Comparison Between Masonry Codes Used in Different Countries

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TABLE OF CONTENTS

CHAPTER **PAGE**

LIST OF TABLES

LIST OF FIGURES

FIGURE PAGE

LIST OF ABBREVIATIONS

A	Area
A_n	Net cross sectional area of member
A_{nv}	Net shear area
A_s	Area of reinforcement in tension
A_{st}	Area of reinforcing steel
a	Depth of an equivalent compression stress block at nominal strength (mm)
b	Width of a section
C_1	Amplification factor for dowel rod action and wall aspect ratio
C_2	Amplification factor for dowel rod action and wall aspect ratio
C_3	Factor based on the type of masonry structure
c	Distance from the fiber of maximum compressive strain to the neutral axis (mm)
D	Dead load or related internal moments or forces
d	Distance from extreme compression fiber to centroid of tension reinforcement
d_v	Actual depth of a member in direction of shear considered
E	Load effects of earthquake or related internal moments or forces
E_m	Modulus of elasticity of masonry in compression
E_s	Modulus of elasticity of steel
E_v	Modulus of rigidity (shear modulus) of masonry
e	Eccentricity
F_a	Allowable compressive stress available to resist axial load only
F_b	Allowable compressive stress available to resist flexure only

F_s Allowable tensile or compressive stress in reinforcement

F_t Allowable flexural tensile stresses of masonry

F_v Allowable shear stress

F_{vm} Allowable shear stress resisted by the masonry

F_{vs} Allowable shear stress resisted by the shear reinforcement

f_{xd1} Design flexural strength of masonry having the plane of failure parallel to the bed joints

$f_{xd1,app}$ Apparent design flexural strength of masonry having the plane of failure parallel to the bed joints

f_a Calculated compressive stress in masonry due to axial load only

f_b Calculated compressive stress in masonry due to flexure only

f_b Normalized mean compressive strength of the units in the direction of the applied load in (N/mm2)

f_c Permissible compressive stress

f_d Compressive stress due to dead loads (IS)

f_d Design compressive strength of masonry (EC6)

f_k Characteristic Compressive Strength of Masonry

f'_m Compressive Strength of Masonry

f_m Compressive strength of the mortar

f_r Modulus of rupture

f_s Permissible shear stress

f_{vd} Design value of shear strength

f_y Specified yield strength of steel for reinforcement and anchors

f_{yd}	Design strength of reinforcing steel
f_{xd}	Design flexural strength appropriate to the plane of bending
h	Effective height of column, wall, or pilaster
I_n	Moment of inertia of net cross-sectional area of a member
K	Constant based on type of masonry unit and Group
k_a	Area reduction factor
k_c	Coefficient of creep of masonry per MPa
k_e	Coefficient of irreversible moisture expansion of clay masonry
k_m	Coefficient of shrinkage of concrete masonry
k_p	Shape modification factor
k_s	Stress reduction factor
k_t	Coefficient of thermal expansion of masonry per degree Celsius
L	Live load or related internal moments or forces
l	Length of wall
l_c	Length of the compressed part of the wall, ignoring any section in tension
L_w	Horizontal length of a wall
M	Maximum moment at the section under consideration
M_{ad}	Additional design moment
M_u	Factored moment
M_{Rd}	Design value of the moment of resistance
M_{Ed}	Design value of the moment applied

N_v Compressive force acting normal to shear surface

N_{Rd} Design value of the vertical resistance of a masonry wall or column

N_{Ed} Design value of the vertical load

N_{nw} Nominal axial load-carrying capacity of bearing wall

N_u Factored compressive force acting normal to shear surface

P Axial load

P_e Euler buckling load

P_n Nominal axial strength

P_u Factored axial load

Q First moment about the neutral axis of an area between the extreme fiber and the plane at which the shear stress is being calculated

R Rain load or related internal moments or forces

r Radius of gyration

S Snow load or related internal moments or forces

S_n Section modulus of the net cross-sectional area of a member

t Thickness

V Shear force

V_{Ed} Design value of shear load

V_{Rd} Design value of shear resistance

V_{nm} Nominal shear strength provided by masonry

V_{ns} Nominal shear strength provided by shear reinforcement

v_g Maximum permitted type-dependent total shear stress

v_m Maximum permitted grade-dependent shear stress

v_n Total shear stress corresponding to nominal shear

v_p Shear strength provided by axial load

v_s Shear strength provided by shear reinforcement

W Wind load or related internal moments or forces

W_{Ed} Design lateral load per unit area

X_m Standard deviation of strength of masonry strength

Z Elastic section modulus of a unit height or length of the wall

α Constant based on the type of mortar used

α_t Coefficient of thermal expansion of masonry

$\alpha_{1,2}$ Bending moment coefficients

β Constant based on the type of mortar used

γ_g Grouted shear wall factor

γ_m Partial factor based on type of material

Φ Strength reduction factor

ϕ_∞ Final creep coefficient of masonry

ϕ_{fl} Flexural strength reduction factor

σ_d Design compressive stress

SUMMARY

Masonry is one of the first building materials to be used structurally, these designs can be seen all throughout the world and across history. Every country has their own masonry code or guidelines that are followed for masonry construction. This paper will compare four masonry codes from four different countries to identify the similarities and differences in design aspects and analyze which code may have a stronger design approach than other codes. The codes being analyzed are the United States code TMS 402-16, European code (EC6) EN 1996:2005, the New Zealand code NZS 4230:2004 and the Indian code IS 1905:1987. Strength design and allowable stress design methodologies are both looked at in this paper for reinforced and unreinforced masonry. The code comparison shows many similarities but also many differences that can provide useful for future updating of these countries' masonry codes.

CHAPTER 1: USE OF MASONRY AS A STRUCTURAL MATERIAL

1.1 Introduction: The History of Masonry

The use of masonry as a structural material can be seen in designs as early as the 27^{th} century, with the creation of the pyramids in Egypt. For many centuries, masonry was the predominant material for buildings across much of the world. Early uses of masonry in structural designs can be seen in China, Rome, New Mexico, Istanbul, Italy and even in Chicago [1]. Many of the historic masonry structures are examples that show the beauty, strength, and versatility of this building material. Today still in Chicago, there is one of the tallest load-bearing brick buildings in the world today, Monadnock building. This building is 17 stories high, 60m tall.

Before steel and concrete came into the building industry, masonry was the material of the time producing strong and durable designs. Many of the masonry structures from early on are still in great shape even without the complex and rigorous design philosophies we have now. Masonry has many advantages [2]:

- Non-combustible- improves fire protection of the structure.
- High resistance against rotting, weather, and natural disasters (hurricanes and tornadoes)
- Overall aesthetic
- Durable- can carry large amount of compressive load.
- Long lifespans

Due to the rise of concrete and steel and masonry's structural limitations the structural use of masonry decreased. Masonry is still prevalent in structural and non-structural designs.

1.2 Research Objective

Standards and design specifications for the design of masonry structures began as early as 1910. Today, most countries have their own codes or guidelines as to how they go about designing masonry structures. With different countries having their own masonry codes this thesis will provide a comparison between codes from four different countries. The objective is to identify the similarities and differences within the different countries' codes and come to an overall conclusion about these codes surrounding the strength of designs and potential applications to other codes. The codes under comparison are the United States code TMS 402-16, European code (EC6) EN 1996:2005, the New Zealand code NZS 4230:2004 and the Indian code IS 1905:1987.

1.3 History of the Countries' Codes

The European code was not always around, the first code of practice that covered any aspect of masonry design was after 1948 [3]. The document was called CP 111 and was not originally written to include loadbearing concrete brickwork but did include unreinforced concrete walls. Years later the European Union wanted to create a document that unified aspects of many design materials so that there was one design code for the whole union instead of separate codes in each country. To create the Eurocode many aspects of design had to be left open for different methods based on national choice.

Similarly, the first masonry standard in New Zealand was introduced as NZS 95 Part X in 1948 [4]. The standard included stone, burned clay, concrete blocks and brick construction. The standard also included design requirements for reinforced and unreinforced masonry structures. Over the next 37 years, many revisions on format and technical content created the new document NZS 4230P in 1985. After another 5 years NZS 4230 was finally released in 1990. In

2004, the NZS removed the design of unreinforced masonry with the exception for low rise veneer structures.

The first draft for a single masonry design standard began in the United States in 1977 [2]. The Masonry Society developed the first standard for brick and block masonry which became chapter 24 of the 1985 Uniform Building Code (UBC). In 2009, the UBC then became the International Building Code. The American Concrete Institute- American Society of Civil Engineers published ACI-ASCE 530 in 1988 then was further revised to its current version of TMS 402. TMS 402 includes design for reinforced and unreinforced masonry structures.

Masonry construction is commonly used today in India however there is a lack of Indian standards for masonry construction. The code of practice seen in India was first published in 1961 and had only guidelines for unreinforced masonry construction [5]. Today that still holds true, reinforced masonry does not have a code of practice due to the quality of bricks available in the area are not suitable for the use in reinforced applications. Although reinforced masonry is not covered in IS 1905, India's design alternative for reinforced masonry is a confined masonry system but this system will not be included in this paper.

CHAPTER 2: DESIGN PHILOSOPHIES

Allowable stress and strength design are two main philosophies to analyze and design masonry members.

2.1 Allowable Stress Design

Allowable stress method designs masonry members to resist service loads based on the masonry material strength. The only masonry codes that use this design method are the TMS code for reinforced and unreinforced masonry and the IS code for unreinforced. Calculated design stresses are compared to maximum allowable stresses found in the masonry code. The applied stresses need to be less than or equal to the maximum allowable stresses specified by the code.

Some assumptions and design principles used in allowable stress design are [6]:

1) Plane masonry sections subjected to bending remain plane after bending.

2) Stress is linearly proportional to strain within allowable stress range.

3) Tensile stresses are to be resisted by reinforcement.

4) Any tensile strength of masonry is ignored.

2.2 Strength Design

Strength design also known as ultimate limit state, includes load factors that accounts for uncertainties and the probability of multiple loads acting simultaneously. Strength design includes strength reduction factors to account for uncertainties and the probability that multiple loads may be acting simultaneously.

Some assumptions and basic engineering mechanics used in strength design are [7][8][9]:

1) Plane masonry sections subjected to bending will remain plane after bending.

2) If reinforced, the distance from the neutral axis is proportional to the strain in the masonry and reinforcement. For unreinforced, the flexural stresses are assumed proportional to the strain. (strain relationships between reinforcement, grout, and masonry)

3) For reinforced and unreinforced the maximum masonry compressive stress is 0.80f'm (TMS code) or 0.85*f'm* (NZ code).

4) For reinforced masonry, the compressive stress block is rectangular and is uniformly distributed over an equivalent compression block having a depth of a=0.8c (TMS 402) or 0.85c (NZS 4230).

5) Maximum strain is 0.0025 for concrete masonry and 0.0035 for clay masonry. The New Zealand code specifies a maximum strain of 0.003 for unconfined masonry.

6) For reinforced masonry, compressive and tension stresses below the yield strength is taken as elastic modulus of the reinforcement multiplied by the strain in the steel. If

strains are greater than the yield strain, then stress in reinforcement is taken as yield strength (fy).

2.3 Load Combinations

Load combinations and factors are given in this section for both allowable stress design and strength design for all codes. This paper does not use load combinations to compare the strength of masonry members however the combinations and load factors are presented to give an idea how each code calculates loads per design.

Load combinations for allowable stress design do not amplify or increase the loads on the structure where in strength design the loads on the structure are increased for the design. Although the Indian Standard only designs based on allowable stress, the code allows for a one third increase in permissible stresses or a 25% decrease on loads [10]. This increase in permissible stresses or decrease in loads only applies to load combination equations 2-4, involving wind or earthquake forces.

Load combinations used for all designs including masonry design in the United States are found in the Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7-10 [11]. The load combinations used in the design with the Indian standard can be found in the code and commentary of IS 1905, these load combinations are also consistent with those given in other Bureau of Indian Standard codes [10]. Load combinations for allowable stress design can be found in Table 1.

Code	Load Combinations: Allowable Stress Design
TMS (ASCE 7-10) [11]	1. D 2. $D + L$ 3. $D + (L_r \text{ or } S \text{ or } R)$

	4. *$D + 0.75*L + 0.75*(L_r$ or S or R)* 5. *D + (0.6*W or 0.7*E)* 6. *$D + 0.75*L + 0.75*(0.6*W) + 0.75*(L_r$ or S or R)* 7. *D + 0.75*L + 0.75*(0.7*E) + 0.75*S* 8. *0.6*D + 0.6*W* 9. *0.6*D + 0.7*E*
IS (5.2.4) [10] IS (5.2.5) [10]	1. *DL + IL* 2. *DL + IL + (WL or EL)* 3. *DL + WL* 4. *0.9DL + EL* Modified load combinations with 25% reduction 1. *0.75[DL + IL + (WL or EL)]* 2. *0.75[DL + WL]* 3. *0.75[0.9DL + EL]*

Table 1: Load Combinations for Allowable Stress Design

Strength design load combinations for designing structures in the US can also be found in ASCE 7-10. These load combinations apply a load factor to the service loads so they can be safely compared to the members strength while maintaining the service loads to stay within the elastic range. Strength design load combinations for Eurocode are found in the Basis of Structural Design BS EN 1990 [12]. The equations shown for EC6 are fundamental combinations, EC6 also has combinations for accidental situations and seismic design, these equations are not shown in this paper but can be found in the BS EN 1990. The load combinations for the New Zealand standard can be found in AS/NZS 1170 [13]. The New Zealand load combinations incorporate short- and long-term load factors, combination factors and earthquake factors. These factors can be seen in a table within AS/NZS 1170. NZS also has load combinations for snow, liquid pressure, rainwater ponding, ground water and earth pressure but will not be presented in this paper but can be found in AS/NZS 1170 as well. Load combinations for strength design can be found in Table 2.

Code	Strength Design Load Combinations
TMS (ASCE 7-10)	1. *1.4*D* 2. *1.2*D + 1.6*L + 0.5*(L_r or S or R)* 3. *1.2*D + 1.6*(L_r or S or R) + (L or 0.5*W)* 4. *1.2*D + 1.0*W + L + 0.5*(L_r or S or R)* 5. *1.2*D + 1.0*E + L + 0.2*S* 6. *0.9*D + 1.0*W* 7. *0.9*D + 1.0*E*
EN (EN 1990 6.4.3.2) [12]	Persistent or transient design $E_d = E\{\gamma_{G,j}G_{k,j};\gamma_P P;\gamma_{Q,1}Q_{k,1};\gamma_{Q,i}\Psi_{0,i}Q_{k,i}\}$ $j \geq 1$; $i > 1$ Combinations in brackets can be express as; $\sum_{j\geq1}\gamma_{G,j}G_{k,j}\ "+"\ \gamma_P P\ "+"\ \gamma_{Q,1}Q_{k,1}\ "+"\ \sum_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i}$ Or STR for internal failure or excessive deformation of the structure or structural members $\sum_{j\geq1}\gamma_{G,j}G_{k,j}\ "+"\ \gamma_P P\ "+"\ \gamma_{Q,1}\Psi_{0,1}Q_{k,1}\ "+"\ \sum_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i}$ Or GEO for failure or excessive deformation of the ground $\sum_{j\geq1}\xi\gamma_{G,j}G_{k,j}\ "+"\ \gamma_P P\ "+"\ \gamma_{Q,1}Q_{k,1}\ "+"\ \sum_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i}$ "+" implies to be combined with Summation implies the combined effect of ξ is a reduction factor for unfavorable permanent actions G
NZ (NZS 1170 4.2.2) [13]	1. *1.35G* Permanent (dead) action only 2. *1.2G, 1.5Q* Permanent and imposed action 3. *1.2G, 1.5Ψ_lQ* Permanent and long-term imposed action 4. *1.2G, W_u, Ψ_cQ* Permanent, wind and imposed action 5. *0.9G, W_u* Permanent and wind action reversal 6. *G, E_u, Ψ_EQ* Permanent, earthquake and imposed action 7. *1.2G, S_u, Ψ_cQ* Permanent, specific action and imposed action

Table 2: Load Combinations for Strength Design

2.4 Strength Reduction Factors and Factors of Safety

Reduction factors and factors of safety are used to make sure a structure is stronger than it needs to be to allow for any variance in an emergency or any unexpected situation. Reduction factors for strength design can be seen in Table 3.

The EC6 code does not have strength reduction factors like the NZS or TMS codes, the EC6 code uses partial factors (γ_m) to obtain the resistance design value and this partial factor is based on the type of unit and mortar used in the design. These strength reduction factors, or partial factors are applied at the end to obtain the design capacity. The partial factors for EC6 are categorized in two levels, Category 1, and Category 2 and both categories have multiple classes within the categories. There are different partial factors for reinforced and unreinforced and for when the masonry is in a different loading state. Common partial factors for masonry design can be found in a table in EN 1996-1 section 2.4.3 [15]. EC6 does state a serviceability limit is to be analyzed with the value of the partial factor (γ_m) to be taken as 1.

Code	Reduction Factors
TMS (9.1) [14]	Reinforced flexure or axial Φ= 0.90 Unreinforced flexure or axial Φ= 0.60 Shear Φ= 0.80 Bearing Φ= 0.60
NZS (3.4.7) [8]	Flexure with or without axial tension/compression Φ= 0.85 Axial tension Φ= 0.85 Bearing Φ= 0.65 Shear Φ= 0.75 **if design moments, axial loads, or shear forces are derived by overstrength of adjacent sections Φ=1.0**

Table 3: Reductions factors for Strength Design

CHAPTER 3: MATERIALS

Masonry design just like any other structural design relies on the material properties to make the design satisfactory. Material properties for masonry vary depending on how they are produced and what type of components are available. Main materials used in masonry design are the masonry unit itself, grout, mortar, and reinforcement, if applicable. The masonry unit comes in a couple different materials and sizes. The most common material used for masonry units are clay and concrete. This paper will focus on masonry design with this material even though codes do have specifications to design with other material like stone, glass, and autoclaved aerated concrete.

3.1 Compressive Strength of Masonry

One main factor that is needed when designing masonry members is the compressive strength of the masonry unit. Each code has their own material strengths recorded from prism tests or other methods based off prism test data and compiled to determine the strength for that unit. All the codes provide general compressive strengths for both clay and concrete units and can be seen below in Table 4. These values given by the codes are based on many prism tests to determine common strengths of the materials.

Code	**Compressive Strength of Masonry**	
	Concrete	Clay
TMS (9.1.9.1) [14]	f'_m = 10.34 N/mm^2 - 27.58 N/mm^2 Units less than 102mm nominal height reduce the value by 85%.	f'_m = 6.90 N/mm^2 – 41.37 N/mm^2
EC6 (3.1.2) [16]	Class A f_b = 3.5 – 35 N/mm^2 Class B f_b = 2.8 - 7 N/mm^2	f_b = 5 – 69 N/mm^2
NZS (3.4) [8]	Type C Max f'_m= 4 MPa	*NZS 4230 does not include clay*

	Type B f'_m= 12-12.5 MPa Type A $f'_m \geq 12$ MPa Common 15 MPa	
IS (3.1) [17]	$f_b = 3 - 35$ N/mm^2	$f_b = 3.5\text{-}40$ N/mm^2

Table 4: Compressive Strengths of Masonry Units

Both the EC6 and NZS codes give an expression to calculate characteristic compressive strength of masonry. This value takes into consideration the strength of the unit and the mortar and determines the design compressive strength to be used in the design. The TMS and IS codes do not provide an expression for characteristic strength of masonry. The TMS code does have a table that provides values for the compressive strength of masonry based on the unit strength and type of mortar used which can be seen in table 1 and 2 for clay and concrete masonry respectively in TMS 602 specification section 1 [14]. The IS code provides a table that has values for compressive stresses based on the strength of the unit and type of mortar used; this table can be found in section 5 in IS 1905. The characteristic compressive strength of masonry for EC6 uses coefficients based on the type of mortar being used, the expression for characteristic compressive strength for general purpose mortar is [15]:

$$f_k = K f_b^{\alpha} f_m^{\beta} = K f_b^{0.7} f_m^{0.3} \text{ (EN 1996 -1-1 Eqn. 3.1)} \qquad (1)$$

Table 3.3 in EN 1996- 1-1 gives values for the constant *K*. The expression the NZS code gives to calculate compressive strength is based on prism test data and is defined as [8]:

$$f'_m = f_m - 1.65 x_m \text{ (NZS 4230 Eqn. B-2)} \qquad (2)$$

X_m in the above expression is the standard deviation of the strength of masonry and f_m is the mean compressive strength of masonry. Expressions for these values can be found in appendix B in NZS 4230. Like TMS and IS, the NZS code will state the compressive strength of masonry based on prism tests done on units that will be used or the compressive strength will be stated by the engineer. Prism tests will provide the most accurate value for compressive strength of masonry and all codes have a standard they follow to obtain that.

3.2 Variable Material Properties

Another factor that effects the design of masonry are the material properties of the unit itself. Masonry like any other structural material can be affected by climate and limitations of the material itself. Table 5 lists material properties for each specific code. The NZS 4230 code is the only code that does not consider clay, to design with clay using a New Zealand standard NZS 4210 needs to be used; this standard will not be looked at in this paper. Some material properties to consider in masonry design are thermal expansion, movement due to moisture, and creep. The IS code is the only code that does not mention a creep coefficient for masonry design.

Code	**Masonry Material Properties**	
	Clay	**Concrete**
TMS (4.2) [14]	Thermal expansion (10^{-6}/ºC): 7.2 Moisture expansion: k_e= 3 x 10^{-4} Creep: k_c= 0.1 x 10^{-4} per MPa of stress	Thermal (10^{-6}/ºC): 8.1 Shrinkage: k_m= 0.5s_l Creep: k_c= 0.36 x 10^{-4} per MPa of stress
EC6 (3.7.4) [15]	Thermal expansion (10^{-6}/ºC): α_t= 4 to 8	Thermal expansion (10^{-6}/ºC): 6 to 12 Moisture coefficient (mm/m): -1.0 to -0.2 Creep: ϕ_∞= 1.0 to 3.0

	Moisture coefficient (mm/m): -0.2 to +1.0 Final creep coefficient: ϕ_∞= 0.5 to1.5	
IS (IS 3414 4.2.1.1) [17]	Thermal expansion (10^{-6}/ºC): 5 to 7	Thermal expansion (10^{-6}/ºC): 10 to 14 Moisture coefficient: dense concrete 0.2 to 0.5 mm/m lightweight concrete 0.5 to 0.8 mm/m
NZS (3.5.2.6) (CA3.5.1) [19]	*Clay not covered in NZS 4230*	Thermal expansion (10^{-6}/ºC): 5– 12 Moisture expansion/contraction: Un-grouted- 0.4 mm/m Grouted- 0.7 mm/m Creep: 2.5

Table 5: Material coefficients

The modulus of elasticity is an important factor to calculate deformations associated with a material. All the codes except the NZS code give an expression for the modulus of elasticity. The modulus of elasticity is given as a constant in the NZS code, but the code does provide a recommended relationship for concrete and clay masonry if required in the handbook and is like the expression given in EC6 [19]. The expressions used for the elastic modulus per code for masonry can be found in Table 6. All the codes use the same elastic modulus value for the reinforcing steel; the modulus of elasticity for the reinforcing steel is taken to be 200,000 N/mm^2.

Code	Elastic Modulus
TMS (4.2.2)	E_m= 900 f'_m (concrete) E_m= 700 f'_m (clay)
EC6 (3.7.2)	E_m= $K_E f_k$ (clay and concrete) K_E = 1000
IS (3.3.2)	E_m= 550f_m (clay and concrete)
NZS (3.4.2)	E_m= 15000 N/mm^2 E_m= 1000f'_m (clay and concrete)

Table 6: Elastic Modulus for Masonry Per Code

3.3 Masonry Movement

To accommodate the expansion and contraction of the masonry, movement joints need to be designed to allow for a change in volume in the material or other reasons the masonry may move. If proper movement is not allowed in the design, it could lead to cracking in the masonry. The type, size and spacing of these movement joints are important to the overall functionality of the structure. Control or expansion joints are used to provide the necessary room so structural integrity of the structure will not be compromised. Spacing of control joints per code can be found in Table 7. The US also has criterion based on location, if located in a high seismic zone and a large amount of horizontal reinforcement is used the spacing of the control joints can be increased from the value given in Table 7 [21]. EC6 bases control joint spacing from unreinforced masonry designs, when used in reinforced masonry designs the maximum spacing of the joints can be increased and the code suggests getting guidance from the manufactures of the bed reinforcement to determine the spacing [22]. The IS code requires movement joints in masonry structures at a maximum spacing of 30-meter intervals [23].

Code	Movement Joints
TMS (TEK 10-1A) [21]	The distance between joints shall not exceed the lesser of length to height ratio 1.5:1 or 7.62 meters
EC6 (2.3.4.2) [22]	Clay masonry l_m= 12m Aggregate concrete masonry l_m= 6m
NZS (C3.5.2.6) [8]	Control joints: 6 to 8 m Thermal joints every 30 to 50m
IS (IS 3414 4.4) [23]	Joint spacing should not exceed 30m and not be less than 15mm in width.

Table 7: Space Requirements for Masonry Movement

CHAPTER 4: UNREINFORCED MASONRY

Prior to 1975, unreinforced masonry was one of the frequently used materials for structures. Today there are still many unreinforced masonry structures all around the world. Building codes for masonry construction became stricter in the beginning of 1973 and then after is when reinforced masonry started to be used more [24].

In this section, specifications regarding unreinforced masonry design are presented, compared, and discussed for the various codes. The section will be broken up into two sections, one for allowable stress design and the other for strength or ultimate limit state design. The analysis of masonry members will be done by looking at members subjected to axial compression, flexure, shear and then combined axial and flexure.

4.1 Allowable Stress Design for Masonry Design

As stated previously, the only two codes that use allowable stress design are TMS and IS. This section will compare the TMS and IS code for the various states of interest.

4.1.1 Axial Compression

Axial compression on a member is due to vertical loads mainly from dead and live loads. Walls and columns are typical masonry elements in which compressive forces will act upon. The slenderness of the masonry member plays a role in determining the allowable capacity for both the IS and TMS in the allowable stress design for unreinforced masonry. In the IS code, to determine the allowable stress, multiple factors are multiplied together to account for characteristics within the design. [18]. The equation given to calculate allowable stress for IS can be found below.

$$f_c = f_b * k_s * k_a * k_p \quad \text{(IS Eqn. 5.4)} \qquad (3)$$

Table 8 in IS 1905 provides values for basic compressive stresses (f_b) of masonry based on units with a height to width ratio being less than 0.75 and having a compressive strength within a range of 3.5-40 N/mm^2. The other three factors are stress reduction factor (k_s), area reduction factor (k_a), and shape modification factor (k_p). Table 9 in IS 1905 provides the stress reduction factor based on the slenderness ratio and the eccentricity of the loading divided by the thickness of the member. The area reduction factor is given as an expression and is applied only when the sectional area of an element is less than 0.2m^2. This area reduction factor is based on the concept of failure in a small section due to sub-par units compared to a larger area with adequate units [17]. This factor is appropriate to include when analyzing masonry structures in Indian due to the varying unit strength obtained during the manufacturing process of units in the country, whereas in North America or Europe the manufacturing process of units is more reliable and does not result in such a variation of unit strengths. The expression for k_a can be found below.

$$k_a = 0.7 + 1.5A \quad \text{(IS 5.4.1.2)} \qquad (4)$$

The last factor the IS considers is the shape modification factor (k_p). This factor accounts for the shape of the unit, it takes into consideration the height to width ratio of the units as they are laid and is only applicable for masonry up to 15 N/mm^2 in strength [18]. Values for the shape modification factor can be found in table 10 in IS 1905.

The TMS calculates the allowable stress (F_a) a little differently than the IS code. One aspect the TMS does that the IS code does not is consider a wide range of slenderness ratios. The TMS code provides two separate equations to allow for slenderness ratios less than or equal to 99 or ratios greater than 99. These equations for the TMS can be found below. The IS code does account for slenderness ratios but only considers a slenderness ratio less than or equal to 27 [18].

$$F_a = \frac{1}{4} f'_m * \left(1 - \left(\frac{h}{140r}\right)^2\right) \quad \text{For h/r} \leq 99 \text{ (TMS Eqn. 8-13)} \qquad (4)$$

$$F_a = \frac{1}{4} f'_m * \left(\frac{70r}{h}\right)^2 \quad \text{For h/r} > 99 \text{ (TMS Eqn. 8-14)} \qquad (5)$$

For a design to be satisfactory for both codes the allowable stress needs to be greater than the calculated stress. The calculated stress is obtained by taking the axial force divided by the net area of the section. The TMS equation does not directly consider or have factors for eccentricity like the IS, however the TMS code provides a buckling equation that is used to check against premature stability failure due to eccentric axial loading [14]. The IS code does not mention anything about additional stability or buckling checks. The buckling equation provided by the TMS is not to be solely used to check adequacy of a member under combined axial and flexure but along with to make sure buckling does not control. This check should be done otherwise if a member has a slenderness ratio 99 or greater the compressive load may exceed the maximum eccentricity of $0.1t$ and then the allowable compressive stress (TMS Eq. 8-19) will be overestimated, and the bucking equation may control the design. The buckling equation the TMS provides can be found below.

$$P \leq \frac{1}{4} P_e \quad \text{(TMS Eqn. 8-12)} \qquad (6)$$

$$P_e = \frac{\pi^2 E_m I_n}{h^2} * \left(\frac{1-0.577e}{r}\right)^3 \quad \text{(TMS Eqn. 8-16)} \qquad (7)$$

A comparison of calculated allowable stress values based on the TMS and IS codes can be found in Figure 1. The allowable stress values were calculated with varying strengths of masonry. A sample calculation of allowable stress for TMS and IS code can be found in Appendix A in this paper. The graph in Figure 1 shows the TMS calculates a larger allowable stress for all strengths of masonry compared to the IS code. The IS code calculates a smaller allowable stress because the IS code uses basic compressive stress values for varying strengths of masonry whereas the TMS code uses the strength of masonry and the slenderness ratio. Axial capacity for the IS code is limited by the basic compressive stresses given in the specification. For the IS code, the shape modification factor (k_p) and area reduction factor (k_a) equal 1.0, the stress reduction factor (k_s), based off zero eccentricity and the slenderness ratio from table 9 provided in IS 1905 equals 0.735 and from table 8 in IS 1905 the basic compressive stress (f_b) for a strength of masonry of 20 N/mm^2 equals 2.20 N/mm^2. Multiply all these factors together and the IS code calculates an allowable stress of 1.62 N/mm^2. The slenderness ratio for the TMS code is less than 99, equation 8-13 from the TMS code applies. Substituting in f'_m and the slenderness ratio in the TMS equation results in an allowable stress of 4.57 N/mm^2. Detailed calculations can be found in Appendix A at the end of this paper.

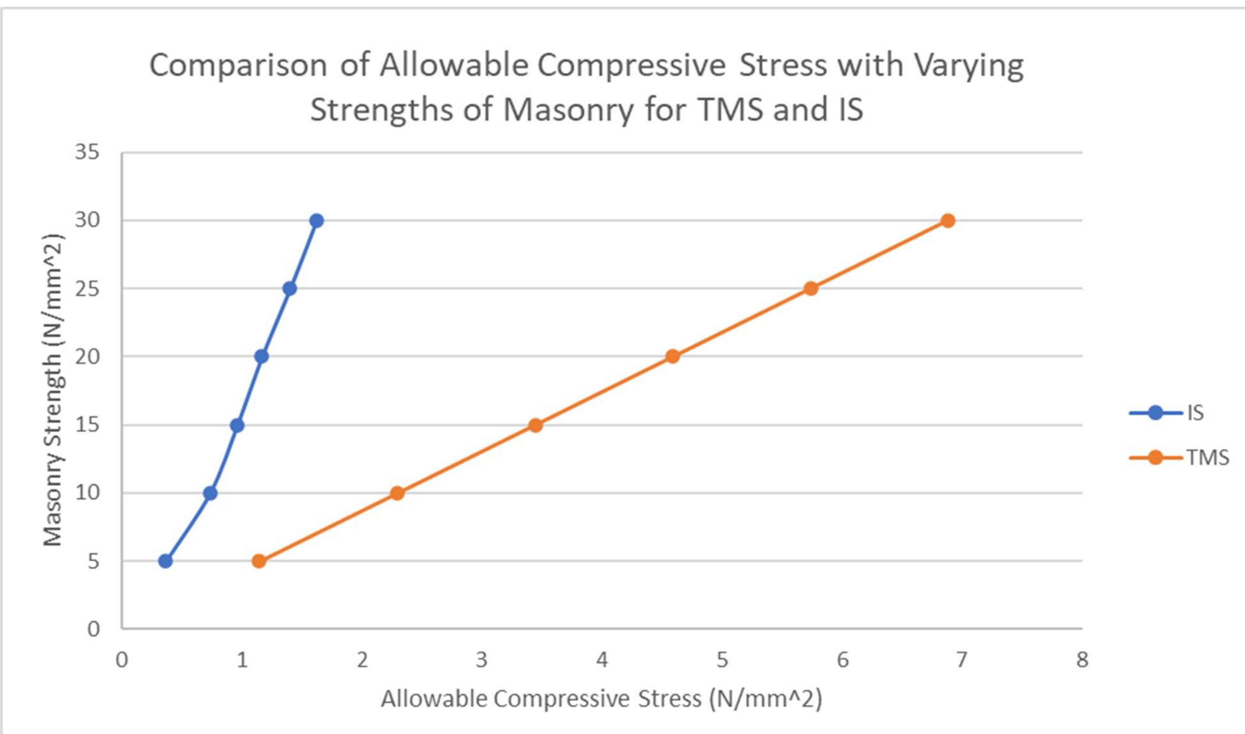

Figure 1: Comparison of Allowable Stress with Varying Strengths of Masonry for TMS and IS

4.1.2 Flexure

Masonry members subjected to only flexure have lateral loads applied to the members. Lateral loads are horizontal forces applied to the members by events like wind or earthquakes. Lateral loads when applied cause the masonry member to bend so the member needs to be able to withstand the applied lateral force. The TMS states that the allowable bending stress needs to be less than the applied bending stress [14].

$$F_b < f_b = \frac{M}{S_n} \qquad (8)$$

$$F_b = \frac{1}{3} f'_m \qquad \text{(TMS Eqn. 8-15) (IS 5.7)} \qquad (9)$$

For the IS code, bending is checked against its own permissible values. The Indian code states masonry elements subjected to lateral loads only shall be designed based on allowable tensile stress [10]. The Indian code gives values of allowable tensile stress based on the mortar used and can be found in section 5.4.2 of IS 1905. Both codes consider the tensile stresses when looking at flexural strength. Permissible values for bending according to the IS code can be obtained by increasing the basic compressive stress then reducing it to account for eccentric loading that is causing flexure. The IS code gives three different criteria to follow for different eccentricities [18].

1) $e < \frac{t}{24}$
2) $\frac{t}{24} < e < \frac{t}{6}$
3) $e > \frac{t}{6}$

For the first category, if the eccentricity is less than *t*/24 then an increase in permissible stresses is not allowed [10]. For the second category, if the eccentricity is greater than *t*/24 but less than *t*/6 the IS code allows for a 25% increase in permissible compressive stresses. Lastly, if the eccentricity is greater than t/6 then the code still allows for a 25% increase in permissible stresses but any area under tension is ignored when calculating capacity.

4.1.3 Tension

Primarily in masonry design it is assumed that the masonry member is not capable of taking any tension. Any tension in the section is ignored and not considered when calculating the overall strength of the member. If tension needs to be considered in the case of lateral loads normal to the plane wall which can cause flexural tensile stresses, then it is permitted to consider the tension in that specific case.

The TMS code checks tension based on the calculated axial compressive stresses plus the calculated flexural stresses needing to be less than or equal to the allowable flexural tensile stresses [14].

$$-f_a + f_b \leq F_t \text{ (TMS 8.2)} \qquad (10)$$

$$f_b = \frac{M}{S_n} \qquad (11)$$

$$f_a = \frac{P}{A} \qquad (12)$$

Table 8.2.4.2 in the TMS code provides allowable flexural tensile stresses for masonry (F_t). In the IS code, the allowable tensile stresses are based on the type of mortar, these permissible values can be found in Table 8 below. These are the same values that are used to check pure flexural for the IS code as well.

Mortar type	Tension Develops	Allowable Tensile Stresses
Grade M1 or better (5.4.2)	Normal to bed joints	0.07 N/mm^2 for bending in vertical direction
	Parallel to bed joints	0.14 N/mm^2 for bending in longitudinal direction
Grade M2 (5.4.2)	Normal to bed joints	0.05 N/mm^2 for bending in the vertical direction
	Parallel to bed joints	0.01 N/mm^2 for bending in the longitudinal direction

Table 8: Allowable Tensile Stresses for IS Code

Both these codes account for the direction the tension is developed to the bed joints. The IS code has limitations in terms of not allowing for the consideration of tensile stresses when the masonry members are used in water retaining or earth retaining structures [17]. The IS allows for an increase in allowable tensile stresses in bending in the vertical direction of 0.1 N/mm^2 for M1 or better mortar and 0.07 N/mm^2 for M2 mortar. This increase in allowable tensile stress can only be applied in the case of boundary walls and at the discretion of the engineer. The TMS code prior to 2011 allowed for a 1/3 increase in allowable tensile stresses when considering loading combinations involving wind and seismic, now based on reliability analysis the masonry Code Committee allows for a 4/3 increase in allowable tensile stresses [14].

4.1.4 Combined Axial and Flexure

Combined axial and flexural forces are mainly due to gravity and lateral loads. Both the TMS and the IS use an interaction or unity equation to design members subjected to these combined forces. The unity equation is used by both codes and is conservative [25].

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \text{ (TMS Eqn. 8-11)} \qquad (13)$$

The equations for the unity equation for the TMS and IS are:

$$F_a = \frac{1}{4} f'_m * \left(1 - \left(\frac{h}{140r}\right)^2\right) \quad \text{For h/r} \leq 99 \text{ (TMS Eqn. 8-13)} \qquad (14)$$

$$F_a = \frac{1}{4} f'_m * \left(\frac{70r}{h}\right)^2 \quad \text{For h/r} > 99 \text{ (TMS Eqn. 8-14)} \qquad (15)$$

$$F_b = \frac{1}{3} f'_m \quad \text{(TMS Eqn. 8-15)} \qquad (16)$$

$$F_a = \frac{1}{4} f'_m \quad \text{(IS 5.7)} \qquad (17)$$

$$F_b = 1.25F_a = 1.25 \times \frac{1}{4} f'm = 0.31 f'_m \text{ (IS 5.7)} \qquad (18)$$

The IS allows checking the stresses by the unity equation but also combined loading can be designed off the basis of separate calculations of the bending and axial stresses and then added together for the total stress [25]. The unity equation for IS code is not in the main part of the code but specifies it in the commentary section [17]. In Figure 2, a maximum wind load with varying applied vertical loads was calculated for both the TMS and IS code. The maximum wind load calculation is based off the vertical applied load and the allowable tensile stresses. For a detailed calculation on combined loading refer to sample calculations in Appendix A at the end of this paper. The TMS code calculates a higher maximum wind load compared to the IS code. The wind load is restricted by the tensile stress of the masonry which is why the IS code calculates a smaller load than the TMS. The IS code limits the tensile stress of the masonry to be 7 kN/m^2 whereas compared to the TMS code the tensile stress is 2280 kN/m^2. Having a much lower tensile stress value to begin with will result in a lower wind loading for the IS code.

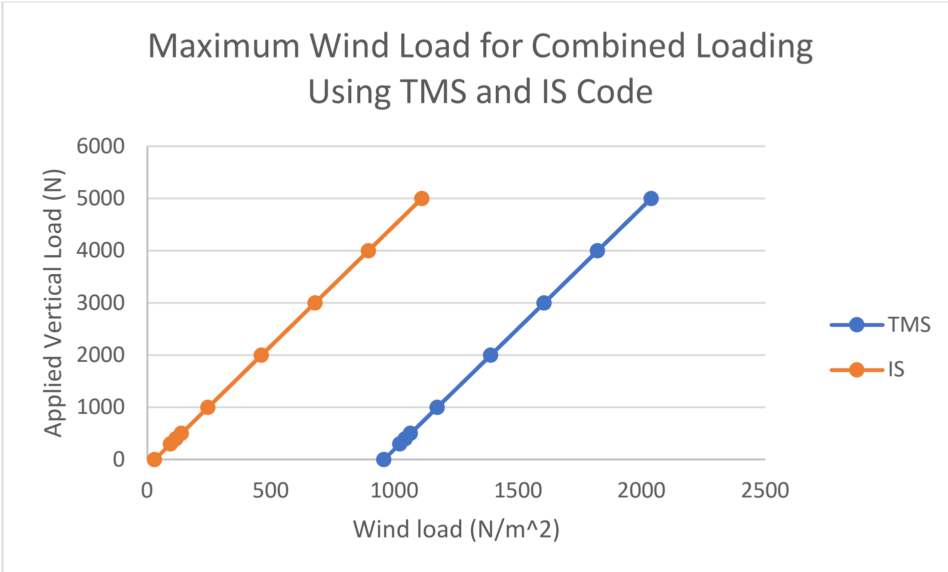

Figure 2: Finding Maximum Wind Load Based on Applied Vertical Load for TMS and IS Code

4.1.5 Shear

Masonry members are subjected to shear forces when lateral loads from wind, earthquakes or seismic forces are present. The masonry members that resist these shear forces are called shear walls. Shear walls are responsible to resist in-plane and out-of-plane loads. Generally, the capacity of the shear wall is based off the in-plane load on the members because the in-plane stiffness of the shear wall is greater than the out-of-plane stiffness. The typical shear failures seen in masonry are [26]:

1) Diagonal shear failure- stair step cracks in the mortar from head joint to bed joint: usually masonry is strong, but mortar is weak.
2) Flexural shear failure- tension cracks through the mortar and masonry units: usually masonry is weak, and mortar is strong.
3) Sliding shear failure- Sliding along horizontal crack in bed joints: usually when masonry has small vertical load.

The TMS and IS code account for two out of the three shear failure modes. The shear failure modes accounted for in both codes are diagonal and flexural shear failure. The TMS only considers sliding shear failure for autoclaved aerated concrete masonry. The TMS code calculates the applied shear stress based on the sectional properties of the masonry [6].

$$F_v = \frac{VQ}{Ib} \text{ (TMS Eqn. 8-17)} \qquad (19)$$

Since masonry tends to be rectangular shaped the applied shear equation can be reduced to:

$$F_v = \frac{3V}{2A} \text{ (TMS 8.2.6.1)} \qquad (20)$$

This equation will calculate the maximum shear stress of the masonry at mid-depth. This applied shear stress is then compared to the codes permissible shear stresses $F_v \leq f_v$.

For the TMS the permissible shear stress (f_v) values are [14]:

1) $0.125 f'_m{}^{1/2}$ MPa

2) 0.827 MPa

3) Running bond not fully grouted:

   $255 + 0.45 N_v/A_n$ kPa

4) Constructed of open-end units and fully grouted (masonry not laid in running bond):

   $414 + 0.45 N_v/A_n$ kPa

5) Constructed of other than open-end units and fully grouted (masonry not laid in running bond):

   103 kPa

Shear design for the IS code does not allow for tension stresses [18]. If designing for a certain load or checking the capacity, the permissible tension stress needs to be less than when in compression or equal to 0. If there is any tension in any part of the section, the IS code assumes that section is most likely cracked and cannot depend on resisting any shear. The basis of design for shear using the TMS does not neglect or assume that tension is zero. The permissible shear stress equation for IS 1905 can be found below. Where f_d is the axial compressive stress due to dead loads.

$$f_s = 0.1 + \frac{f_d}{6} \leq 0.5 \text{ N/mm}^2 \qquad \text{(IS Eqn. 5.4.3)} \qquad (21)$$

The IS code also states that in-plane shear shall not exceed any of the following [17]:

1) 0.5 MPa
2) $0.1 + 0.2f_d$
3) $0.125f_m^{1/2}$

4.2 Unreinforced Strength Design

The use of strength design in unreinforced masonry can be seen in EC6 and TMS. The design of unreinforced masonry assumes that the masonry members will behave elastically under design loads.

4.2.1 Axial Compression

In the TMS code the nominal axial strength is given by equation 9-11 and 9-12 and is based on the slenderness ratio of the masonry member [14].

$$P_n = 0.80\left\{0.80A_n f'_m\left[1-\left(\frac{h}{140r}\right)^2\right]\right\} \quad \text{For h/r} \leq 99 \text{ (TMS Eqn. 9-11)} \qquad (22)$$

$$P_n = 0.80[0.80A_n f'_m\left(\frac{70}{h}\right)^2 \quad \text{For h/r} > 99 \text{ (TMS 9-12)} \qquad (23)$$

A reduction factor (ϕ) accounts for slenderness and eccentricity and is based on a rectangular stress block in Eurocode's design of axial compression. In the TMS code, strength design can easily be done with interaction diagrams. The design of an unreinforced masonry structure using the TMS code is limited to the axial strength calculated by equations 9-11 or 9-12, compression controlled $0.80f'_m$ and by tension controlled not exceeding the modulus of rupture f_r.

In EC6 the design capacity of axial compressive strength is given by N_{Rd} and the applied axial load is given as N_{Ed}. For the design to be satisfactory under axial loads $N_{Ed} \leq N_{Rd}$ [15]. The axial capacity for a single width wall under axial loading only is:

$$N_{R_d} = \phi t f_k \quad \text{(EC6 1-1 Eqn. 6.2)} \qquad (24)$$

$$\phi = 1 - 2\, e_i/t \quad \text{(EC6 1-1 Eqn. 6.4)} \qquad (25)$$

The reduction factor (ɸ) has two conditions one for taking the moment at the top or bottom of the wall and another for the middle of the wall. Both conditions take account for initial eccentricity and eccentricity due to lateral loads. When considering in the middle of the wall an additional eccentricity is used that accounts for creep. If the slenderness ratio is 15 or less, then the eccentricity due to creep can be taken as zero. The expressions to calculate eccentricity for all these conditions can be found in EC6 in section 6.1.2.2 [15]. The TMS again does not have a factor that directly includes eccentricity in the axial compression calculation. Buckling is checked in a separate equation.

To compare the axial capacity between EC6 and TMS, the strength of masonry was varied, then the axial capacity was calculated and plotted on a graph which can be seen in Figure 3. Looking at the axial comparison graph EC6 calculates a larger axial capacity up to a masonry strength of 40 N/mm^2 after this point the TMS code calculates the larger capacity. There are several factors in which effect the axial capacity from both codes. The EC6 code uses the slenderness ratio to calculate the eccentricity due to creep and for this problem the slenderness ratio is small enough (less than 15) that eccentricity due to creep is ignored. The TMS uses the slenderness ratio to determine the axial equation to apply but also plays into calculating the capacity even if the slenderness ratio is small. To simplify the design and make the capacity more conservative the slenderness ratio can be ignored in the TMS. Calculating the axial capacity while ignoring the slenderness ratio can be seen in Figure 4. When designing with EC6 the strength of the masonry is determined by a characteristic compressive strength, taking into separate account of the unit strength and the mortar strength whereas the TMS considers only the unit strength. The percent difference at a strength of masonry of 40 N/mm^2 is only about 1.23% difference between the axial capacities. It can also be seen in the final axial calculation that the EC6 not only uses a reduction factor but also considers the thickness of the masonry and then designs compressive strength.

An experimental study done in 2017 by Kuddus and Fabregat analyzed the buckling behavior of masonry walls with EC6 and the US code ACI-530, which is now TMS 402. This study investigates load bearing masonry walls under vertical loads with and without eccentricities and with varying slenderness ratios. The overall study suggests that both codes underestimate the strength of the walls with the US code having a higher error percentage than EC6. In this study the US code substantially underestimates the bearing capacity of masonry walls. The TMS code analyzes buckling failure and cross section material failure separately where as EC6 considers both of those failure modes in one equation [27]. EC6 provides more accurate collapse loads for lower eccentricities and slenderness ratios. The US code gives the most conservative results in all cases with an error percentage of about 88% compared to EC6 which has about a 31% error percentage in determining the bearing capacity of masonry walls.

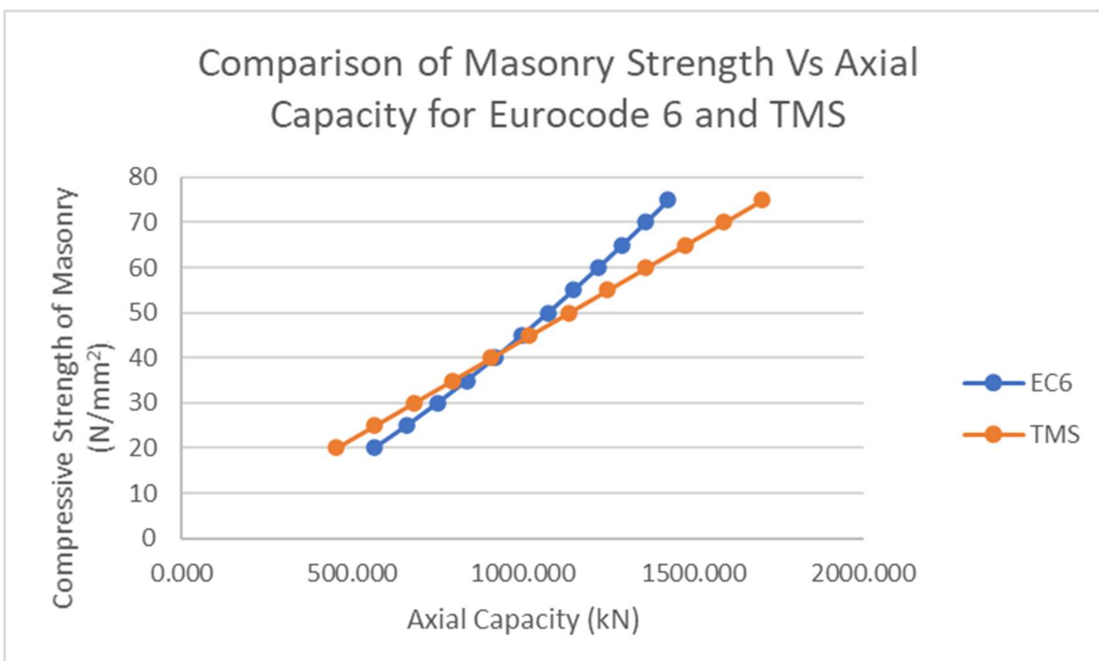

Figure 3: Axial Strength Comparison for TMS and EC6 Codes

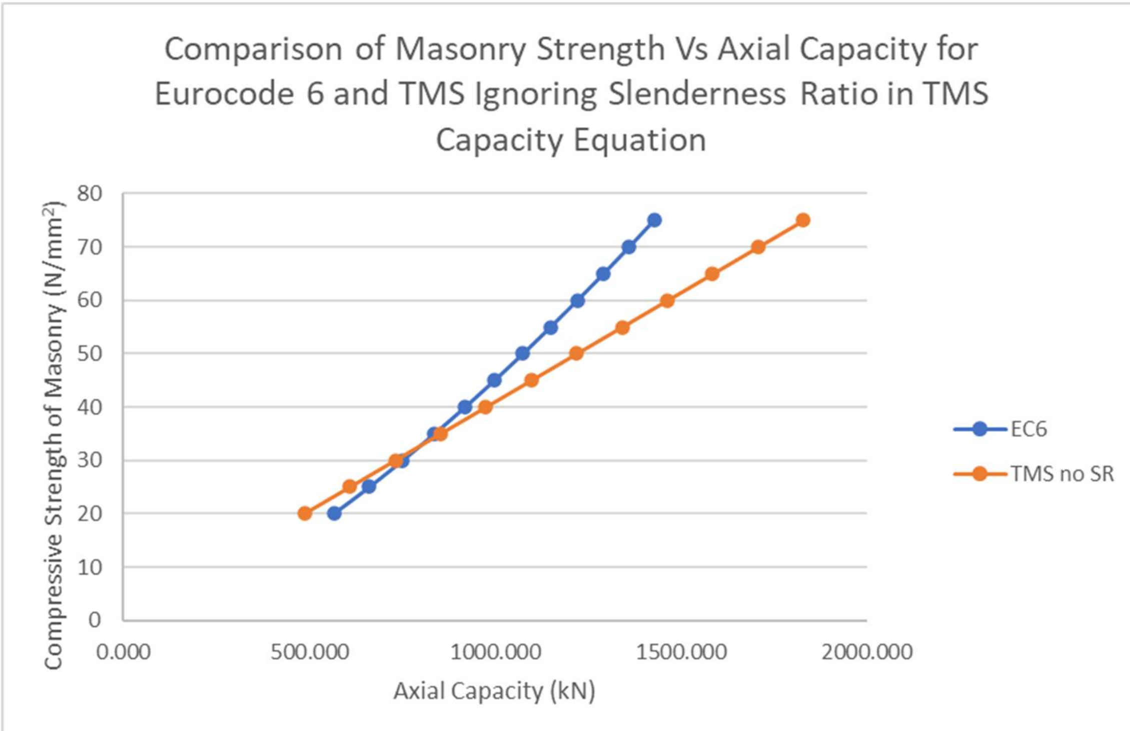

Figure 4: Axial Strength Ignoring Slenderness Ratio for TMS and EC6 Codes

4.2.2 Flexure

There is no explicit equation given in the TMS to calculate the nominal flexural strength of masonry however the usual assumption is the masonry is limited by tensile and compressive stresses and are assumed proportional to the strain [7]. With those assumptions and relationships, the nominal flexural strength can be calculated as:

$$F_u = \frac{M_u t}{2I_n} - \frac{P_u}{A_n} \text{ (TEK 14-4B Eqn. 4)} \qquad (26)$$

A positive value of F_u means the masonry is controlled by tension and the modulus of rupture (f_r) is reduced by ϕ=0.60 [7]. However, if F_u is calculated as a negative then compression is controlling and the design compressive stress of $0.80f'_m$ needs to be applied. Only dead loads or permanent loads should be used in determining P_u if trying to offset flexural bending.

The Eurocode offers two methods to design members for flexure. The first method uses the flexural strength of masonry calculated in section 3.6.3 in EC6 and bending moment coefficients that are based on the edge conditions [28]. The bending moment coefficients are based on a yield line analysis [28].

The other method is based on a three pinned arch forming in the wall [28]. Out of the two methods the first one is used most often simply because it does not depend on rigid supports having to resist an arch thrust. The code does specify and state the flexural strength of masonry should not be used when a permeant lateral load is present (ie retaining walls). The design moment equation for EC6 is given as:

$$M_{Rd} = f_{xd}Z \quad \text{(EC6 1-1 Eqn. 6.15)} \qquad (27)$$

Where f_{xd} is the design flexural strength according to the plane of bending and is calculated using section 3.6.3 in EC6 part 1 and Z is the elastic section modulus of the unit height or length of the wall. Unreinforced masonry walls subjected to lateral loading according to the Eurocode have an orthogonal strength ratio factor and should be considered [15]. This ratio is given as μ and the necessary values, f_{xk1} and f_{xk2}, needed for this ratio is found in EC6 in section 3.6.3 in two different tables. To calculate the applied moment due to a lateral load using EC6 bending moment coefficients are used. For a plane of failure that is parallel to the bed joints the moment is calculated as [15]:

$$M_{Ed_1} = \alpha_1 W_{Ed} l^2 \quad \text{(EC6 1-1 Eqn. 5.17)} \qquad (28)$$

For a plane of failure perpendicular to the bed joints the moment is calculated as [15]:

$$M_{Ed_2} = \alpha_2 W_{Ed} l^2 \quad \text{(EC6 1-1 Eqn. 5.18)} \qquad (29)$$

The alpha coefficients account for the edge conditions of the wall. The edge conditions of the wall can be free, fixed, or simply supported. W_{Ed} is the applied lateral load on the structure and l is the length of the wall. For a design that is satisfactory in flexure [15]:

$$M_{Ed} \leq M_{Rd} \quad \text{(EC6 1-1 Eqn. 6.14)} \qquad (30)$$

When comparing the flexural designs per TMS and EC6, both codes check for failure parallel and perpendicular to the bed joints. Since the TMS does not explicitly give an equation to calculate flexural strength the bending stress can be compared to the modulus of rupture as a design check. The flexural strength using EC6 may be less conservative than TMS due to EC6 only basing the required moment on the axis of bending and the section modulus whereas the TMS compares the flexural capacity to the modulus of rupture allowing for a higher design moment.

The flexural strength of masonry for both EC6 and TMS rely on the strength and type of mortar used and the direction of failure to the bed joints. For the EC6, a characteristic flexural strength value is given based on the strength of masonry and the material used for the masonry unit [29]. The TMS code gives a modulus of rupture, based on the type of mortar used as well but instead of considering the material used the TMS considers the type of unit (i.e., solid, un-grouted, and fully grouted) [14]. To determine pure bending using EC6, the characteristic flexural strength, section modulus and material property are used. If general purpose mortar is used, then there are only two possible values for the characteristic flexural strength per plane of failure [29]. This limits the design moment to two possible required moments for one unit size.

The TMS is similar but instead of two possible required moments there is only one per plane of failure given only one type of block is being considered. However, looking at the results from the flexural strength example problem, the TMS calculates a much higher required moment than EC6. This is due to the modulus of rupture that is used. There is not a provided equation in the TMS code to calculate flexural strength alone, however by using the relationship that the required moment must be less than the modulus of rupture times the section modulus can be used to check bending only. This calculation for pure bending is conservative.

4.2.3 Tension

Checking tension based on the TMS code for strength design is the same approach when designing with allowable stress design where the compressive and flexural stresses are added together and is considered tension controlled if the sum is greater than the modulus of rupture (f_r) [7].

$$\frac{M_u t}{2I_n} - \frac{P_u}{A_n} \leq f_r \quad \text{(TEK 14-04B Eqn. 4)} \qquad (31)$$

Flexural tension is checked using EC6 by having a separate partial factor γ_m in the design that accounts for it. These partial factors can be found in the National Annex NA.1 in table 2.1 [9]. In the commentary in Eurocode's design guide, states no significant increase will be seen in the design values when flexural tension is considered because the strength of the unit is more [9].

4.2.4 Combined Axial and Flexure

Eurocode and TMS consider two failure modes for combined forces which are parallel and perpendicular to the bed joints.

The Eurocode has three different methods that can be used when walls are subjected to combined forces [15]:

1) ϕ Factor method- a stability check with an additional eccentricity at mid-height, to allow for the wind moment. Or by using the slenderness reduction factor Φ, would be replaced by ϕ_{fl}, which considers the flexural strength (EC6 part 1-1 6.4.2).

2) Apparent Flexural Strength- allows the design flexural strength to be increased in the weak direction to account for the dead load stress (EC6 part 1-1 6.4.3).

$$F_{xd1,app} = f_{xd} + \sigma_d \qquad (32)$$

$$\sigma_d < 0.2 f_d \qquad (33)$$

3) Equivalent bending moment- allows for a combined loading moment to be calculated (EC6 part 1-1 6.4.4).

The additional eccentricity factor in the ϕ factor method is similar when analyzing axial compression capacity. For the apparent flexural strength method, the design flexural strength can be increased to the apparent flexural strength due to the permanent vertical load on the member. The last method for EC6 is the equivalent bending moment method and this method uses a combination of the ϕ factor method and apparent flexural strength method to account for combined loading. Annex 1 in EC6 gives an adjustment to modify the bending coefficients in section 5.5.5 to account for horizontal and vertical loading.

The TMS uses an interaction diagram to see the effects of combined loading. The diagram takes the axial strength limit, compression controlled and tension-controlled values of the masonry. The axial strength limit for the TMS uses equations 9-11 or 9-12. The compressive stress is limited to $0.8f'_m$ and the tensile stress does not exceed the modulus or rupture.

Connecting these three points on the graph will create a boundary to see what the masonry structure can handle. Using an interaction diagram is a quick, easy, and conservative way to see the capacity of masonry. EC6 does not mention anything about interaction diagrams for masonry structures. Reinforced masonry design in EC6 is largely based off BS 5628 and in part 2 of this code interaction diagrams are given and can be used if necessary.

4.2.5 Shear

Both the TMS and EC6 consider two modes of failure for shear, which are flexural and diagonal shear. TMS section 9.2.6 [14] states that the shear stress follows a parabolic distribution where EC6 Part 1-1 6.2 [15] states that the calculated design shear is based on assuming a linear stress distribution of the compression stresses on the length of the wall. The design value of shear resistance for EC6 is given by [15]:

$$V_{Rd} = f_{vd} t l_c \qquad \text{(EC6 1- Eqn. 6.13)} \qquad (34)$$

Where l_c is the length of the compressed section of the wall, t is the thickness and f_{vd} is the design value of shear strength. EC6 does not give limiting values or conditions for shear design like the TMS code does. For the TMS the shear strength shall be the lesser of these conditions listed in section 9.2.6.1 [14]:

1) $3.8A_{nv}f'_m$

2) $300A_{nv}$

3) Running bond masonry not fully grouted:

$$56A_{nv} + 0.45N_u.$$

4) Constructed of open-end units and fully grouted (masonry not laid in running bond):

$$56A_{nv} + 0.45A_{nv}$$

5) Fully grouted masonry laid in running bond.

$$90A_{nv} + 0.45A_{nv}$$

6) Constructed of other than open-end units and fully grouted (masonry not laid in running bond):

$$23A_{nv}$$

CHAPTER 5: REINFORCED MASONRY

Plain masonry is strong in compression and weak in tension, so the use of reinforcement is commonly used in masonry design to carry the tensile stresses. When determining the strength of reinforced masonry any tensile resistance from the masonry is neglected and the section of masonry subjected to any tensile stresses is assumed to be cracked which in turns transfers all tensile forces to the reinforcement.

In 1813, a man named Marc Brunel, a chief engineer for New York City, proposed the idea to use reinforcement in a masonry chimney that was under construction [30]. This was the first time that reinforced masonry was used but it was not until 1825 that the first major application of reinforced masonry was seen in the construction of the Thames Tunnel in London. The use of reinforced masonry started to spread throughout the world, especially in areas that were familiar with the potential damage from earthquakes, even though limited research and tests were available at this time. Brunel was credited for first proposing the idea of reinforced masonry while another man named A. Brebner, who was an Under Secretary in the public Works Department in India, is credited for the modern development of reinforced masonry due to his extensive tests and research on reinforced brick masonry in 1923. Following his report, the use of reinforced masonry increased in areas of high seismic zones.

Between 1880 and 1920, there is little record of major use of reinforced masonry except for the construction of The Palace Hotel in San Francisco, California in 1875. It was not until after the 1906 San Francisco earthquake that the United States really took an interest in reinforced masonry research in the late 1920's and early 30's, however unreinforced masonry design was still being used. After the 1933 Long Beach earthquake in California, it was realized that unreinforced structures were susceptible to major damage in seismic areas and significant changes were made. The first codes for reinforced masonry can be seen in the United States in the early 1950's and in Europe in the 1960's and 70's. Still over the year's earthquakes were destroying unreinforced, reinforced, and retrofitted unreinforced masonry buildings in seismic areas and it was not until the 90's and 2000's that a better reinforced masonry design, proper detailing, and quality control was implemented [2].

5.1. Allowable Stress Design for Reinforced Masonry

The only code that utilizes allowable stress design for reinforced masonry design is the United States TMS code. This section will discuss the limit states associated with this code.

5.1.1 Axial Compression

To calculate axial compression using the TMS code uses the following equations are used [14]:

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)[1 - (\frac{h}{140r})^2] \quad \text{for } h/r \leq 99 \text{ (TMS Eqn. 8-18)} \qquad (35)$$

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)(\frac{70r}{h})^2] \quad \text{for } h/r > 99 \text{ (TMS Eqn. 8-19)} \qquad (36)$$

The first part of the equation deals with the capacity of the masonry and provides a factor of safety against the crushing of masonry. The second part deals with the steel reinforcement used in the design and the last part deals with the slenderness ratio of the design. Reinforcement in axial compression requires ties or stirrups to confine the reinforcement.

5.1.2 Flexure

The TMS flexural requirements for reinforced masonry are very similar to the TMS unreinforced masonry requirements.

$$f_b \leq F_b = 0.45f'_m \text{ (TMS 8.3.4.2.2)} \tag{37}$$

The difference lies in how the masonry units are grouted. If the units are fully grouted, then the section is analyzed as a cracked section and the reinforcement is transformed into an equivalent masonry area [7]. If the units are partially grouted there are two types of behaviors

1) If the neutral axis lies within the compression zone, then the unit is analyzed using the method for fully grouted units.
2) If the neutral axis does not lie within the compression zone, then the portion of un-grouted units must be subtracted from the total masonry area that is carrying compression stresses.

The location of the neutral axis depends on several factors. The first factor is the modular ratio ($n=E_s/E_m$), this is a ratio of the elastic moduli of masonry and steel. The other factors are the spacing of the reinforcement, the reinforcement ratio (ρ), and the distance between the reinforcement and the extreme compression face (d) [7]. To make design a little easier though it is usually assumed that the neutral axis is within the compression zone and the effective width of the compression zone is taken as the smaller of [14]:

1) Six times the wall thickness
2) Center-to-center spacing of the reinforcement.
3) 1,829 mm

This requirement is only applied when masonry is laid in running bond and stacked bond and has beams spaced at or less than 1,219 mm on center.

5.1.3 Combined Axial and Flexure

To check combined loading the TMS provides an equation that can be used for standalone flexure or combined flexure and axial [31].

$$f_a + f_b < 0.45f'_m \quad \text{(TMS 8.3.4.2.2)} \qquad (38)$$

Adding the stresses from both axial and flexural conditions will give a resultant and then the masonry members are designed so the maximum combined stress does not exceed the allowable limit of [6]:

- $f_b \leq F_b$
- and $P \leq 0.25P_e$
- or the unity equation.

If the combined stresses are relatively large, then the compressive strength of masonry f'_m can be increased which will result in smaller cross sections for the walls, reduction in material, and an increase in construction productivity [9].

5.1.4 Shear

Any shear acting on the masonry structure will be resisted by the masonry or the shear reinforcement. The applied shear stress is calculated by [14]:

$$f_v = \frac{V}{A_{nv}} = \frac{V}{bd} \quad \text{(TMS Eqn. 8-21)} \qquad (39)$$

$$f_v \leq F_v$$

The allowable shear stress can be calculated by taking into consideration the shear resistance provided by the masonry and the steel reinforcement. The allowable shear stress in the TMS is broken up into two separate categories that are based on a ratio involving the moment divided by shear force times depth of the member in the direction of shear being considered. These categories the TMS uses can be seen below in equation 8-23 and 8-24.

$$F_v = (F_{vm} + F_{vs})\gamma_g \quad \text{(TMS Eqn. 8-22)} \qquad (40)$$

1) $\frac{M}{Vd_v} \leq 0.25$

a. $F_v \leq (3f'_m{}^{\frac{1}{2}})\gamma_g$ (TMS Eqn. 8-23) (41)

2) $\frac{M}{Vd_v} \geq 1.0$

a. $F_v \leq (2f'_m{}^{\frac{1}{2}})\gamma_g$ (TMS Eqn. 8-24) (42)

If the value of $M/(Vd_v)$ happens to be in between these two conditions linear interpolation is permitted. $M/(Vd_v)$ shall be taken as a positive number but shall not be greater than 1. To simplify the design, M/Vd_v can be assumed as 1, assuming M/Vd_v as 1 makes the design conservative.

The allowable shear resisted by masonry for special reinforced and all other shear walls can be calculated from the equations below respectively [14]:

$$F_{vm} = 0.25[(4.0 - 1.75(\frac{M}{Vd_v})f'_m{}^{\frac{1}{2}}] + 0.25\frac{P}{A_n} \quad \text{(TMS Eqn. 8-25)} \qquad (43)$$

$$F_{vm} = 0.5[(4.0 - 1.75(\frac{M}{Vd_v})f'_m{}^{\frac{1}{2}}] + 0.25\frac{P}{A_n} \qquad \text{(TMS Eqn. 8-26)} \qquad (44)$$

The allowable shear resisted by the steel reinforcement is calculated with equation 8-27 in the TMS code and is shown below [14]:

$$F_{vs} = 0.5(\frac{A_v F_s d_v}{A_{nv} s}) \qquad \text{(TMS Eqn. 8-27)} \qquad (45)$$

$$A_v = \frac{V_s}{F_s d} \quad \text{(TMS 8.3.5.1.4)} \qquad (46)$$

Shear reinforcements are provided when f_v exceeds F_v.

5.1.5 Allowable Stress Design of Lintels

Lintels in masonry design are masonry beams. A lintel is a horizontal structural member that is subjected to lateral, axial and shear loads while spanning between support points. The function of the lintel is to transfer the loads to the adjacent masonry [33]. Axial loads in lintels are generally ignored. Advantages of a concrete masonry lintel are easily blending into the surrounding masonry and is easily constructed without special equipment [34]. Lintels also can be designed as a continuous bond beam. This design has advantages that include better performances in high seismic or high wind areas, control of wall movement due to temperature differentials, and lintel deflection.

Allowable stress design for a reinforced lintel using the TMS code requires the depth of the neutral axis (j) and stresses from the masonry (f_b) and steel (f_s) to be determined and compared to the allowable stresses [42]. The TMS neglects all tensile strength from the masonry, mortar and grout and assumes all tension is being carried by the reinforcing steel. Members are sized so that tensile and compressive stresses stay within the allowable limits. The equation the

TMS uses to calculate the distance to the neutral axis can be found below. This equation uses the modular ratio (n) and the reinforcement ratio (ρ).

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho \text{ (Masonry Structural Design 8.2)} \tag{47}$$

$$\rho = \frac{A_s}{bd} \text{ (Masonry Structural Design 8.2)} \tag{48}$$

The distance to the neutral axis (k) is then used to find the internal moment arm (j) and is calculated with the following equation:

$$j = 1 - \frac{k}{3} \text{ (Masonry Structural Design 8.2)} \tag{49}$$

One common practice is assuming 0.9 for the depth of the neutral axis (j) and then determining a trial area of steel reinforcement and checking the design. The TMS calculates the maximum compressive stress in the masonry as [42]:

$$f_b = \frac{2M}{bjkd^2} \leq F_b \text{ (Structural Engineering Handbook 13.4)} \tag{50}$$

The maximum allowable tensile stress in the reinforcement is given as [42]:

$$f_s = \frac{M}{A_s jd} \leq F_s = 0.45f'_m \text{ (Structural Engineering Handbook 13.4)} \tag{50}$$

The calculated stresses need to be less than or equal to the allowable for the lintel design to be satisfactory. When the allowable masonry stress controls the design the lintel design becomes conservative. The TMS does allow the use of design tables given in the technical note, TEK 17, on allowable stress design of concrete masonry lintels [34]. These tables provide the allowable shear and moment capacities for various lintel sizes, bottom covers, and type of reinforcement.

The maximum compressive stresses in the steel and masonry can be rewritten to obtain the moment capacity for the steel and masonry as well. To find the maximum moment capacity the equations below can be used [45].

$$M_s = A_s f_s jd \qquad (51)$$

$$M_m = \frac{1}{2} bjkd^2 F_b \qquad (52)$$

5.2 Strength Design for Reinforced Masonry

The codes to be discussed in this section will include TMS, EC6 and NZS.

5.2.1 Axial Compression

EC6 does not have an axial only equation for reinforced masonry design. Axial compression can be checked using the method for unreinforced masonry in EC6 however that does not include the reinforcement in the calculation so it would not provide an accurate capacity. The New Zealand code calculates the nominal axial strength of a wall as [8]:

$$N_{nw} = 0.5 f'_m A_g [1 - (\frac{L_n}{40b})^2] \qquad \text{(NZS 4230 Eqn. 7-1)} \qquad (53)$$

To use axial strength equation given in the NZS code the wall should not be required to act in any type of ductile manner (i.e., no seismic loads). The NZS code does state that if a wall is less than 790mm long it is then considered a short wall and should be designed as a column. To design short walls using NZS 4230, section 7.3.1.5 should be used. The axial capacity provided by the NZS equation does not consider the contribution of the reinforcement either. The TMS code is the only code that breaks up the masonry contribution and reinforcement contribution separately for axial capacity. Like unreinforced design using TMS, the axial capacity for reinforced masonry design also has separate equations that account for a variety of slenderness ratios and the axial stress is limited to $0.20f'_m$. The equations the TMS code provides for reinforced axial design are given as [14]:

$$P_n = 0.80[0.80f'_m(A_n - A_{st}) + f_y A_{st}][1 - (\frac{h}{140r})^2] \quad \text{h/r} \leq 99 \text{ (TMS Eqn. 9-15)} \qquad (54)$$

$$P_n = 0.80[0.80f'_m(A_n - A_{st}) + f_y A_{st}](\frac{70r}{h})^2 \qquad \text{h/r} > 99 \text{ (TMS Eqn. 9-16)} \qquad (55)$$

5.2.2 Flexure

The TMS, EC6 and NZS code all use an equivalent rectangular stress block to calculate the nominal flexural strength. If the units are fully grouted, then to compute the capacity the internal moment arm between the compressive and tensile forces is used. Partially grouted units are analyzed the same way but with additional circumstances due to parts of the units not being solid influencing the overall strength. The nominal flexural capacity is based on the position of the neutral axis and the neutral axis depends on the reinforcement spacing as well. The flexural capacity for the TMS, NZS and EC6 code are given as follows [44][19][15]:

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) \quad \text{(TEK 17 Eqn. 12)} \qquad (56)$$

$$a = \frac{A_s f_y}{0.80 f'_m b} \quad \text{(TEK 17 Eqn. 13)} \qquad (57)$$

$$M_n = C_m(c - \frac{a}{2}) + T(d - c) + N_n(\frac{L_w}{2} - c) \text{ (NZS Masonry Manual 4.1)} \qquad (58)$$

$$M_{Rd} = A_s f_y d Z \qquad \text{(EC6 1-1 Eqn. 6.22)} \qquad (59)$$

$$Z = d(1 - 0.5\frac{A_s f_{yd}}{bdf_d}) \leq 0.95d \quad \text{(EC6 1-1 Eqn. 6.23)} \qquad (60)$$

For EC6 the design value of the moment of resistance, M_{Rd}, shall not be greater than [9]:

- $M_{Rd} \leq 0.4 f_d b d^2$ for Group 1 units does not include lightweight units
- $M_{Rd} \leq 0.3 f_d b d^2$ for Group 2, 3 and 4 and Group 1 lightweight unit

Alternatively, the NZS code also has a simplified flexural capacity design that can be used for a structure that does not exceed three or four stories and that also has limited ductility [8]. This method is not used as much as the equivalent stress block. To calculate the flexural strength using this approach from NZS the following equation is used:

$$\phi M_n \geq M_G^* + M_{Qu}^* + 1.5 M_E^* \text{ (NZS Eqn. 3-3)} \qquad (61)$$

5.2.3 Combined Axial and Flexure

In EC6 when designing masonry members for flexure and axial that have a slenderness ratio less than 12, the members can be designed using the method for unreinforced members. However, if the masonry members have a slenderness ratio greater than 12, two additional factors need to be considered [15].

1) Extra moment will account for second order effects.

   a. $M_{ad} = \frac{N_{Ed} {h_{ef}}^2}{2000t}$ (EC6 1-1 Eqn. 6.25) (62)

2) Additional eccentricity

   a. $e = \frac{{h_{ef}}^2}{2000t}$ (EC6 1-1 6.26) (63)

If the axial load is relatively small ($\sigma_d \leq 0.3 f_d$) EC6 states the axial force can be ignored, and the masonry member can be designed just for bending [15].

Both TMS and NZS codes check for combined loading using the equivalent rectangular stress block [7][19]. This stress block gives an estimation of magnitude and location of the resultant compressive force within the masonry.

$$M_n = T(d - \frac{h}{2}) + C(\frac{h}{2} - \frac{a}{2}) \qquad \text{(TEK 14-7B)} \qquad (64)$$

$$C = 0.80 f'_m ab \qquad (65)$$

$$T = A_s f_y \qquad (66)$$

$$a = 0.80c \qquad (67)$$

$$M_n = C_m(c - \frac{a}{2}) + T(d - c) + N_n(\frac{L_w}{2} - c) \qquad \text{(NZS Masonry Manual 4.1)} \qquad (68)$$

$$C_m = 0.85 f'_m ab \qquad (69)$$

$$T = A_s f_y \qquad (70)$$

$$c = a/0.85 \qquad (71)$$

The TMS code also offers a way to make an interaction diagram for strength design including combined loading of masonry. The other codes do not have a formal procedure for a masonry interaction diagram however the other codes do have interaction diagrams for reinforced concrete that can be used but are not specific for masonry design like the TMS provides.

5.2.4 Shear

Strength shear design for TMS is based on diagonal and flexural shear failure. Sliding shear failure for the TMS is only included in the design of autoclaved aerated concrete masonry. Nominal shear strength can be calculated using these equations below included in the TMS code

[14]. The allowable shear stress again is based on the ratio of the moment divided by the shear stress times the depth of the member in the direction of the shear.

$$V_n = (V_{nm} + V_{ns})\gamma_g \qquad \text{(TMS Eqn. 9-17)} \qquad (72)$$

1) $\frac{M}{Vd_v} \leq 0.25$

a. $V_n \leq (6A_{nv}\sqrt{f'_m})\,\gamma_g$ (TMS Eqn. 9-18) (73)

2) $\frac{M}{Vd_v} \geq 1.0$

a. $V_n \leq (4A_{nv}\sqrt{f'_m})\,\gamma_g$ (TMS Eqn. 9-19) (74)

Linear interpolation is permitted and the value of *M/(Vd$_v$)* is to be taken as a positive and shall not exceed 1. To simplify the design, the TMS code allows the value of *M/Vd$_v$* to be assumed as 1. The TMS code breaks up the masonry and steel reinforcement contribution into separate equations. The next two equations are nominal masonry shear strength and nominal shear strength of the steel reinforcement, respectively for the TMS code [14].

$$V_{nm} = [4.0 - 1.75\frac{M_u}{V_u d_v}]A_{nv}f'_m{}^{\frac{1}{2}} + 0.25P_u \qquad \text{(TMS Eqn. 9-20)} \qquad (75)$$

$$V_{ns} = 0.5\frac{A_v}{s}f_y d_v \qquad \text{(TMS Eqn. 9-21)} \qquad (76)$$

Shear design for EC6 is based on flexural and diagonal shear just like the TMS code. These codes only consider two out of the three modes of failure for shear. EC6 states that a member is satisfactory under shear loading when $V_{Ed} \leq V_{Rd}$ [15]. There are two ways that can be used to calculate the shear resistance of masonry members when using EC6:

1) Minimum shear reinforcement is not provided, and any shear reinforcement incorporated in the member is ignored.

   $V_{Ed} \leq V_{Rd1}$ (EC6 1-1 Eqn. 6.33) (77)

   $V_{Rd1} = f_{vd}tl$ (EC6 1-1 Eqn. 6.34) (78)

   An enhancement may be considered in the evaluation of V_{Rd1} to allow for vertical reinforcement.

2) The minimum shear reinforcement is provided, and any shear reinforcement used is considered.

   $V_{Ed} \leq V_{Rd1} + V_{Rd2}$ (EC6 1-1 Eqn. 6.35) (79)

   $V_{Rd2} = 0.9A_{sw}f_{yd}$ (EC6 1-1 Eqn. 6.36) (80)

   When also considering shear reinforcement using EC6:

   $V_{Rd1} + V_{Rd2}/tl \leq 2.0$ N/mm^2 (EC6 1-1 Eqn. 6.37) (81)

Just like the TMS and EC6 codes, the NZS code calculates shear based on flexural and diagonal shear but also includes sliding shear failure [19]. The NZS is the only code to consider all three modes of failure for shear. Shear calculation using the NZS accounts for contributions from the masonry and reinforcement but also adds the contribution of axial stress. The NZS is also the only code that considers the contribution of axial stress when calculating shear strength. These next few equations below are used to calculate shear using the NZS code.

$$v_n = v_m + v_p + v_s \text{ (NZS Masonry Manual 4.1)} \quad (82)$$

$$v_m = (C_1 + C_2)v_{bm} \text{ (NZS Masonry Manual 4.1)} \quad (83)$$

$$v_p = 0.9\frac{N*}{b_w d} tan\alpha \text{ (NZS Masonry Manual 4.1)} \quad (84)$$

$$v_s = C_3\frac{A_v f_y}{b_w s} \text{ (NZS Masonry Manual 4.1)} \quad (85)$$

An experimental study done to compare in-plane shear models for masonry was done between eight different codes to establish the most effective model [32]. The study looked at using units with a compressive strength less than 15 MPa and greater than 25 MPa. This study included the TMS, EC6, and NZS among the eight code being studied. The results showed that designing using the NZS code produced the lowest mean factor of safety and was one of the best models at predicting how the masonry wall was going to fail. Designing using the TMS code was also very similar to NZS even though the TMS does not include sliding failure. The design using EC6 was one of the least accurate ones and would predict the wrong failure mode. The EC6 code would predict failure in flexure which would indicate that EC6 flexural shear provisions are conservative compared to the other codes [32].

5.2.5 Strength Design of Lintels

TMS, EC6, and NZS codes all have a strength design provision for reinforced masonry lintels within their codes where the IS code does not. If a reinforced lintel is needed while designing using IS then it is best to follow IS 456, which is the code for plain and reinforced concrete [35].

Strength design method for lintels using TMS assumes no axial loads, making the flexural capacity equal to the compression or tension force multiplied by the lever arm or distance between the two forces. For simplicity, when designing masonry beams the internal lever arm can be assumed as $0.9d$ [41]. Commonly, a trial area of steel is assumed and then checked or if tension controls the steel area can be calculated and same if compression were to control, equations for TMS lintel calculations can be found below.

$$M_n = A_s f_y \left(d - \frac{\beta_1 c}{2}\right) = A_s f_y \left(d - \frac{a}{2}\right) = As f_y (0.9d) \tag{86}$$

(Masonry Structural Design 6.1)

For masonry members, the EC6 has two methods for designing lintels and beams. The first method calculates the design moment for rectangular members using a lever arm like the TMS code however if an expression for the lever arm cannot estimate the amount of steel required then the code allows for the design moment to be in terms of a moment resistance factor, Q [43]. The code also states the lever arm should not be greater than $0.95d$. The moment equations for the lever arm and Q method are given as:

$$M_{Rd} = \frac{A_s f_y z}{\gamma_{ms}} \leq 0.4 \frac{f_k b d^2}{\gamma_{mm}} \text{ (E6 1-1 Eqn. 6.22)} \tag{87}$$

$$z = d\left(1 - \frac{0.5 A_s f_y \gamma_{mm}}{b d f_k \gamma_{ms}}\right) \text{ (E6 1-1 Eqn. 6.23)} \tag{88}$$

$$M_{Rd} = Q b d^2 \text{ (BS 5628-2)} \tag{89}$$

$$Q = 2c(1 - c) f_k / \gamma_{mm} \text{ (BS 5628-2)} \tag{90}$$

To design a lintel with the NZS code, the moment is taken about the neutral axis or the NZS code provides tables that specify the moment capacity for a given member based on the steel and masonry properties [19].

$$M_n = C_m(c - a/2) + T_i(d_i - c) \text{ (NZS Masonry Manual 4.1)} \quad (91)$$

$$C_m = 0.85f'_m ab \text{ (NZS Masonry Manual 4.1)} \quad (92)$$

$$T = A_s f_y \text{ (NZS Masonry Manual 4.1)} \quad (93)$$

There is not a specific design procedure for a masonry lintel in IS 1905. To design a reinforced lintel with an Indian Standard, IS 456 needs to be used. The code specifies calculating the ultimate moment and then calculating the required amount of steel needed [35].

$$M_u = 0.149 f_{ck} bd^2 \quad (for\ Fe\ 250\ steel) \text{ (IS 456)} \quad (94)$$

$$M_u = 0.138 f_{ck} bd^2 \quad (for\ Fe\ 415\ steel) \text{ (IS 456)} \quad (95)$$

$$M_u = 0.87 f_y A_{st} d(1 - \frac{A_{st} f_y}{bd f_{ck}}) \text{ (IS 456)} \quad (96)$$

One main design aspect for lintels is checking for deflection. Deflection occurs when a lintel is deformed by an applied load. Limiting deflection for masonry allows the design to control cracking in the supported masonry. The loads used to calculate deflection in lintels are unfactored loads or serviceability limit state. Table 8 shows deflection limits per the country's codes. The TMS and NZS codes are the only codes that have deflection limits specifically for masonry. The Eurocode does not give specific deflection limits for masonry; the deflections given in Table 8 for EC6 are general deflection limits that can be used to check masonry deflection. The New Zealand Standard NZS 1170 gives a table of deflection limits based on what type of element is being checked; this table can be found in NZS 1170.0 in Appendix C [13]. IS

456 does have specific deflection limits for plain and reinforced concrete and can be used to check masonry deflection as well.

Code	Deflection Limit
TMS (5.2.1.4)	L/600
EC6 (BS 5628-2 7.1.2.2.1)	L/500 or 20mm after construction
NZS (NZS 1170 Appendix C)	L/240 but <12mm
IS (IS 456 23.2)	L/350 or 20mm

Table 9: Deflection limits

Determining deflection of a lintel for all codes except Eurocode use a standard midspan deflection equation. EC6 provides a moment-curvature relationship for deflection calculations if an expression for theta cannot be made, then a standard midspan equation can be used and the procedure is like the TMS. The moment- curvature relationship for midspan deflection EC6 assumes can be found below [16]. Deflection calculations for this paper will use the standard midspan deflection for simplification and for across-the-board comparison.

$$\theta = \frac{M}{EI_u} + \frac{M - M_{cr}}{0.85EI_{cr}} \qquad (97)$$

$$M_{cr} = \frac{I_{cr} f_t}{H - d_c} \qquad (98)$$

Theta is based on the applied moment (*M*), flexural rigidity of the uncracked and cracked sections, and the cracking moment. EC6 states if a reinforced wall or beam is sized to the limiting dimensions given in section 5.5.2.5 of the code it can be assumed the vertical and horizontal deflections will be acceptable and deflection does not need to be checked [15]. New

Zealand and IS 456 also have limiting dimensions for beams but unlike the Eurocode these codes do not state that deflection is satisfied if these limitations are solely followed. The IS code states vertical deflection can be assumed satisfied if the span to depth ratios are not greater than limits given in the code based on span length and type of support. These ratios can be found in IS 456 section 23.2.1 [35]. NZS code has minimum dimensions for lintels to avoid damage by large deflections [8]. These minimum dimensions can be found in NZS 4230 section 8.3.4. The TMS code does not have limiting dimensions to follow for deflection.

For simplicity, simply supported masonry lintels and standard deflection equations will be addressed. The TMS handbook states one key feature to consider when considering deflection is tension stiffening [26]. Tension stiffening is the resistance by the undamaged masonry between flexural cracks and areas of low tensile stresses. To account for tension stiffening, the codes use an empirical equation to calculate the effective moment of inertia. Then the effective moment of inertia is used in standard deflection equations for either mid-span beam deflections or short-term deflections. The empirical equations to calculate the effective moment of inertia for all codes can be found in Table 10. The NZS code gives a table of suggested effective moment of inertias for different reinforced elements in NZS 3101 [36]. New Zealand assumes an uncracked section if the masonry's tensile flexural stress is less than $0.3f'm^{0.5}$ under sustained loading or less than $0.4f'm^{0.5}$ under short term loading [13]. NZS code also assumes the effective moment of inertia will be 40% of the net moment of inertia. Like NZS the TMS code states the effective moment of inertia is typically 50% less than the gross moment of inertia. The TMS code does state for a more accurate value the transformed cracked section or effective moment of inertia should be used. For IS 456, the effective moment of inertia is based on short term deflection [37]. The TMS, EC6 and the IS 456 consider the net moment of inertia and the cracked moment of inertia to determine the effective moment of inertia. Once the effective moment of inertia is calculated it is then used in the midspan deflection equations. Midspan deflection equations for simply supported masonry lintels with a uniform load for each code can be found in Table 11. These midspan deflections are then compared to the deflection limits in Table 9. The design deflection must be less than the deflection limit for a satisfactory design.

Code	Effective Moment of Inertia
TMS (11.1.6.2) & **EC6** (7.3)	$I_e = I_g(\frac{M_{cr}}{M})^3 + [1 - (\frac{M_{cr}}{M})^3]I_{cr}$
NZS (NZS 3101 6.8.3)	$I_e = 0.4I_g$
IS (IS 456 C-2)	$I_e = \frac{I_r}{1.2 - \frac{M_r}{M}\frac{z}{d}(1 - \frac{x}{d})}$ $M_r = \frac{f_{cr}I_{gr}}{y_t}$

Table 10: Effective Moment of Inertia

Code	Midspan Deflection
TMS	$\frac{5wL^4}{384EI_e}$
EC6	$\frac{\theta L^2}{9.6}$ $\frac{5wL^4}{384EI_e}$
NZS	$\frac{5wL^4}{384EI_e}$
IS	$\frac{5wL^4}{384EI_e}$

Table 11: Midspan Deflection Equations Under a Uniform Load

Sample calculations for each country's lintel design can be found in Appendix B at the end of this paper. A uniform compressive load of 40kN/m was applied and a compressive strength of 20 N/mm^2 was used. To compare deflection calculations for lintels amongst the codes a design of 2 #22 for reinforcement was used. A design of 2 #22 was used because the IS and NZS code required a larger area of steel compared to EC6 and TMS; this was due to the moment capacity controlling the designs for NZS and IS. Table 12 gives the calculated deflection and deflection limits per code. The calculated deflection per code was less than the deflection limit making the designs satisfactory. Figure 5 was created to see how the compressive strength of masonry influences each code's deflection calculations. Only the compressive strength of masonry was varied, everything else remained the same. The New Zealand code gives a constant for the elastic modulus where the other codes have an expression for the elastic modulus that accounts for the compressive strength of masonry. Even though the NZS code gives the elastic modulus as a constant it does allow an expression for the elastic modulus that is like the elastic modulus expression in EC6. To consider the NZS code the elastic modulus was not taken as a constant and the allowed expression given in the NZS code for elastic modulus was used. The NZS expression for the elastic modulus can be found in Table 6 in this paper. The NZS and TMS code calculated deflections that were closer compared to the other codes. The TMS and NZS code have a similar procedure to calculate deflections. The difference is in the way the codes calculate the effective moment of inertia. The TMS code uses the cracked and gross moment of inertia to calculate the effective moment of inertia where the NZS code assumes the effective moment of inertia will be 40% of the gross moment of inertia. The 40% assumption the NZS code uses is derived from many experiments and research and has proved its validity. EC6 is closer to TMS and NZS when the strength of masonry increases. The IS code calculates the largest deflections but as the strength of masonry increases the calculated deflections become closer to the other code's deflection calculations. The main difference between IS code and the others is the effective moment of inertia is calculated a bit differently. Also, the lintel design for the IS may be conservative for masonry design due to IS 1905 not having a specification on reinforced masonry lintels so a reinforced concrete beam method with masonry values was used. Concrete and masonry are similar so the results should not be too conservative.

	Code and Reinforcement Used in Lintel Design			
	EC6 2 #22	TMS 2 #22	NZS 2 #22	IS 2 #22
Deflection (mm)	2.22	1.32	2.05	5.89
Deflection Limit (mm)	L/500 = 7.32 OK	L/600 = 6.10 OK	L/240 = 15.25 OK	L/350 = 10.45 OK

Table 12: Calculated Deflections for Lintel Design

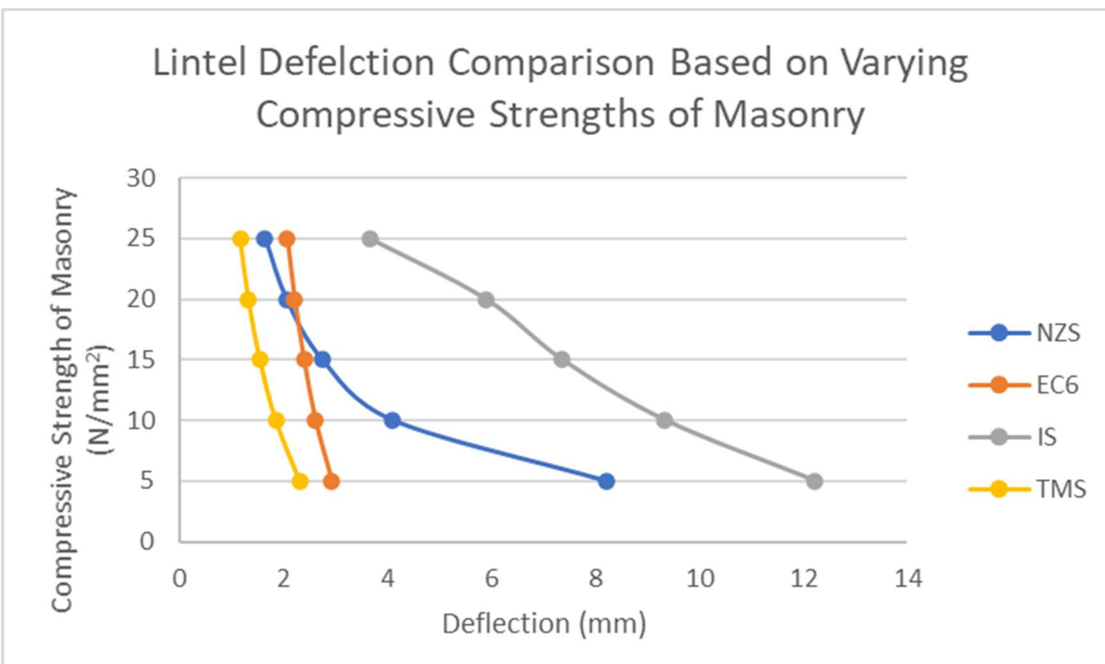

Figure 5: Lintel Deflections for Varying Strengths of Masonry Per Code

5.4 Seismic Design

Seismic design is based on the degree of ground motion within an area. The amount of ground motion a structure will be subjected to is based on factors such as soil characteristics, structure location and many others. Designing a masonry structure to withstand seismic activity requires the elements in the design to be designed for ductility. Designing the masonry elements as ductile elements will allow the masonry elements to retain their structural integrity even beyond their elastic limit [26]. Designing seismic elements for ductility will provide resistance to cyclical inelastic deformation without major loss of strength. The TMS code provides reinforcement requirements and limitations based on the seismic design category (SDC). The SDC is based on the structure's occupancy and the amount of ground motion in the area and is either based off an adopted building code or per ASCE 7 which ever one controls the design [14]. Table 13 and Table 14 outline the reinforcement requirements and SDC requirements, respectively for seismic design using TMS code [38]. Additional requirements for masonry seismic design for Eurocode can be found in Eurocode 8 (EC8), these additional requirements can be found in Table 15 [40]. Spacing requirements for reinforcement in NZS are the same for all designs and can be found in Table 16 [8]. For seismic loading, NZS 4230 has additional reinforcement requirements that focus on anchorage into columns and splices but no additional reinforcement requirements for masonry walls. Out of all the codes New Zealand has the least additional requirements for seismic design.

The NZS code has the smallest spacing requirement for vertical reinforcement with a maximum spacing of 400mm. The TMS code has the largest vertical spacing requirement with a maximum spacing of 1219mm. The TMS code also has the smallest minimum area for vertical reinforcement at 129mm^2. The minimum area for vertical reinforcement for the NZS and EC8 is

200mm^2. The NZS code has the smallest maximum spacing requirement due to having higher seismic areas to design for compared to the other countries under consideration.

Type of Shear Wall	Reinforcement Requirements	SDC Permitted In
Ordinary Plain (7.3.2.4)	None and comply with 7.3.2.3.1	A & B
Detailed Plain (7.3.2.3.1)	Min area of vertical: 129 mm^2 Provided: • At corners • Within 406 mm of each side of openings • Within 203 mm of each side of movement joints • Within 203 mm of end of wall • Max spacing 3048 mm on center. Horizontal • Min of two longitudinal wires W1.7 spaced at 406 mm or less on center. • Min of 129 mm^2 of bond beam reinforcement spaced at 3048 mm or less on center. Provided: • At bottom/top of wall openings and extends min of 610 mm past opening • Continuously at connected roof and floors • Within 406 mm of top of the wall	A & B
Intermediate (7.3.2.5)	Same as detailed plain except max spacing of vertical reinforcement is 1219 mm	A, B, & C
Special (7.3.2.6)	Spacing of reinforcement will be the lesser of:	A, B, C, D, E, & F

	• 1/3 length of the shear wall • 1/3 the height • 1219 mm masonry laid in running bond. • 610 mm masonry not laid in running bond. Min area of vertical reinforcement will make up 1/3 of shear reinforcement. Total reinforcement ratio: $\rho_t = \rho_v + \rho_h \geq 0.002$ Vertical: $\rho_v \geq 0.0007$ Horizontal: $\rho_h \geq 0.0007$ running bond $\rho_h \geq 0.0015$ other than running bond. Shear reinforcement anchored to vertical bars with a standard hook.	

Table 13: Reinforcement Requirements and Allowable Seismic Design Category for Different Types of Shear Walls Based on TMS Code

Seismic Design Category	Additional Requirements
A (7.4.1)	None
B (7.4.2)	Cannot be used with empirical design. Shear wall design must at least be a detailed plain shear wall.
C (7.4.3)	Non-participating elements: Horizontal reinforcement: • At least 2 longitudinal bed joint reinforcement spaced maximum 406mm on center for walls having a wall width of more than 102mm. • At least 1 longitudinal wire spaced maximum of 406mm on center for walls having a width of 102mm or less.

	• 406mm inches from both the top and bottom of the wall shall be provided with horizontal reinforcement. Vertical reinforcement: • Have at least 1 No 4. bar with maximum spacing of 3048mm • Within 406mm of the end of the wall vertical reinforcement shall be provided
D (7.4.4)	Vertical reinforcement: • At least 1 No 4. bar spaced at a maximum of 1219mm. • Within 406mm of the end of the wall vertical reinforcement shall be provided
E & F (7.4.5)	Nonparticipating elements: • Minimum horizontal reinforcement increased by 0.0015 multiplied by the gross sectional area of masonry. • Maximum spacing of 610mm. • Constructed out of fully grouted hollow open-end units or 2 wythes of solid units

Table 14: Additional Requirements for Seismic Design Categories Using TMS Code

Reinforcement Requirements for Eurocode (EC8 9.5.4)
• Horizontal reinforcement - maximum spacing of 600mm. • Reinforcement bar diameter shall be minimum of 4mm. • Minimum percentage of horizontal reinforcement with respect to gross area shall be 0.05% Vertical Reinforcement: • Minimum spacing of reinforcement is lesser of the bar diameter, maximum size of aggregate plus 5mm or 10mm. • Maximum spacing not to exceed 600mm.

• Minimum percentage of vertical reinforcement with respect to gross area shall be 0.08% • Minimum cross-sectional area shall be $200mm^{2.}$ • Placed at free edges of wall elements, at wall intersections, and spacing to not exceed 5m between bars.

Table 15: Additional Reinforcement Requirements for Seismic Design Using Eurocode

Reinforcement Type	Reinforcement requirements
Vertical (7.4.5.1)	• Minimum reinforcement size D12 • Minimum cross-sectional area 200mm • Maximum spacing 400mm • Minimum number of bars - 4
Horizontal (7.4.5.2)	• Maximum spacing for less than three stories or 12m – 400mm • Maximum spacing for greater than three stories or 12m- 200mm • Reinforcement is not to be lapped within the greater of 600mm or $0.2L_w$

Table 16: Requirements for Seismic Design Using NZS Code

Shear walls play a major role in resisting seismic forces. Sample calculations for designing a shear wall based on EC6, TMS and NZS were done for comparison. The IS 1905 code does not include a design procedure for a reinforced masonry shear wall and was not included in this comparison. Detailed calculations for each codes shear wall design can be found in Appendix B and a summary of the results can be found in Table 17. The designs given in Table 17 follow the additional seismic requirements for reinforcement. The TMS design has the largest spacing for reinforcement due to breaking seismic areas up into different categories. The NZS and EC6 code do not have seismic design categories and make reinforcement parameters the same for all seismic designs no matter the location.

Code	Design
EC6	Flexural reinforcement: #19 @ 600mm Shear reinforcement: #10 @ 300mm
TMS	Flexural reinforcement: #19 @ 1200mm Shear reinforcement: #16 @ 1000mm
NZS	Flexural reinforcement: #19 @ 400mm Shear reinforcement: #10 @ 200mm

Table 17: Shear Wall Design Results for EC6, TMS, and NZS

Seismic design is all about making sure the structural elements have enough resistance to hold structural integrity for ground motion. Designing elements for ductility involve using some factors. For the TMS code the response modification coefficient, R, is like the EC6 codes behavior factor, q and NZS 4230 structural ductility factor, μ [39][8]. All these factors deal with a ratio between a completely elastic design and a nominal strength elastic design. The response modification coefficient, R, is termed as the ratio between required strength and structural integrity, this term reduces the linear elastic response spectra to inelastic ones and can be found in ASCE 7. The behavior factor, q, according to Eurocode is a ratio between the seismic forces the structure would experience if the systems behavior were completely elastic and the seismic forces at the ultimate limit state [40]. The structural ductility factor for NZS is also defined like EC6. Table 18, 19, 20 show values for R, q and μ, respectively.

Type of Shear Wall	Response modification factor
Special	5.0
Intermediate	3.5
Ordinary reinforced & detailed plain	2.0
Ordinary plain	1.5

Table 18: Response Modification Factor Using TMS Code

Type of Construction	Behavior Factor
Unreinforced masonry following EN 1996 alone (low seismic only)	1.5
Unreinforced masonry following EC8	1.5 to 2.5
Confined masonry	2.0 to 3.0
Reinforced masonry	2.5 to 3.0

Table 19: Behavior Factor for EC6 Code

Design Philosophy	Structural Ductility Factor
Elastic	1.0
Nominally ductile	1.25
Limited ductile	2.0
Ductile	4.0

Table 20: Structural Ductility Factor for NZS Code

5.4.1 Boundary Elements

Boundary elements are thickened sections provided at the end of walls to allow for special reinforcement. The TMS 402 code section 9.3.6.6.1 states a boundary element does not need to be provided in shear walls if [14]:

1. $P_u \leq 0.10 A_g f'_m$ (symmetrical walls) (99)

   $P_u \leq 0.05 A_g f'_m$ (unsymmetrical walls) (100)

   AND

2. $\frac{M_u}{V_u d_v} \leq 1.0$ (101)

   OR

3. $V_u \leq 3 A_{nv} \sqrt{f'_m}$ and $\frac{M_u}{V_u d_v} \leq 3.0$ (102)

If a shear wall does not meet any of these requirements, then boundary elements will need to be provided and is designed using TMS 402 section 9.3.6.6.3 or 9.3.6.6.4.

The New Zealand standard's alternative to boundary elements are horizontal confining plates. To increase ductility metal plates are placed in critical sections of the mortar beds within the plastic hinge region. Unconfined masonry has an ultimate compression strain of 0.003 when confining plates are added the ultimate compression strain increase to 0.008 [8]. Requirements for designing confining plates can be found in NZS 4230 section 7.4.6.5. Eurocode 6 or 8 does not address boundary elements for masonry design but in Eurocode 8 in the reinforced concrete section boundary elements are addressed and can be found in EC8 in section 5.4.3.4.2 [40].

CHAPTER 6: CONCLUSION

6.1 Summary

Four masonry codes from four different countries were analyzed and compared. The four codes under consideration were TMS 402 from the United States, EC6 from Europe, IS 1905 from India and NZS 4230 from New Zealand. The analysis was broken down into unreinforced and reinforced and based on the limit state of the overall design, either allowable stress design or strength design.

For general masonry design, EC6 uses a characteristic compressive strength of masonry where the other codes use the nominal compressive strength of masonry stated from prism test data. The TMS code is the only code to have separate equations for the elastic modulus of concrete and clay and the NZS code takes the elastic modulus as a constant.

Unreinforced masonry is not as common as it was when it first emerged as a structural material. Some places still design using unreinforced masonry and some areas still have buildings standing prior to the use of reinforcement. For unreinforced ASD, the codes included were TMS 402 and IS 1905. The TMS 402 calculates a higher capacity than IS 1905 due to use of nominal compressive strength of masonry compared to the basic compressive stress used in the IS code. The TMS code also has higher allowable stresses compared to the IS code. When considering eccentricity, the IS code considers eccentricity and slenderness ratio together in equations where the TMS uses a separate buckling equation. For unreinforced strength design, the codes that include this design are EC6 and TMS. EC6 has a more accurate bearing capacity compared to the TMS. However, the TMS code has more accurate shear provisions than EC6. The percentage of error compared to experimental results are lower for EC6 than TMS. EC6 also considers partial material factors where the TMS code uses a strength reduction factors.

The only code that uses ASD for reinforced masonry is TMS 402. This method is like the unreinforced ASD used in TMS just with the added contribution of the steel reinforcement. The TMS does not directly consider eccentricity but is checked with a separate equation. Designing with ASD can be conservative compared to SD. For strength design of reinforced masonry, the codes that use this method are EC6, NZS 4230 and TMS 402. EC6 has a more accurate capacity expect in shear compared to TMS and NZS. NZS code predicts a more accurate shear however the TMS calculates values close to the NZS code as well. Partial material factors are used in EC6 design and strength reduction factors are used in TMS and NZS designs.

6.2 Recommendations for Future Work

For a better understanding of how each code impacts masonry design and what would be the most accurate code at calculating the capacity more research and tests would need to be done specifically looking at the codes and comparing them to testing results. There are studies that have done this but not to the extent that could unify the design of masonry structures. Experimental tests need to be done based on similar design procedures, some of the experiments out there take allowable stress design in the US and compare it to Europe’s ultimate state design. Using more masonry construction versus any other type of structural material can be viewed as eco-friendlier and more sustainable. Having a better understanding and more unified design procedure and code could increase the structural use and application of masonry in today’s modern world.

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APPENDICES

APPENDIX A

UNREINFORCED SAMPLE CALCULACTIONS

Sample Calculations for Unreinforced Allowable Stress Design

Allowable Axial Stress

Given for TMS calculation:

203x203 CMU

H=3m
r= 72.84mm
$f'm$= 20 N/mm^2
h/r= 41.19 < 99 so TMS equation 8-16 applies

$F_a = 0.25f'm[1- (h/140r)^2] = 0.25(20)[1- (3000mm/(140 \times 72.84mm))^2] = 4.57$ N/mm^2

Allowable axial stress = 4.57 N/mm^2

Given for IS axial capacity calculation:

M1 mortar
203x203 CMU
t=190mm
L= 3000mm
H= 3000mm
Unit strength = 20 N/mm^2

Allowable axial stress IS 5.4: $f_c = f_b k_s k_a k_p$

Effective height (IS 4.3):
H_{eff}=0.75h = 0.75 x 3000mm = 2250mm

Effective length (IS 4.4):
L_{eff} = 0.8L = 0.8 x 3000mm = 2400mm

Since $H_{eff} < L_{eff}$ slenderness ratio is controlled by h/t

SR = h/t = 3000mm / 190 mm = 15.79

From Table 9 (Stress Reduction Factor) in IS:
from interpolation
$k_s = 0.735$

From Table 10 (Shape modification):

$k_p = 1.0$

From Table 8 (Basic Compressive Stresses):

$f_b = 2.20\ N/mm^2$

$k_a = 0.7 + 1.5A$

$A = tL = 190mm \times 3000mm = 570\ m^2 > 0.2\ m^2$ so $k_a = 1.0$

$f_c = f_b k_s k_a k_p = 2.20\ N/mm^2 \times 0.735 \times 1.0 \times 1.0 = 1.62\ N/mm^2$

Combined Axial and Flexural Capacity Calculations for TMS and IS Code

Given for TMS combined loading calculation:

Finding maximum wind load:

$P = 1000\ N$
$A_n = 0.0194\ m^2$
$S = 0.00472\ m^3$
$e=0$
$h=3m$
$F_t = 228\ kPa$
$M = wh^2/8 + Pe/2 = w(3m)^2/8 = 1.125w$

$-f_a + f_b = F_t$

$228000\ N/m^2 = -1000/0.0194 + 1.125w/0.00472$

$W = 1172.85\ N/m^2$

$f_a/F_a + f_b/F_b \leq 1.0$ (TMS 8-14)

$f_a = 51546.4\ N/m^2$

$F_a = 3450000\ N/m^2$

$f_b = M/S = 1.125(1172.85\ N/m^2)/0.00472 = 279546$

$F_b = 4600000\ N/m^2$

$f_a/F_a + f_b/F_b \leq 1.0$

$0.08 < 1.0$ OK

Cannot increase wind load because it's based on allowable tension

Given for IS combined loading calculation:

Finding maximum wind load: **(IS 5.5.3)**
P=1000N
A_n= 0.0194 m^2
S_n = 0.00472 m^3
e= 0
h= 0

$f_t = f_b - f_a$

f_t = 7000 N/m^2 (IS 5.4.2)

$M = wh^2/8 = 1.125w$

$7000\ N/m^2 = 1.125w/0.00472 – 1000/0.0194$

$w = 245.64\ N/m^2$

$f_{allowable} = f_c + f_b$

$= P/A + M/S$

$= 1000/0.0194 + 245.64/0.00472$

$= 0.104\ N/mm^2$

Shear Capacity Calculations for TMS and IS Code

Given for TMS shear calculation:

$S = 2 \times 0.03175m^2 \times (4m)^2 / 6 = 0.169\ m^3$

$A = 0.03175m \times 2 \times 4m = 0.254\ m = 254mm^2$

$M_B = Vh = V \times 3m = 3V$

$f_b = M_B/S = 3V/0.169 = 17.75V$

APPENDIX A (continued)

$f_a = P/A = 1000/0.254 = 3937.01\ N/m^2$

$-f_a + f_b = F_t$

$-3937.01 + 17.75V = 228000\ N/m^2$

$V = 13066\ N$

$f_a/F_a + f_b/F_b \leq 1.0$

$F_b = 1/3 \text{ x } f'm^{0.5} = 1/3 \text{ x } 13.79^{0.5} = 4.60\ N/mm^2$

$F_a = 0.25f'm[1- (h/140r)^2] = 0.25 \text{ x } 13.79[1 - (3/140 \text{ x } 72.84)^2] = 3.48\ N/mm^2$

$(0.0039/3.48)N/mm^2 + (1.775 \text{ x } 10^{-5}V/4.60)N/mm^2 = 1.0$

$V = 258864\ N$

So shear calculated by tension controls $V = 13066\ N$

$f_v = (3/2)\ V/A = (3/2)\ 13066/254 = 0.0772\ N/mm^2$

$F_v = 0.125f'm^{0.5} = 0.46\ N/mm^2$

$= 0.827\ N/mm^2$

$= 225 + 0.45N_v/A_v = 225 + 0.45(1000/254) = 226.7\ N/mm^2$

$f_v < F_v$ OK

Given for IS shear calculation:

When designing shear for IS the design is based on having no tension stresses.

$M=Vh= 3V$

$t=190mm$

$A= 0.254m^2$

$S= 0.19m \text{ x } 4m^2 / 6 = 0.169m^3$

$f_t \leq f_b - f_a$ since design based on no tension $f_t = 0$

$f_t = M/S - P/A$

$0 = 3V/0.169 - 1000/0.254$

APPENDIX A (continued)

$V = 221.79$ N

Allowable shear (IS 5.4.3)

$f_a = P/A = 1000/254000 = .0039$ N/mm^2

$f_s = 0.1 + f_a/6 \quad f_{smax} \leq 0.5$

$= 0.1 + 0.0039/6 = 0.1$ N/mm^2

$f_v = (3/2)V/A = (3/2) \times (221.79/254000) = 0.0013$ N/mm^2

$f_v < f_s$ OK

Sample Calculations for Unreinforced Strength Design

E6 Axial Capacity

Given:
Group 1 CMU
Block dimensions: Nominal (mm) 400 x 200 x 200
Actual (mm) 390 x 190 x 190
Class 2
General purpose mortar
$f_b = 20$ N/mm^2
$t = 190$mm
$f_m = 10$ N/mm^2 (E6 part 1-1 section 3.2.3.1) $f_m > 4$ N/mm^2

Calculate characteristic strength of masonry f_k

(eqn 3.2 E6 part1-1) $f_k = K f_b^{0.7} f_m^{0.3} = 0.55 \times 20^{0.7} \times 10^{0.3} = 8.93$ N/mm^2

from table 3.3 in E6 Part 1-1: K= 0.55

Checking capacity:

Effective height:

(eqn 4.6 E6 Part 1-1) $h_{ef} = \rho_n h = 0.75 \times 3000 = 2250$ mm

APPENDIX A (continued)

ρ_n= reduction factor where

= 1.0 if the wall is acting as end support to a floor

= 0.75 for all other walls

Effective thickness: t_{ef}= 190mm

Slenderness ratio = h_{ef} / $t_{ef} \leq 27$

= 2250 / 190 = 11.84 < 27 OK

Reduction for slenderness and eccentricity (E6 Part 1-1 section 6.1.2.2)

$\Phi_i = 1 - 2\, e_i / t$ (E6 part 1-1 eqn 6.4)

e_i = initial eccentricity accounts for construction imperfections assumed as $h^{ef}/450$ (E6 part 1-1 section 5.5)

$e_m = (M_{id} / N_{id}) + e_{he} \pm e_i$

$= (M_{id} / N_{id}) + e_{he} + h^{ef}/450$

$= 0 + 0 + 2250 / 450 = 5.0$ mm

e_{he}= eccentricity due to lateral loads

$e_{mk} = e_m + e_k \geq 0.05t$ e_k= eccentricity due to creep

(6.1.2.2 states if slenderness ratio < λ_c = 15 then e_k may be taken as 0)

e_{mk}= 5.0 + 0 = 5.0 mm < 0.05t = 0.05(190) = 9.5 mm

So e_i = 0.05t

$\Phi_i = 1 - 2\, e_i / t = 1 - 2\,(0.05t / t) = 0.9$

From Annex G: $\Phi_m = A_1\, e^{-u\text{^}2/2}$

$A_1 = 1 - 2\, e_{mk} / t = 0.9$

APPENDIX A (continued)

For E= $1000f_k$:

$U = ((h_{ef} / t_{ef}) -2) / (23 – 37 (e_{mk}/t)) = (2250/190) – 2 / (23 – 37(0.05t/t)) = 0.47$

$\Phi_m = A_1 e^{-u^2/2} = 1.01$

So $\Phi = 0.9$

For Group 1 Class 2 units: Material property γ_M (E6 part 1-1 section 2.4)

$\gamma_M = 2.7$

Design resistance: $N_{Rd} = \Phi\ t\ f_d$ (eqn 6.2 E6 part 1-1)

$f_d = f_k / \gamma_M = 8.93 / 2.7 = 3.31 N/mm^2$

$N_{Rd} = 0.90 \times 190 \times 3.31 = 566$ kN/m

TMS Axial Capacity

8” CMU block
$f'_m = 13.79 N/mm^2$
h= 3000 mm
$A_n = 63{,}510 mm^2/m$
$I_n = 421{,}557{,}015 mm^3/m$
$r = (I_n / A_n)^{0.5} = 81.47$ mm
$\phi = 0.60$

h/r = 3000 mm / 81.47 mm = 36.82 < 99
so $P_n = 0.8[0.8A_nf'_m(1-(h/140r)^2)]$ (eqn 9-11)

$P_n = 0.8[0.8(63{,}510 mm^2/m)(13.79 N/mm^2)(1 – (3000 /(140 \times 81.47)^2)] = 521.74$ kN/m

$\Phi P_n = 0.60(390.86$ kN/m$) = 313.04$ kN/m

Flexural Capacity

E6

Given:
Group 1 CMU

APPENDIX A (continued)

Block dimensions: Nominal (mm) 400 x 200 x 200
Actual (mm) 390 x 190 x 190
Class 2
General purpose mortar
f_b= 20 N/mm^2
t= 190mm
f_m= 10 N/mm^2 (E6 part 1-1 section 3.2.3.1)

γ_M = 2.7

Design moment:
Failure to be parallel to bed joints
E6 part 1-1 section 3.6.3
f_{xd1}= 0.10 N/mm^2

$M_{Rd} = f_{xd}Z$

$= (f_{xd1} \text{ x } Z) / \gamma_M$

= 0.10 N/mm^2 x (190 mm)^2 x10^{-3}\ (2.7 x 6) = 0.22 kN-m/m

Design moment:
Failure to be perpendicular to bed joints
f_{xd2} = 0.40 N/mm^2

$M_{Rd} = f_{xd}Z$

$= (f_{xd2} \text{ x } Z) / \gamma_M$

= 0.40 N/mm^2 x (190 mm)^2 / (2.7 x 6) = 0.89 kN-m/m

TMS Flexural Capacity

Modulus of Rupture F_r

S = 81 in^3/ft = 4354803 mm^3/m

$f_a + f_b \leq f_r$

$M / S \leq f_r$

$M = f_r \text{ x } S$

APPENDIX A (continued)

Design moment: (flexural tensile stresses parallel to bed joints)

f_r = 579 kPa

$M = f_r \times S = 579000 N/m^2 \times (0.004355 m^3)$= 2.52 kN-m

$\phi M = 0.60(2.52$ kN-m/m) = 1.51 kN-m/m

Design moment: (flexural tensile stresses perpendicular to bed joints)

f_r = 1149 kPa

$M = f_r \times S = 1149000 N/m^2 \times (0.004355 m^3)$ = 5.00 kN-m/m

$\phi M = 0.60(5.00$ kN-m/m) = 3.0 kN-m/m

Combined Axial and Flexural

E6

Section 6.3.1

$\sigma_d \leq 0.2 f_d = 0.2 \times 2.51 N/mm^2 = 0.502 N/mm^2$

$f_{xd1\text{-}app} = f_{xd1} + \sigma_d$

$Z = 190^2 \times 10^3 / 6 = 6.0 \times 10^6 mm^3$

$M_{Rd} = ((f_{xk1} / \gamma_M) + \sigma_d)Z$

$= [(0.19 N/mm^2 / 2.7) + 0.502 N/mm^2] \times 6.0 \times 10^6 mm^3$

= 3420 N-m/m = 3.42 kN-m/m

TMS Combined Axial and Flexural Capacity

$f_a + f_b \leq f_r$

$P/A + M/S \leq f_r$

$M = (f_r – P/A) \times S$

$= [11490 kN/m^2 – (420.38 kN / 0.06351 m^2)] \times 0.00435 m^3$

= 21.19 kN-m/m

APPENDIX A (continued)

Shear Capacity

E6

L= 4m
H=4m
f_k= 6.79 N/mm^2
Class 2
f_b = 20 N/mm^2
t= 190mm
f'm= 4 N/mm^2

M_{Rd}= 0.42 kN-m/m

f_{vk}= f_{vko} + 0.4σ_d (ignoring self-weight)
f_{vk} = f_{vko} = 0.20 N/mm^2 (Table 3.4 E6 part 1-1)
f_{vd} = f_{vk} / γ_m = 0.20 / 2.7 = 0.07 N/mm^2
V_{Rd} = f_{vd} x t x l = 0.07 N/mm^2 x 190 mm x 4000 mm = 53.2 N/m

TMS
$M_u = V_u h$
$f_b = M/2 = V_u h/2$
$f_t = f_b - f_a$
366000 N/m^2 = V_u(4m)/2
V_u = 183.0 kN/m

APPENDIX B

REINFORCED MASONRY SAMPLE CALCULATIONS

TMS code:

Given: L= 1.8m   t=140mm   #4 @ 400mm f'_m= 12MPa   D= 12.7mm

Axial:

$$P_n = 0.80[0.80f'_m(A_n - A_{st}) + f_y A_{st}]$$

A_{st}= πD^2/4 = 126.68 mm^2

P_n= 0.80[0.80*12(140*400-126.68) = 429 kN

ϕP_n= 0.9(429107/1.8) = 214.55 kN

Flexural:

d= 140/2= 70mm

a= $A_s f_y$/0.8f'_mb = 126.68*300/0.8*12*400 = 9.90 mm

M_n= $A_s f_y$(d-a/2) = 126.68*300(70-9.90/2) = 2472.16 kN-m

ϕM_n= 0.9* (2472.16/1.8) = 1236.08 kN-m

Combined:

c/d-c = $\varepsilon_{mu}/\varepsilon_y$

c= d($\varepsilon_{mu}/\varepsilon_{mu}$ + ε_y) = 70(0.0025/(0.0025 + 0.00207)) = 38.5 mm

T= $A_s f_y$ = 126.68*300= 38004 N

C= 0.80f'_m(β_1c)b= 0.80(12)(0.80*38.5)*400= 118272 N

P_n= C-T= 80268 N

ϕP_n= 40134 N

M_n= T(d-h/2) + C(h/2-β_1c/2) = 38004(70-140/2) + 118272(140/2 – (0.8*38.5)/2)

=6457.65 kN-m

ϕM_n= 0.90(3587.58) = 3228.83 kN-m

APPENDIX B (continued)

NZS code:

Given: L= 1.8m   t=140mm   #4 @ 400mm f'_m= 12MPa   D= 12.7mm

Axial:

$$N_{nw} = 0.5f'_m A_g[1 - (\frac{L_n}{40b})^2]$$

N_{nw} = 0.5(12)(140*400)[1- (400/40*140) = 334.29 kN

Combined:

T = 4(126.68)(300)= 152.01 kN

C_s= 126.68(300)= 38 kN

$C_m + C_s = T + N_n$

$C_m = T + N_n – C_s$

C_m= 0.85f'_mab

0.85(12)a(140) = 152.01+135 -38

a= 249.01 x 10^3/0.85(12)(140)= 174.38 mm

c= 174.38/0.85= 205.15mm

ε_s/c-100 = ε_m/c

ε_s= 0.003/205.15(205.15-100) = 0.001538

M_n= C_m(C-a/2) + T(d-c) + N_n(L_w/2 -c)

C_m= 0.85(12)(174.38)(140)= 249.02 N-mm

M_n= 249.02(205.15-(174.38/2))+38(205.15-100)+38(500-205.15)+38(900-205.15)+38(1300-205.15)+38(1700-205.15)+135(1800/2 -205.15)= 263.19 kN-m

E6

Group 1
f_b= 20 N/mm^2
H= 3m
L= 6m
Lateral load= 225kN
M_{ED}= 225kN(3m)= 675 kN-m

APPENDIX B (continued)

$f_k= Kf_b^{0.7}f_m^{0.3} = 0.55(20N/mm^2)^{0.7}(10\ N/mm^2)^{0.3}= 8.93\ N/mm^2$
$f_d= f_k/\gamma_m= 8.93/2.7 = 3.31\ N/mm^2$

$Q= M_{ED}/db^2 = 675$ kN-m(1000 N/kN)(1000mm/m)/(194 x 6000^2) = 0.09665
$Q= 2c(1-c)f_d$ Solve for c
c= 0.9852
z= cl= 0.9852(6000mm)= 5911.2mm
$f_{yd}= f_y/\gamma= 500/1.5= 435\ N/mm^2$
$A_s= M_{RD}/f_{yd}z$ = 675 kN-m(1000N/kN)(1000mm/m)(0.9)/(435 x 5911.2)= 236.26 mm^2

Use #19 @ 600mm

TMS lintel:
203x203 CMU
Nominal= 203.20 mm
Actual= 193.80 mm
$f'_m= 20\ N/mm^2$
L= 3.66m
$W_u= 40$ kN/m
$f_y= 413.69\ N/mm^2$
$E_m=18\ kN/mm^2$
$E_s= 200\ kN/mm^2$
n= 11.11

$M_u= WL^2/8= 40(3.66)^2/8= 66.98$ kN-m
$V_u= WL/2= 40(3.66)/2 = 73.20$ kN

Try 4 courses with 2 #16(#5)
h= 4(203.20)= 812.80mm
d= (4-1)(203.20)+ 193.80- 193.80/2= 706.50mm
$A_s= 2(200) = 400mm^2$
$a= A_sf_y/0.8f'mb$
a= 400(413.69)/(0.8*13.79*193.80)= 49.92mm

M_n= 400(413.69)(706.50-(49.92/2))= 112.78 kN-m
ϕM_n= 0.9(112.78)= 101.50 kN-m > M_u OK

f_r= 1.84 N/mm^2
S_n= 193.80(812.80)^2/6= 21338796.03 mm^4
M_{cr}= (1.84) 21338796.03= 39.28 kN-m
1.3M_{cr}= 1.3(39.28)= 51.07 kN-m < ϕM_n OK

Shear:
V_{nm}=(4-1.75*1)193.80(706.50)13.79$^{0.5}$= 1144 kN
V_{ns}= 0 (no shear reinforcement)
V_n= 1144 +0 = 1144 kN
ϕV_n= 0.8(1144)= 915.21 kN > V_u OK

Deflection based on 2#22:

ΣM=0
193.80x(x/2)=nA_s(d-x)
x= 209.92mm

I_g= bh^3/12 = 193.80(812.80)^3/12= 8672086707.40 mm^4

I_{cr}= (bx^3/3) + nA_s(d-x)^2
=193.80(209.92)^3/3 + 11.11(774)(706.50-209.92)^2= 2.72 x 10^9 mm^4

I_e= (M_{cr}/M_a)^3I_g + (1- (M_{cr}/M_a)^3)I_{cr}
=(39282.59(1000)/(48559.05*1000))^3(8672086707.40) + (1-(39282.59(1000)/(48559.05*1000))^3) 2.72 x 10^9 = 3.92 x 10^9 mm^4

Δ= 5(40/1000)(3.66*1000)^4/(384*18*3.92 x 10^9)
= 1.32mm < L/600= (3.66*1000)/600= 6.10mm OK

E6 lintel:
Aggregate Group 1 CMU
M12 Mortar
Nominal= 203.20 mm
Actual= 193.80 mm
L= 3.66m
W_u= 40 kN/m
f_y= 413.69 N/mm^2

d=812.80 mm
L_{ef}= 3.66+ 706/1000= 4.47m
f_b=20 N/mm^2
f_m=12 N/mm^2
γ= 1.15

K= 0.55 (Table 3.3 BS EN 1996)
$f_k= Kf_b^{0.7}f_m^{0.3} = 0.55(20^{0.7})12^{0.3} = 9.44$ N/mm^2
$f_d= f_k/\gamma = 8.21$ N/mm^2

$M_{ed}= WL^2/8 = (40*4.47^2)/8 = 100.03$ kN-m
$V_{ed}= WL/2 = (40*4.47)/2 = 89.46$ kN

$Q=M_{ed}/bd^2$
$=100.03x10^6/(203.20*812.80^2)= 0.7451$
$M_{ed}/bd^2= 2c(1-c)f_d$ Solve for c
c=0.6
z=0.6(812.80)= 487.68mm
f_{yk}= 500 N/mm^2
f_{yd}= 500/1.15= 434.78 N/mm^2
$A_{sreq} = 100.03x10^6/(434.78*487.68)= 471.76$ mm^2

2 #19 A_s=2*284= 568mm^2
$E_m= K_ef_k = 1000(9.44)= 9437.09$ N/mm^2
E_s= 199950 N/mm^2
n= 21.19
x= 295.63mm

$f_{xd1app}= f_{xd1} + 0.2f_d = 0.10 + 0.2(8.21) = 1.74$ N/mm^2
$M_{cr}= I_{cr}f_{xd1app}/(H-z) = 1.51 x10^9(1.74)/(812.80-487.68) = 8083690.41$ N-mm
$M_{Rd}= A_sf_yz/\gamma_m = 568(413.69)487.68/1.15 = 9.96 x10^7$ N-mm > M_{Ed} OK

$I_g= 193.80*706.50^3/12= 5.70 \times 10^9$ mm^4
$I_{cr}= 193.80*295.63^3/3 + 21.19*774(706.50 – 295.63)^2= 4.44 \times 10^9$ mm^4
$I_e = 4.47 x10^9$ mm^4

$\Delta= 5wL^4/384EI = 2.22$mm < L/500=7.32mm OK

APPENDIX B (continued)

NZS lintel:

Nominal= 203.20 mm
Actual= 193.80 mm
f'_m= 13.79 N/mm^2
L= 3.66m
W_u= 40 kN/m
f_y= 300 N/mm^2
h=706.50mm
2 #22 A_s= 774mm^2
A_n= 706.50*203.20= 136919.7 mm^2
$p=A_s/A_n$ = 774/136919.7= 0.0057
$p(f_y/f'_m)$= 0.085
From table 2 in NZs masonry manual:
M_n/f'_mLwt= 0.0387
M_n= (0.0387*20*706.50^2*193.80)/1000^2= 74.90 kN-m > M_u=66.98 kN-m OK

I_g= 5.70 x 10^9 mm^4
I_e= 0.4*I_g= 2.28 x 10^9 mm^4
E= 1000f'_m= 1000(20)/1000= 20 kN/mm^2

Δ= 5wL^4/384EI
= (5*(40/1000)*(3.66*1000)^4)/(384(20)2.28x10^9 = 2.05mm < L/240= 15.25mm OK

IS lintel:

Nominal= 203.20 mm
Actual= 193.80 mm
f'_m= 20 N/mm^2
L= 3.66m
W_u= 40 kN/m
f_y= 413.69 N/mm^2
d=812.80mm
L/d= 3660/812.80 = 4.50 < 20 for simple support deflection should be OK
d_{ef}= 3.66*1000/10= 366 mm
M_u= 66.98 kN-m
d_{req}= M_u/0.138f_m*b= (66.98(1000^2)/(0.138*13.79*193.80))$^{0.5}$= 426.15mm
2 #22 A_{st}= 2*387= 774mm^2
E_s= 200,000N/mm^2
E_m= 550*13.79= 7584.5 N/mm^2
f_{cr}=0.7*13.79$^{0.5}$= 2.60 N/mm^2
M_{cr}= $f_{cr}I_g/y_t$= 1.97 x 10^7 N-mm

$z = 0.95d = 347.7mm$
$y_t = 0.87f_yA_{st}/0.36f_mb = 104.35mm$
$I_g = 7.92 \times 10^8\ mm^4$
$I_{cr} = 1103655256\ mm^4$
$I_e = 1103657589\ mm^4$
$\Delta = 5.89\ mm > L/350 = 10.46mm$ OKAY

Seismic Design Examples

TMS:
$h = 3m$
$L = 6m$
$f'_m = 13.79\ N/mm^2$
$f_y = 413.69\ N/mm^2$
$D = 10\ kN/m$ $E = 225\ kN$ $S_{DS} = 0.5$
Assumed wall weighs $2.15kN/m^2$
Wall Weight= $2.15kN/m^2 \times 6m \times 3m = 38.7\ kN$
$W_u = 10kN/m \times 6m + 38.7kN = 98.7\ kN$
$P_u = (0.9 - (0.2)(0.5)) \times 98.7kN = 78.96\ kN$
$M_u = 225kN(3m) = 675kN\text{-}m$
$d = 0.9(6m \times 1000mm/m) = 5400mm$

$a = d - (d^2 - [2(P_u(d-d_v/2) + M_u]/\phi(0.8f'_mt))^{0.5} = 5400mm - (5400^2 - [2*78.96kN(5400mm-6000/2mm) + 675000kN\text{-}mm]/(0.9*0.8*0.01379kN/mm^2*194mm) = 83.76mm$

$A_s = (0.8f'_mta - P_u/\phi)/f_y = (0.8*0.01379kN/mm^2*194mm*83.76mm - (78.96kN/0.9))/0.41369kN/mm^2 = 221.27mm^2$

Use #19 @ 1200mm o.c

7.3.2.6.1.2
$V_u = 1.5V_u = 1.5*225 = 337.5kN$
$M_u/V_ud_v = 1.0$
$A_{nv} = 2*32mm*6000mm + 5*203mm\ x(194mm - 63.5mm) = 516457.5mm^2$

$V_{max} = (4/3)[5 - 2M_u/V_ud_v]A_{nv}f'_m{}^{0.5} = 767.14kN$
$V_{nm} = (4 - 1.75(M/Vd_v))A_{nv}f'_m{}^{0.5} + 0.25P_u = 433.49kN$
$V_{ns} = V_u/\phi\gamma_g - V_{nm} = 129kN$

#16 $A_{st} = 200mm^2$

$s= (0.5A_{st}f_y d)/V_{ns}= 1924mm$

Maximum spacing requirements (Section 7.3.2.6)
min h/3 lesser of wall length or wall height or 1220mm

= 3000/3= 1000mm
=6000/3= 2000mm
=1220mm

$S_{max}= 1000mm$
Shear Reinforcement:
Use #16 @ 1000mm

Eurocode 6
Group 1
$f_b= 20\ N/mm^2$
H= 3m
L= 6m
Lateral load= 225kN
$M_{ED}= 225kN(3m)= 675\ kN\text{-}m$

$f_k= Kf_b^{0.7}f_m^{0.3} = 0.55(20N/mm^2)^{0.7}(10\ N/mm^2)^{0.3}= 8.93\ N/mm^2$
$f_d= f_k/\gamma_m= 8.93/2.7 = 3.31\ N/mm^2$

$Q= M_{ED}/db^2 = 675\ kN\text{-}m(1000\ N/kN)(1000mm/m)/(194 \times 6000^2) = 0.09665$
$Q= 2c(1-c)f_d$ Solve for c
c= 0.9852
z= cl= 0.9852(6000mm)= 5911.2mm
$f_{yd}= f_y/\gamma= 500/1.5= 435\ N/mm^2$
$A_s= M_{RD}/f_{yd}z = 675\ kN\text{-}m(1000N/kN)(1000mm/m)(0.9)/(435 \times 5911.2)= 236.26\ mm^2$

Use #19 @ 600mm

$V_{ED}= 225(1.5)= 337.5kN$
$V_{ED} \leq V_{Rd1} + V_{Rd2}$
$V_{Rd1}= f_{vd}tl$
(J.1) $f_{vd}= (0.35+17.5\rho)/\gamma_M$
$\rho= A_s/bd = 284mm^2/(194mm \times 6000mm)= 0.000244$

f_{vd}= (0.35+17.5(0.000244))/1.5 = 0.23618 N/mm^2
f_{vd} < 0.7/γ_M= 0.7/1.5= 0.4667 N/mm^2 OK

V_{Rd1}= f_{vd}tl= 0.23618 N/mm^2 (194mm)(6000mm)= 274.91kN < 337.5kN Needs shear reinforcement

V_{Rd2}= 0.9d$A_s f_{yd}$ s_{max}= 300mm or 0.75d
f_{yd}= f_y/γ_m= 200/1.5= 133.33N/mm^2
V_{Rd2}= 0.9(6000mm)A_s(133.33N/mm^2)= 719.98A_s

274.91kN + 719.98A_s ≥ 337.5kN
A_s= 0.0869mm^2
8.2.3(5) Minimum reinforcement 0.05%: =0.0005(194mm)(397mm)=38.51mm^2
Use #10 @ 300mm

NZS

f'_m= 20 N/mm^2
E= 225kN
H= 3m
L= 6m
#19 @ 400mm
d= 0.8(2000mm)= 1600mm
b_w= 194mm
f_y= 300 N/mm^2

$V_n \geq E/\phi$

=225 kN/0.75= 300kN

V_{nMax}= V_n/b_wd = 300kN(1000 N/kN)/(194mm x 1600mm)= 0.9665 N/mm^2

p_w= 5(200mm^2)/(194mm x 1600mm)= 0.0032
C_1= 33p_wf_y/300= 0.1063
H/L = 3/6= 1.5
C_2= 1.0 for H/L ≥ 1.0
V_{bm}= 0.70 N/mm^2

APPENDIX B (continued)

$V_m = (C_1 + C_2)V_{bm} = 0.7744 N/mm^2$

$^{V}{}_{p} < 0.1f'_m A_g = 0.1(20)(194)=$

$V_n = v_m + v_p + v_s$
$V_s = 0.9665 N/mm^2 – 0.7744 N/mm^2 = 0.1921\ N/mm^2$

$C_3 = 0.8$ for masonry walls
$V_s = C_3 A_v f_y / b_w s$
$A_v = V_s(b_w s)/C_3 F_y = 31.05\ mm^2$

Use #10 @ 200mm

VITA

Mr. Laith Aaron Grigaliunas graduated from the University of Illinois at Chicago with a Bachelor of Science in Civil, Materials and Environmental Engineering in 2018. Mr. Grigaliunas is currently a graduate student at the University of Illinois at Chicago in the Civil, Materials and Environmental Engineering Department. He worked on a masonry code comparison between different countries as his Master Thesis project and will earn a Master's of Science from the University of Illinois at Chicago in 2021. In the future, he will pursue a career in structural engineering.