is

$$M_{fp} = P_f e + \frac{w_f h^2}{8} = 325(10^3)(19) + \frac{1.2(4000)^2}{8} = 8.575\ kN.m/m$$

b) Principal load: dead plus wind

U=1.25D+1.4W+0.5L

$P_f = 1.25D + 0.5L = 1.25(200) + 0.5(50) = 275\ \frac{kN}{m}$

$w_f = 1.4\ (3) = 4.2\ \frac{kn}{m^2}$

$$M_{fp} = P_f e + \frac{w_f h^2}{8} = 275(10^3)(19) + \frac{4.2(4000)^2}{8} = 13.625\ \ kN.m/m$$

A wall section must now be chosen to carry the worst of

1) $P_f = 325\ \frac{kN}{m}$ $\quad and \quad M_f = 8.575\ \frac{kN.m}{m}$
2) $P_f = 275\ \frac{kN}{m}$ $\quad and \quad M_f = 13.625\ \frac{kN.m}{m}$

For 20 MPa block and type S mortar, $f_m' = 10\ MPa\ (solid)\ and\ f_t = 0.4\ MPa$

$A_e = 190(10^3)\,mm^2/m,\ S_x = 6.02(10^6)\,mm^3/m, I_x = 572(10^6)\,mm^4/m$

Recall that, $M_{fTOT} = \frac{M_{fp}C_m}{1-\frac{P_f}{P_{cr}}}$ where, $P_{cr} = \frac{\pi^2 EI}{(kh)^2}$ $\quad and \quad EI = \phi_e E_m I_{eff}/(1 + 0.5\beta_d)$

$\phi_e = 0.65$

$E_m = 850 f_m' = (850)(10) = 8500\ MPa\ < 20000\ MPa$

$I_{eff} = 0.4 I_o = 0.4(572)(10^6) = 228.8(10^6)\,mm^4/m$

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} = 0.6 + 0.4 = 1$$

Case 1:

$$\beta_d = \frac{factored\ dead\ load}{factored\ total\ load} = \frac{1.25(200)}{325} = 0.769$$

$$EI = \frac{0.65(8500)(228.8)(10^6)}{1 + 0.5(0.769)} = 913(10^9) \quad N.mm^2/m$$

$$P_{cr} = \frac{\pi^2 (913)(10^9)}{(4000)^2} = 563 \;\; kN/m$$

$$M_{fTOT} = \frac{8.575}{1 - \frac{325}{563}} = 20.28 \;\; kN.m/m$$

Check resistance:

$$\frac{P_f}{A_e} + \frac{M_{fTOT}}{S_x} \leq \phi_m f_m^{'}$$

$$\frac{325(10^3)}{190(10^3)} + \frac{20.28(10^6)}{6.02(10^6)} = 5.08 \leq 6.0 \; MPa$$

For tensile stress check:

$$P_f = 0.9D + 1.25L = 242.5 \; kN/m \quad \rightarrow \quad M_{fTOT} = 13.66 \;\; kN.m/m$$

$$-\frac{P_f}{A_e} + \frac{M_{fTOT}}{S_x} = -\frac{242.5(10^3)}{190(10^3)} + \frac{20.28(10^6)}{6.02(10^6)} = 2.092 \; MPa \nleq \phi_m f_t = 0.24$$

Case 2:

$$\beta_d = \frac{1.25(200)}{275} = 0.909$$

$$EI = \frac{0.65(8500)(228.8)(10^6)}{1 + 0.5(0.909)} = 869(10^9) \quad N.mm^2/m$$

$$P_{cr} = \frac{\pi^2 (869)(10^9)}{(4000)^2} = 536 \;\; kN/m$$

$$M_{fTOT} = \frac{13.625}{1 - \frac{275}{536}} = 27.98 \;\; kN.m/m$$

Check resistance:

$$\frac{P_f}{A_e} + \frac{M_{fTOT}}{S_x} = \frac{275(10^3)}{190(10^3)} + \frac{27.98(10^6)}{6.02(10^6)} = 6.1 \; MPa \nleq \phi_m f_m^{'} = 6.0 \; MPa$$

For tensile stress check: $\quad P_f = 0.9D + 0.5L = 205 \; kN/m \quad \rightarrow \quad M_{fTOT} = 22.06 \;\; kN.m/m$

$$-\frac{P_f}{A_e} + \frac{M_{fTOT}}{S_x} = -\frac{205(10^3)}{190(10^3)} + \frac{22.06(10^6)}{6.02(10^6)} = 2.59 \; MPa \nleq \phi_m f_t = 0.24$$

Therefore, the wall cannot sustain the load combination and strengthening is required. An external FRP strengthening system of CFRP strips is to be applied vertically to both sides of the wall since out-of-plane moment acts in either direction. The effect of CFRP reinforcement on the value of I_{eff} is minimal and will be neglected. The properties of the CFRP strips are as follows:

$f_{frp_u} = 2250\ MPa$

$E_{frp} = 150\ GPa$

$\varepsilon_{frp_u} = 0.015$

$\phi_{frp} = 0.75$

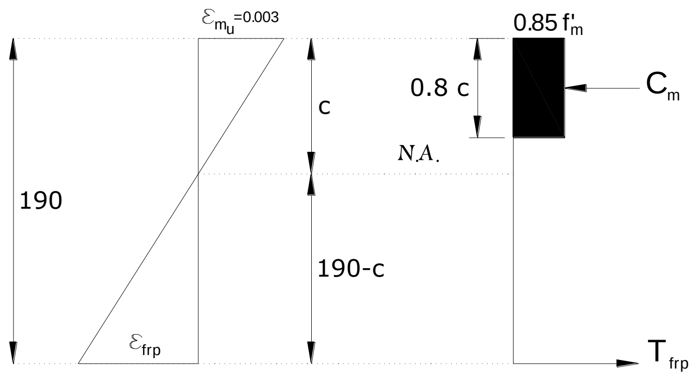


Figure (7): Strain and stress distribution on the wall

$C_m = 0.85\phi_m \chi f_m^{'} b \beta_1 c = 0.85(0.6)(1.0)(10)(1000)(0.8c) = 4080c\ \ N/m$

From the strain distribution shown in figure

$$\varepsilon_{frp} = 0.003\left(\frac{190-c}{c}\right)$$

$$T_{frp} = \phi_{frp} A_{frp} E_{frp} \varepsilon_{frp} = 0.75 A_{frp}(150000)(0.003)\left(\frac{190-c}{c}\right) = 337.5 A_{frp}\left(\frac{190-c}{c}\right)\ \ N/m$$

From equilibrium

$$P_r = C_m - T_{frp}$$

$$205(10^3) = 4080c - 337.5 A_{frp}\left(\frac{190-c}{c}\right)$$

Taking the moment about the centroid of the wall cross-section

$$M_r = C_m\left(\frac{t}{2} - \frac{\beta_1 c}{2}\right) + T_{frp}\left(\frac{t}{2}\right) = C_m(95 - 0.4c) + T_{frp}(95)$$

$$22.06(10^6) = 4080c(95 - 0.4c) + 32062.5\ A_{frp}\left(\frac{190-c}{c}\right)$$

$c = 61.55\ mm$

$\varepsilon_{frp} = 0.0063 < \varepsilon_{frp_u} = 0.015$

$A_{frp} = 65.48\ \ mm^2/m$

Therefore, use 45.0 x 1.5 mm CFRP spaced every meter ($A_{frp} = 67.5 \quad mm^2/m$)

Designing using the ACI 440-02:

Material properties will be different. The calculation for different material properties is calculated as shown:

$f_{fu} = C_E f_{fu}^* = 0.65 \times 2250 = 1462.5\ MPa$ and $\varepsilon_{fu} = 0.65 \times 0.015 = 9.75 \times 10^{-3}$

$f_{fe} = K_m f_{fu}^* = 0.45 \times 2250 = 1012.5\ MPa$ and $\varepsilon_{fe} = 0.45 \times 0.015 = 6.75 \times 10^{-3}$

$P_r = C_m - T_{frp}$

Assume that the failure is due to debonding of steel

$C_m = \gamma f_m^{'} b \beta_1 c = 0.7(10)(1000)(0.7c) = 4900c\ \ N/m$

$205(10^3) = 4900c - 1012.5 \times A_{frp}$

$$A_{frp} = \frac{4900c - 205(10^3)}{1012.5}$$

Taking the moment about the centroid of the wall cross-section

$$M_r = C_m\left(\frac{t}{2} - \frac{\beta_1 c}{2}\right) + T_{frp}\left(\frac{t}{2}\right) = C_m(95 - 0.4c) + T_{frp}(95)$$

$22.06(10^6) = 4900c(95 - 0.4c) + (4900c - 205(10^3)) \times 95$

$c = 49.84\ mm$

$\varepsilon_{frp} = 0.00675\left(\frac{49.84}{190-49.84}\right) = 0.0024 < \varepsilon_m^{`} = 0.003$.... ok

$A_{frp} = 38.74\ \ mm^2/m$

Therefore, use 40.0 x 1 mm CFRP spaced every 600 mm ($A_{frp} = 66.67 \quad mm^2/m$)

When designing using the conventional steel reinforcement. The amount of reinforcement needed was M10@600 ($A_{frp} = 166.67 \quad mm^2/m$).

Table (2) summarizes the results calculated using the different design approach. The labour cost was calculated based on the rates presented here in Edmonton (for a skilled worker the rate is 35 CAD/m^2, for a non-skilled worker 14.16 CAD/m^2). The Cost of the Reinforcement steel reinforcement is 650 Cad/ton. The price of the CFRP is given by one 22 CAD/kg (including the Resin price).

Table (2)*: Summary of the wall result using different design alternatives:

	Steel Bars	CFRP Laminates (CSA-S806)	CFRP Laminates (ACI 440)
Area of Reinf.	M10@600	45×1.5/m'	1.67×40×1.0/m'
M_r (kN.m/m)	27.23	22.78	37.96
Weight (kg)	21.1	1.92	2.4
Mat. Cost	14	42.3	52.92
Labour Cost	560	226.	226
Total Cost	574	268.3	279
Cost/M_r	27.2	11.78	7.34

*All the prices given in the table are in CAD

Designing using the CSA S806-02 is more conservative than the design using ACI 440. Although the cost of the CFRP material is much more expensive than the conventional steel, but the labour cost is less (installation of fibres do not need skilled labours to do the installation). The use of FRP provides better cost per strength. Moreover, repair work using CFRP will not take the same time when it is done with other material as steel.

Example 2:

An URM Shear wall is 2.4 m long, 4.0 m high and 190 mm full grouted thick constructed using 20 MPa concrete units and type S mortar. The applied load is: dead load of 250 kN. Under the recent seismic provisions, the wall has to sustain an in-plane lateral load of 60 kN at the top. It is required to check the adequacy of the wall to resist these forces. If the wall needs to be retrofitted, use the CSA S806-02 approach in the repair.

Solution:

a) Factored load:

U=1.0D+1.0E

$P_{f1} = 1.0D = 1.0(250) = 250\ kN$

$P_{f2} = 0.9D = 0.9(250) = 225\ kN$

$V_f = 1.0V = 1.0(60) = 60\ kN$

$M_f = 1.0M = 1.0(60)(4.0) = 240\ kN.m$

For 20 MPa block and type S mortar, $f_m' = 10\ MPa\ (solid)\ and\ f_t = 0.4\ MPa$

$A_e = (190)(2400) = 456(10^3)\ \ mm^2$

$S_y = tl_w^2/6 = 182.4(10^6)\ mm^3$

b) Axial load and in-plane bending:

When compression controls, use the higher P_{f1} and when tension controls, use the lower P_{f2}.

$$\frac{P_{f1}}{A_e}+\frac{M_{fy}}{S_y}=\frac{250(10^3)}{456(10^3)}+\frac{240(10^6)}{182.4(10^6)}=1.86\leq\phi_m f_m^{'}=6.0\ MPa$$

$$-\frac{P_{f2}}{A_e}+\frac{M_{fy}}{S_y}=-\frac{225(10^3)}{456(10^3)}+\frac{240(10^6)}{182.4(10^6)}=0.82\ MPa\nleq\phi_m f_t=0.24$$

Therefore, the wall is not adequate and retrofitting is required.

c) Diagonal tension shear:

$$\frac{M_f}{V_f d_v}=\frac{240}{60(4)}=1\ \rightarrow\ v_m=0.16\left(2-\frac{M_f}{V_f d_v}\right)\sqrt{f_m^{'}}=0.16(2-1)\sqrt{10}=0.506\ \ MPa$$

$$V_r=\phi_m(v_m b_w d_v+0.25P_d)\gamma_g=0.6[(500)(0.19)(4.0)+0.25(225)]=261.75\ \ kN$$

$$V_{rmax}=0.4\phi_m\sqrt{f_m^{'}}b_w d_v\gamma_g[2-(h_w/l_w)]=0.4(0.6)\sqrt{10}(0.19)(4.0)(1.0)(2-1)(10^3)=576.8\ \ kN>V_r$$

$$\therefore\ V_r=261.75\ kN\ >V_f=60\ kN$$

d) Sliding shear between masonry courses:

$$V_r=0.16\phi_m\sqrt{f_m^{'}}A_{uc}+\phi_m\mu P_1=0.16(0.6)\sqrt{10}(456)+0.6(1.0)(250)=288.4\ kN>60\ kN$$

e) Sliding shear between masonry and the support:

$$V_r=\phi_m\mu C=(0.6)(0.7)(250)=105\ kN>60\ kN$$

The wall shear resistance is higher than the factored shear force and no shear strengthening is needed.

Finally, the wall cannot sustain the extra load and strengthening is required. A NSM FRP strengthening system of CFRP strips is to be applied vertically to the wall. The effect of CFRP reinforcement on the value of I_{eff} is minimal and will be neglected.

The properties of the CFRP strips are as follows:

$$f_{frp_u}=2250\ MPa$$

$$E_{frp}=150\ GPa$$

$$\varepsilon_{frp_u}=0.015$$

$$\phi_{frp}=0.75$$

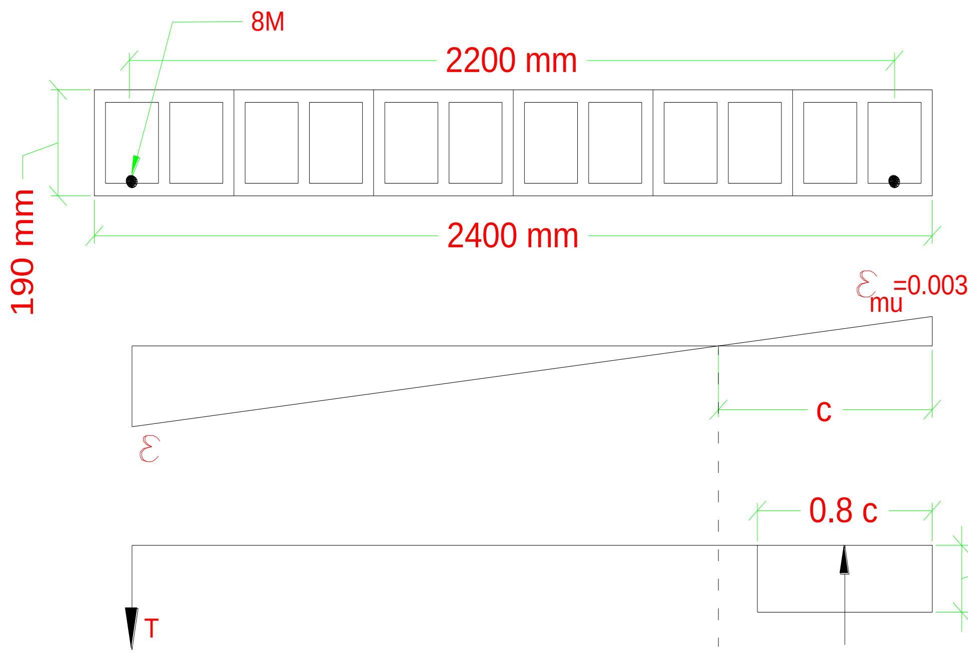


Figure (8): Wall Section, strain distribution and stress distribution

To determine initial vertical reinforcement area, the axial load is ignored for the moment and FRP reinforcement is estimated from pure bending. For uniform vertical FRP, a moment of 2/3 the wall length is assumed.

$$M_f = \phi_{frp} A_{frp} f_{frp} (0.67 l_w) \rightarrow A_s = \frac{240(10^6)}{0.75(2250)(0.67)(2400)} = 88.5 \ mm^2$$

This reinforcement is located only in one side of the wall and the total required area is 177 mm^2. Try 2-8M bars arranged as shown in Figure 8, A_{frp}=100 mm^2.

$$C_m = 0.85 \phi_m \chi f_m^{'} b \beta_1 c = 0.85(0.6)(1.0)(10)(190)(0.8c) = 775.2c \ N$$

From the strain distribution shown in figure 8:

$$T_{frp} = \phi_{frp} A_{frp} E_{frp} \varepsilon_{frp}$$

$$\varepsilon_{frp} = 0.003 \left(\frac{2200 - c}{c} \right) \rightarrow T = (0.75) A (150000)(0.003) \left(\frac{2200 - c}{c} \right) = 337.5A \left(\frac{2200 - c}{c} \right)$$

From equilibrium.

$$P_f = C_m - T \rightarrow 250(10^3) = 775.2c - 337.5A \left(\frac{2200 - c}{c} \right)$$

$$250(10^3) = 775.2c - 16964.6 \left(\frac{2200 - c}{c} \right) \rightarrow c = 416 \ mm$$

$$c = 416 \ mm \rightarrow \begin{cases} T = 72.7 & kN \\ C_m = 322.7 & kN \end{cases}$$

Taking the moment about the centroid of the wall cross-section

$M_r = T(d - l_w/2) + C_m(l_w/2 - 0.4c)$

$\therefore\ M_r = \ 386.7\ kN.m\ >\ M_f = 240\ kN.m$

Therefore, use 2-8mm CFRP rods @ 2200 mm spacing.

Blast Resistance of FRP-Strengthened Masonry

In recent years, the bombing and terrorist attacks have been experienced around the world, which caused the catastrophic loss of innocent people's lives. The ever increasing bombing events which directly affects the social and economical aspects of people's lives, emphasizes the urgent need for strengthening of our civilian structures that are mostly vulnerable against explosives. Lateral pressure that is resulted from blast loads introduces out of plane loading on the structural walls. Recent experiences have illustrated that the failure and fragmentation of Unreinforced Masonry Walls due to their low out of plane resistance are one of the major causes of loss in lives and property.

Compared to the publicly available literature on the different applications of FRP in masonry, the number of analytical and/or experimental researches that have been conducted on the blast resistance of FRP strengthened masonry are very few (Mayers et al. (2004), El-Domiaty et al. (2004), Carney et al. (2003), Muszynski et al. (2003), Tan and Patoary (2009)). The main reason behind this is that investigating the behaviour of structure under blast loading needs explosive testing that would be very expensive and because of the complexity and dynamic nature of the response in a very short time period the real behaviour and failure mode of the structure is difficult to capture. The response of the structure under blast loads should be measured using modern data acquisition system. To investigate the effects of explosion on the masonry wall, it is essential to define fundamental parameters through which the resulting blast pressure can be calculated. Charge weight (Q) and standoff distance (R) are the most important parameters in determining the peak blast pressure resulting from an explosion. The first parameter is the weight of explosive material and the latter is the distance between the source of explosion and the masonry wall that experiences the blast pressure. The peak pressure resulting from the chosen charge weight and standoff distance can be calculated using some empirical equations presented by the United States Army technical manual TM5-855-1 and the Defence Atomic Support Agency (DASA) Report # 1860 (1966). The following equation can be used to determine the peak blast pressure on the masonry wall for chosen TNT charge weight and specified standoff distance.

$$P_{so} = 150\ Q\left(\frac{5}{R}\right)^3 \qquad (30)$$

Where,

P_{so} : Peak pressure at given charge weight and standoff distance (Psi).

Q: Charge weight of TNT in (lb).

R: Radial standoff distance from the center of the explosive to wall (ft).

Blast time history and peak pressure can be obtained using the charts and equations developed by U.S. Department of Army (1986). The created time history is simplified to an equivalent triangular without rise time using the pulse duration (t_d) and the obtained peak pressure. The most common method to determine the dynamic response of masonry walls under blast loading is a fundamental analytical approach using single degree of freedom (SDOF) model. For the aim of analysis and design of FRP strengthened masonry walls the equivalent single degree of freedom model is created using equivalent mass (M_E) and equivalent stiffness (K_E). Then the Natural period of vibration of the equivalent system can be simply calculated using the following equation.

$$T_n = \frac{2\pi}{w} = 2\pi\sqrt{\frac{M_E}{K_E}} = 2\pi\sqrt{\frac{K_{LM}M}{K_E}} \quad (31)$$

Where M is the actual mass of the wall, K_E is the equivalent stiffness of the wall and K_{LM} is the load mass factor can be calculated for different kinds of support condition using Army TM 5-1300 report charts. Knowing the fundamental period of the equivalent system and the equivalent triangular pulse duration (t_d) the Dynamic Load Factor (DLF) can be read from the presented graph in Figure 9. Finally, the equivalent static load (R_s) can be obtained using the following equation which is used for the design purposes. P is the peak load (or pressure) resulting from the blast.

$$DLF = \frac{R_s}{P} \quad (32)$$

It is well worth mentioning that the strain rate effect is an important factor for the design consideration under blast loading. The response of the materials under blast load which is rapidly applied on the wall is different from the static applied load. Increased resistance of material is seen when the load is rapidly applied and the strain rate is fast. So the Dynamic Increase Factor (DIF) is applied to static strength values in design calculations. According to the recommendation of Blast Design Manual (Army TM 5-1300) it is reasonable to apply the enhancement factor of 1.19 to account for the blast effects on walls under flexural action. The calculated equivalent static force is used for the design of FRP-Strengthened Masonry wall. The wall can be designed equating equivalent static load (R_s) to ultimate static resistance (R_u) of the strengthened wall times 1.19.

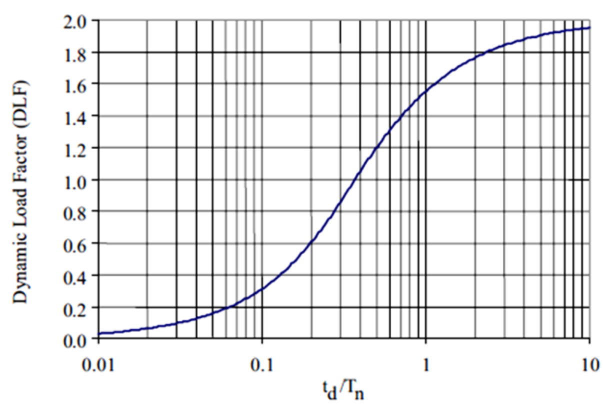


Figure (9): Dynamic Load Factor for the Equivalent

Triangular Pulse (Tan and Patoary (2009))

Knowing the required resistance under selected blast loading (equivalent static load), the required amount of FRP can be designed for the strengthening of wall sections.

Case Study

The best way to understand the various applications and uses of FRP is to examine the various case studies. FRP retrofit of the Energy Efficient Network Building located in Michigan is investigated in this paper. This case study does not explore all the uses and applications of carbon fibre reinforcement, but rather, gives a general view of the most common applications. The East wall of the garage is an exterior wall and the ground slopes on the outside towards south by some eight feet. The wall footing is at grade level, and the ladder cracks in the wall indicate that the wall is settling towards the South. The East and the South walls are both unreinforced block masonry and retain soil pressure from backfill for the garage floor on the inside. The walls needed to be reinforced. The NSM technique using CFRP was chosen to retrofit the walls from the outside.

Figure 10 shows the removal of wood waller of the failed tie-back system by grinding and cutting off the steel bolts.

Figure (10): Removal of the wood waller

In Figure 11, a little further back at the garage wall, a technician is grinding off the bolts holding the waller of the failed tie-back system. Figure 12 shows the South-East corner of the garage wall. This is where the technician opted to go in for drilled-piers diagonally across the corner of the wall footing under which he wants to insert a needle beam that would rest on the drilled piers to stabilize the wall footing at the corner.

Figure (11): Removal of the wood waller

Figure (12): Technique used to stabilize the wall footing

Figure 13 shows the location of the reinforcement and the surface preparation for applying the reinforcement. Another shot of the surface of the South wall is being prepared for application of the reinforcement (Figure 14).

Figure (13): Location of the Required REINF.

Figure (14): Surface preparation

Surface preparation was achieved by grinding the surface of the block masonry on the East wall. (Figure 15).

Figure (15): Surface preparation of the wall

After preparing the wall surface, the technician is applying epoxy on a strip of the CFRP for mounting it to the surface locations. The walls have been toughly cleaned by grinding off the surface and blowing off all the dust and debris (Refer to Figure 16). Figure 17 shows the application of the CFRP to the West-End of the South Wall.

Conclusion and Recommendations.

The use of FRPs to improve the behaviour of an unreinforced masonry wall has been investigated in this paper. They are used to increase the capacity of URM walls when subjected to additional loads than what they are designed for. The durability of these materials depends on many factors starting from storing and handling of these materials. Coating is required for these materials to protect them from aggressive environment and to increase their durability.

Figure (16): Application of the epoxy paint to the CFRP laiminate

a) b)

c)

Figure 17: Mounting the CFRP strip to the wall.

Different design standards dealt with the design of URM reinforced these materials. The design is performed according to the desired mode of failure. Simple examples were used to illustrate the design procedure using the FRP.

Recommendations regarding the use of FRP for retrofitting are:

- The use of FRP for retrofitting walls that are subjected to direct sunlight is not preferable unless proper isolation for the harmful environment is introduced.
- Fire protection of the FRPs can be achieved by isolating them using paints or increasing the cementitious cover.
- Prior to using these materials to the surface of the walls, the surface shall be levelled and free from fine materials and cracks.

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List of Symbols

A_f, A_{frp}: The cross-sectional area of the FRP reinforcement.
A_n: The Area of the net mortared/grouted section.
b: Width of the wall.
c: The position of the Neutral axis from the compression fibre in the balanced case.
C_E: Environmental reduction factor.
C_m: Compression force in the FRP block.
d, d_f: The depth till the FRP centroid.
d_v: The effective masonry depth.
F_i: The force acting on the i-th FRP.
E_f: Modulus of Elasticity of the FRP.
E_{frp}: Modulus of Elasticity of the FRP.
f_{fe}, f_{frp}: The effective stress in the FRP.
f_{fu}: Design ultimate tensile strength of the FRP.
f_{fu}^*: Ultimate tensile strength for the FRP as reported by the manufacturer.
$f_m^`$= The masonry compressive strength.
h_{eff}: The effective wall height.
k: Coefficient for the end conditions of the wall.
K_m: The bond reduction coefficient.
K_v: The bond reduction coefficient for shear controlled failure mode.
M_n: Nominal flexural strength.
M_u: The factored Flexural moment.
n: The number of piles in the FRP laminate.
s_f: The center to center spacing between FRP strips.
t= The depth of the wall.
T_d: The decomposition temperature.
t_f: The nominal thickness of one ply of FRP.
T_{frp}: Tension force in the FRP.
T_g : Glass transition temperature.
V_{bjs}: The nominal lateral strength corresponding to joint sliding.
V_{dt}: The nominal lateral strength corresponding to the diagonal tension.
V_f, V_{frp}: The nominal shear strength of FRP.
V_m: Shear resisted by the masonry.
V_n^{URM}: The nominal shear capacity for the unreinforced masonry wall.
V_r: The shear resistance of the wall.
V_{tc}: The nominal lateral strength corresponding to toe crushing.
V_u: The Ultimate shear force.
w_f: The width of the FRP laminate.
$\beta_1 c$: The depth of the masonry compression block.
ε_{fe}: The effective strain in the FRP.
ε_{fu}: Design ultimate tensile strain in the FRP.
ε_{fu}^*: Ultimate tensile strain for the FRP as reported by the manufacturer.

ε_{frp}: Ultimate strain in the FRP.
ε_{m_u}: Ultimate strain in the masonry.
ω_f: The FRP reinforcement index.
χ: Factor for the compression force direction with respect to the bed joint.
$\emptyset$: The strength reduction factor.
ϕ_m: Masonry strength reduction factor.
ϕ_{frp}: FRP strength reduction factor.
ρ_{frp}: The FRP reinforcement ratio.