OPTIMIZATION OF RANDOM RUBBLE MASONRY RETAINING WALL DESIGN

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Dissertation submitted in partial fulfillment of the requirements for the degree
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DECLARATION

I declare that this is my own work and this dissertation does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other University or institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

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Dr. Mrs. D, Nanayakkara

ABSTRACT

The conventional earth retaining structures built using Random Rubble Masonry (RRM) are designed as gravity retaining structures where weight of the structure is used for its stability. In Sri Lanka, RRM retaining walls is the most common type of retaining structure for low retaining heights .However; in general, engineers are reluctant to adopt RRM for retaining heights more than 3m high, due to comparatively large sections obtained as the result of conventional design practice. More optimal and creative solutions could be obtained even for low retaining heights, if design material properties of RRM are known.

In this study, use of flexural strength of RRM and adopting a Reinforced Concrete (RC) Tie-back at the top of the retaining wall to optimize the conventional design was explored. The experimental investigation was carried out to find out the flexural, compressive and shear strength of RRM. Further, bond strength between Reinforced Concrete (RC) and RRM was investigated. These tests results have been used to ascertain the adoptability of suggested optimizations.

From the experimental study, it was concluded that magnitude of material strengths of RRM are sufficient for considerable optimization by taking into account the effect of flexural strength of RRM and adopting a Tie- back. The width of the base of wall section reduction for 3m high retaining wall was 28% as the result of the optimization.

Keywords: Random Rubble Masonry, Retaining walls, Optimization, Tie- back, Flexural Strength.

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LIST OF ABBREVIATIONS

Abbreviation	Description
RRM	Random Rubble Masonry
RC	Reinforced Concrete
BS	British Standard
ICTAD	Institution of Construction Training & Development
HM	Hydraulic Mortar
AM	Air Lime Mortar
ASTM	American Society for Testing and Materials

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Chapter 01

INTRODUCTION

1.1 General

Masonry built with rubble stones of random sizes and shapes are referred as Random Rubble Masonry (RRM). The stones used in RRM are only hammer dressed and they are bonded together with comparatively thick mortar joints.

RRM is one of the oldest forms of masonry construction used all over the world. It was widely used due to its durability and availability of materials. However, nowadays, higher cost of stones, depletion of sources of materials due to excessive extraction to produce aggregates for construction works, difficulties associated with transporting and handling, identification of cheaper construction materials have limited its usage. Nevertheless, RRM has been commonly used in Sri Lankan construction industry, mainly for walls exposed to moisture environments and for walls where aesthetic considerations are governed.

Earth retaining walls built using RRM have also been wildly used in Sri Lanka over the centuries, especially in housing construction and in infrastructure development projects. Still, the RRM retaining wall is the most common retaining wall type in Sri Lanka for low retaining heights. Figure 1.1 shows a retaining wall built along a property line.

Figure 1.1: RRM Retaining Wall in front of Nuwara Eliya Post Office

1.2 Need for Research

In general, RRM retaining wall is designed as a Gravity retaining structure where the weight of the structure is used for its stability. Engineers do the design by proportioning the sections using “Middle-third rule” or “Flexural Formula”.

Usually, the use of RRM for retaining heights over 3m is not economical. Even if stones are readily available and cheap in the locality, engineers are reluctant to use RRM for larger retaining heights due to large sections gained as a result of the traditional design practice. More optimal and creative solutions could be obtained even for low retaining heights, if design material properties of RRM are known.

However design guild lines or strength parameters given in Codes of Practice, National Standards or Specifications relevant to RRM are not sufficient for engineers to deal with this material with confidence. Even the Sri Lankan engineers have not taken much of interest in doing research in this area compared to research carried out in respect of other construction materials. Hence study on RRM is worthwhile to enhance the effective use of RRM.

1.3 Optimization of Gravity Retaining Wall

In this study, the following effects were taken into consideration to optimize traditional gravity retaining wall design.

1.3.1 Effect of Tie - back

The Tie- back reduces the Bending moment developed in the retaining wall due to lateral loads. This can be achieved by introducing Reinforced Concrete (RC) beams which are laterally restrained, cast on top of retaining walls. Application of this arrangement for a house constructed on a sloping ground is illustrated in Figure 1.2. In this case, Retaining walls along grid 1 and 2 are supported by the Tie-back arrangement of RC beams.

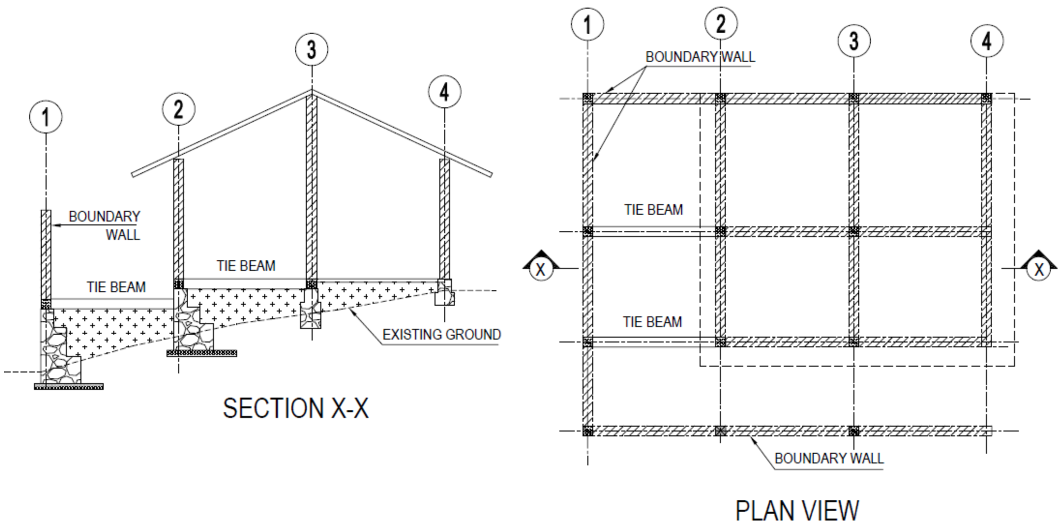


Figure 1.2: Application of Tie-back for RRM retaining walls

1.3.2 Effect of Flexural Strength

In general, gravity retaining walls are proportioned so that no tension is developed at the Soil- Base interface, and hence no flexural stresses develop within the material as well. However, masonry possesses low flexural strength and hence, use of this flexural strength for optimizing traditional gravity wall was investigated.

1.4 Objectives of the Research Study

The objective of this research was to investigate the following material properties of RRM experimentally in order to optimize the conventional RRM gravity Retaining wall design:

a) Flexural Strength;
b) Shear Strength;
c) Compressive Strength and
d) Bond between RC- RRM interface.

1.5 Methodology

The main objective of this study is to optimize the traditional gravity retaining wall design by considering the effects of the tie-back and the flexural strength of RRM. To achieve the above goal, following Methodology was adopted:

- A literature review on previous research wok carried out in the area of study;
- Review the literature evolved for design guidance for RRM;
- Gather information related to RRM retaining walls, including the most commonly used mortar mixes for RRM, Retaining wall heights & wall thicknesses and currently adopted design methods etc.;
- Study test methods to investigate flexural strength, shear strength, compressive Strength and the bond strength at the concrete- masonry interface;
- Carry out experimental investigation to evaluate flexural strength, shear strength, compressive strength and bond between Concrete – RRM interface;
- Comparison of the results with previous research findings of similar nature; and
- Optimizing the Retaining wall design, considering the effect of strength parameters found from the experimental study.

1.6 Outline of the Dissertation

The second chapter of this dissertation deals with the literature review, which includes the study on retaining wall design approaches, previous attempts of optimization of RRM retaining walls, Standard guidance/construction practices of RRM and the previous research findings relevant to RRM.

The third chapter summarizes the different approaches used for design of RRM retaining wall, which were carried out to assess the effect of tie- back and the flexural strength of RRM to conventional design approach.

The fourth chapter provides all details of experimental investigation and the analysis of results is presented in Chapter 5. Further, it includes a comparison of obtained results with the previous research findings of similar nature.

The dissertation concludes with Chapter 6, indicating the conclusions of the study and giving suggestions for further research work.

Chapter 02

LITERATURE REVIEW

2.1 Introduction

The literature review was carried out to gather information from previous research studies in this area of study and to acquire the knowledge on widely adopted design approaches of RRM.

This chapter summarizes the important and most relevant information gathered from the literature for this research study.

2.2 Design of Retaining Walls

There are two widely accepted design approaches available for design of gravity retaining walls as follows:

i) Limit state design method; and

ii) Conventional design method.

2.2.1 Limit State Design Method

The principle of Limit state design was introduced with the BS 8002- Code of Practice for Earth retaining structures, 1994[6]. The latest revision of the British Standard for it, which is Eurocode 7 [7] is also based on same design principle.

According to the principles of limit state design, an earth-retaining structure must comply with both ultimate and serviceability limit states. Ultimate limit states are those involving the collapse or instability of the structure as a whole or the failure of one of its components. Serviceability limit states are those involving excessive deformation, leading to damage or loss of its function [8].

In this approach, earth retaining structure is checked as to whether both these limit states are satisfied, after application of partial factors to actions and soil properties. Characteristic values of soil strength parameters are divided by an appropriate partial safety factor to obtain design values. The design values of actions, on the other hand, are obtained by multiplying characteristic values by an appropriate partial safety factor.

2.2.2 Conventional Design Method

As its name implies conventional method is a much older approach which is based on code CP2- Earth retaining structures, 1951. This method deals with "Factor of safety" in terms of moments, sliding force and bearing capacity. The Factor of safety considered to take into account all the uncertainties in the load evaluation and material properties.

With the introduction of BS 8002,1994 [6] CP2 had to be officially withdrawn; however, still its design approach is very popular among design engineers. It is a rather simple approach which has been successfully adopted for design of earth retaining structures [9].

Several popular geotechnical and soil mechanics text books published before the year 2000 and even after, have illustrated their design examples based on conventional method. This is one of the main reasons for its popularity. The Craig's Soil Mechanics book which has been published seven times, in 1974, 1978, 1983,1987,1992,1997 and 2004, contains design examples of retaining wall design based only on conventional approach up to 1997edition [10]. In its 2004 version [8], design examples are explained based on both approaches.

2.2.2.1. Stability Analysis in Conventional Method.

In conventional method, stability analysis of a retaining walls deals with following failure modes:

i) Sliding;

ii) Overturning; and

iii) Bearing Capacity.

Since code CP2 was published more than 60 years ago, it is now a rare document in design offices. Due to this reason, the procedures given in text books for the stability analysis, are widely used. The procedures of the stability analysis given in the following two text books were considered in this study.

a) Craig,R.F., Craig's Soil Mechanics, 6th Edition[10]

b) Cheng Liu, Evett J.B, Soil and Foundations, 7th Edition[11]

2.2.2.1.1 Stability Analysis, Craig, R.F [10]

Checking for sliding, overturning and bearing capacity are done using two basic equations. Figure 2.1 shows Loads and reactions acting on a retaining wall.

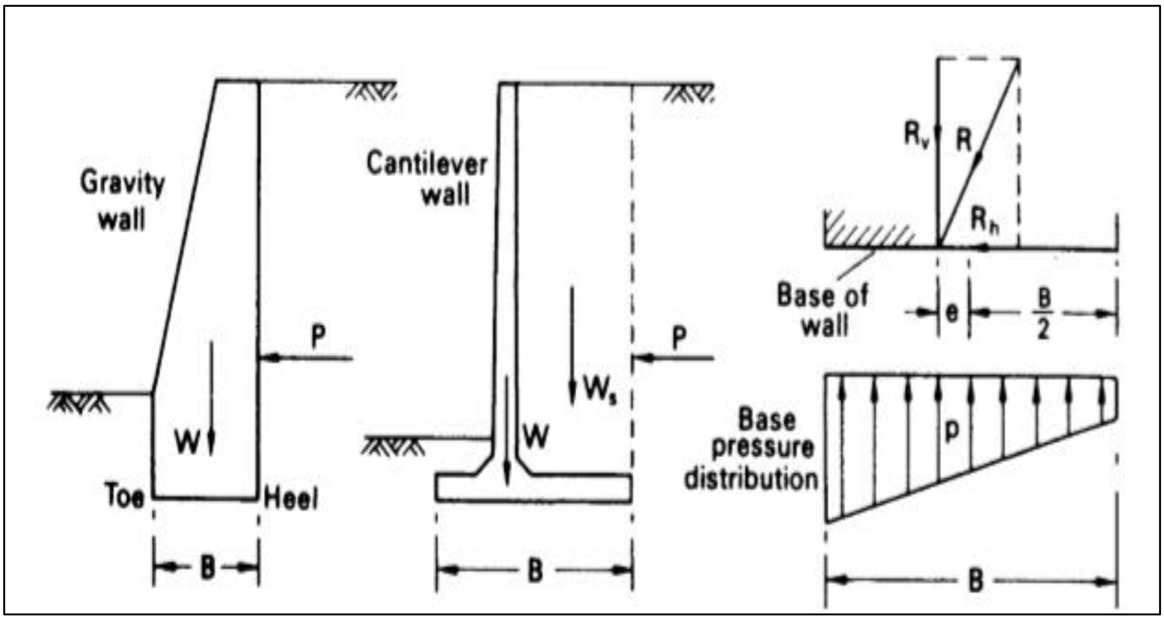


Figure 2.1 : Loads and base reactions of retaining walls

Source : Craig,R.F., [10]

Factor of safety (F_s) against sliding, ignoring the passive resistance, is given by,

$$F_s = \frac{R_v \, tan\delta}{R_h} \qquad (2.1)$$

Where,

R_v, R_h : Vertical and horizontal components of the resultant force R

δ : angle of friction between the base and the underlying soil

Usually, for factor of safety, a value of at least 1.5 is specified.

Both overturning and bearing capacity checks are done with single equation (2.2) which can be derived from the flexural formula. The Flexural formula can be applied to the base of the retaining wall to find out the maximum and minimum base pressure as follows,

$$p = \frac{R_v}{A} \pm \frac{My}{I}$$

Where,

R_v : Vertical components of the resultant force R

p : Minimum or maximum base pressure

A : Area of the base. (Since 1m length of wall is under consideration, this equals to base width of B)

M : Moment about center line of the base

I : Second moment area about the base center line

y : the distance to the edge of the base from base center line.

The derivation can be made as follows,

$$p = \frac{R_v}{B} \pm \frac{R_v \times e \times B/2}{(1 \times \frac{B^3}{12})}$$

$$= \frac{R_v}{B} \pm \frac{R_v \times e \times 6}{B^2}$$

$$p = \frac{R_v}{B}\left(1 \pm \frac{6e}{B}\right) \qquad (2.2)$$

Where,

p : Minimum or maximum base pressure

B : Base width

e : Eccentricity of base resultant

If the maximum base pressure is less than the allowable bearing capacity of the underlying soil, check on the bearing capacity is considered to be satisfactory.

If the minimum base pressure is greater than zero, overturning criterion is considered satisfactory. When base pressure is positive throughout the base, the whole base width is in contact with the underlying soil. Therefore, overturning is considered satisfactory without evaluating the factor of safety against overturning.

2.2.2.1.2 Stability Analysis by Liu,C and Evett, J.B[11]

The three factors of safety with regard to stability analysis are given as follows:

$$\text{F.S}_{\text{(sliding)}} = \frac{\text{Sliding Resistance Force}}{\text{Sliding Force}} \qquad (2.3)$$

$$\text{F.S}_{\text{(overturning)}} = \frac{\text{Total righting moment about toe}}{\text{Total overturning moment about toe}} \qquad (2.4)$$

$$\text{F.S}_{\text{(bearing capacity)}} = \frac{\text{Ultimate bearing capacity of soil}}{\text{Maximum base presussure}} \qquad (2.5)$$

The minimum factors of safety for sufficient stability specified in this book are as follows,

F.S $_{(Sliding)}$ = 1.5 (If the passive earth pressure of the soil at the toe in front of wall is neglected);

F.S $_{(Overturning)}$ = 1.5 (for granular backfill soil); or

= 2.0 (for cohesive backfill soil); and

F.S $_{(Bearing\ capacity)}$ = 3.0.

The ratio between ultimate bearing capacity and allowable bearing capacity varies from 2.5 to 3. Hence, when equation 2.5 is re-written for allowable bearing capacity instead of ultimate bearing capacity, the factor of safety against bearing capacity of 1 could be considered as satisfactory.

The maximum base pressure is found from the flexural formula. Before applying this formula, it was checked to make sure that vertical reaction of the base is within the one- third of the base width. This is to ensure that base pressure is positive throughout the base width.

The one- third concept can be explained using the equation 2.2

In order to have a positive contact throughout,

$$p_{min} = \frac{R_v}{B}\left(1 - \frac{6e}{B}\right) > 0$$

Hence,

$$\left(1 - \frac{6e}{B}\right) > 0$$

$$e > \frac{B}{6}$$

2.3 Adopting Tie- back Effect in Gravity Retaining Wall Design

Some applications where Tie- back effect has been utilized effectively are discussed in this section. The first application has been used for retaining wall design of a printing complex at Mawaramandiya [12]. The second application is an own experience of the author in which the Tie- back effect is adopted effectively.

In both cases, proper lateral load transferring from wall to Tie back has been assumed by design engineers through their judgment. If there were proper design information, even more optimized designs would have been adopted.

2.3.1 Innovative Earth Retaining System Adopted at the Proposed Printing Complex at Mawaramandiya [12]

The proposed land for this project was located in a sloping terrain, thus earth retaining both along the site boundary and at some intermediate locations had been adopted.

The heights to be retained have varied from 1m to 7m. Concrete anchor blocks tied back by RC tie beams have been used to enhance the lateral stability of the gravity retaining walls. Figure 2.2 illustrates the details of modified RRM retaining wall system adopted and Figure 2.3 shows the Construction of Tie back arrangement at the site.

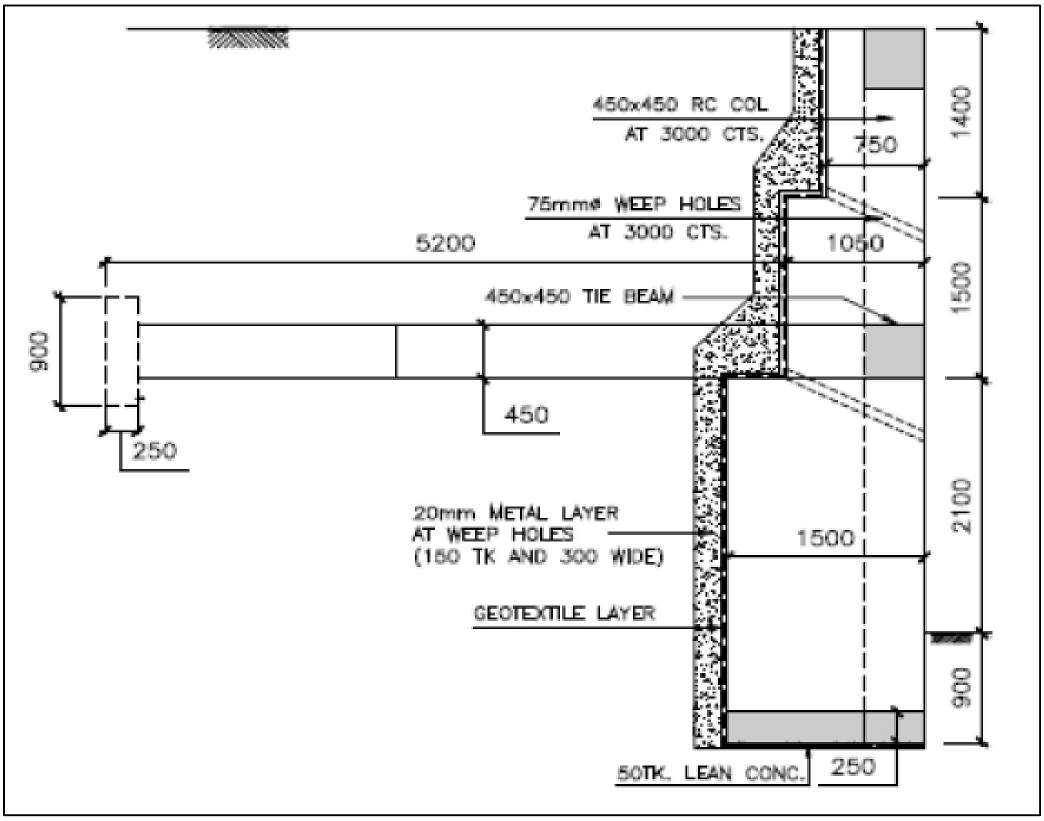


Figure 2.2 : Modified RRM retaining wall system adopted.

Source : Fernando S et al. [12]

Figure 2.3 : Construction of Tie back arrangement at the site

Source : Fernando S et al. [12]

According to the cost comparison analysis, it was reported that about 15% of overall cost could be saved by adopting this system compared to conventional gravity retaining wall.

2.3.2 Wall Foundations of Proposed Block No.10- Mahinda Rajapaksha Vidyalaya, Homagama

The proposed building was a four storied class room block built on a land with a steep slope. The height of the wall foundations at the rear side of the building had to be increased up to 3m and were designed as retaining walls to retain the soil to support ground slab as well. Figure 2.4 illustrates the wall foundations at the rear side of the class room block.

Figure 2.4 : Wall Foundations at the Rear Side of the Class Room block

Set of RC tie beams were cast on the retaining walls and wall foundations in order to reduce slenderness of columns. These tie beams have been advantageous in terms of improving the lateral stability of the retaining walls as well.

2.4 British Standards relevant to Random Rubble Masonry Design

During the past fifty years, British Standard has been consistently limiting its design recommendation on natural stone and stone masonry. Prior to 4-5 decades, some of codes of practice entirely dealt with natural stone masonry. However when these were replaced with modern British and European standard, very limited guidance for natural stone masonry design was provided. Table 2.1 shows the different Code of Practices existed and currently operating in the field of stone masonry.

Table 2.1: Summary of guidance on British and British European Standards relevant to Natural Stone

Standard	Title	Status
CP 121.201: 1951	Code of practice for masonry walls ashlared with natural or cast stone	Withdrawn
CP 121.202 :1951	Code of Practice for Masonry Rubble Walls	Withdrawn
BS 5390:1976	Code of practice for Stone masonry	Withdrawn
BS 5628-1:2005	Code of practice for the use of masonry. Part 1: Structural use of unreinforced masonry	Withdrawn
BS 5628-3:2005	Code of practice for the use of masonry. Part 3: Materials and components, design and workmanship	Withdrawn
BS EN 1996-2:2006	Eurocode 6. Design of masonry structures. Part 2: Design considerations, selection of materials and execution of masonry	Current
BS EN 771-6:2011	Specification for masonry units; Natural stone masonry units	Current

Source : 1) Urquhart[13]

2) http://shop.bsigroup.com

2.5 Standard Construction Practices of Random Rubble Masonry in Sri Lanka

Currently, Random Rubble Masonry is the most widely used stone masonry construction method in Sri Lanka. The Cabook stone masonry and Coursed Rubble masonry (rubble masonry with approximately rectangular bond patterns) have some applications, but extremely small compared to RRM.

However, limited references are available for construction specifications of Random Rubble Masonry specifically in Sri Lankan context. Though not comprehensively addressed, ICTAD specification for Building Works [14] is considered to be the main reference available for rubble masonry construction in Sri Lanka. Few research papers [5, 15] on Random Rubble Masonry also have contributed towards covering the deficiency. Few design guidelines specified by above references regarding selection, preparing and laying of stones are summarized below.

2.5.1 Type of Stones

For Random Rubble Masonry, Granite, Charnockites and Gneiss are mainly used. They shall be hard, sound, free from decay, weathering and defects like cavities, cracks, flaws, sand holes, veins patched of soft or loose materials. Stones shall be angular as far as possible and stones with round surfaces shall not be used to avoid single point contact. ICTAD specifications[14] does not specify material properties of stones, but minimum compressive strength of 10 N/mm^2 and water absorption less than 10% [5] are generally preferred.

2.5.2 Sizes of Stones

Sizes of stones are specified basically considering practical aspects and the stability of the masonry construction. Length of stones is limited in longitudinal direction of the wall to allow for differential settlements (Refer Figure 2.5). Following criteria have been established by ICTAD specifications [14] in terms of selecting suitable sizes of stones:

i. Stone shall be small enough to lift manually; so limiting weight can be considered as 25kg;
ii. Length of stone < 3 x Height of stone ;
iii. Breadth of stone < 0.75x wall thickness (except for Through stones);
iv. Breadth of stone > 150mm; and
v. Height of stone < 300mm.

In case of Hearting stones,

i. Stone shall not pass through a circular ring of 150mm diameter; and
ii. Length or Breadth of stone > 100mm.

Figure 2.5 illustrates typical bond patterns and Figure 2.6 shows the types of stones used in RRM.

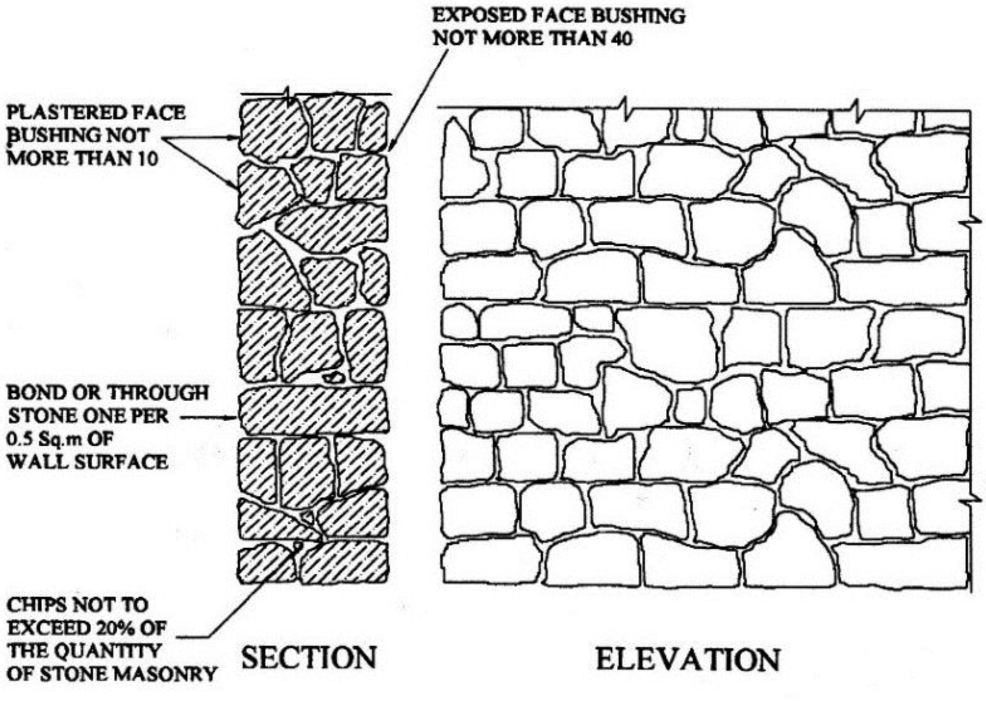


Figure 2.5: Typical Bond Patterns and Specifications for Bushing, Amount of Chips and Through Stone.

Source: ICTAD specification for Building works [14]

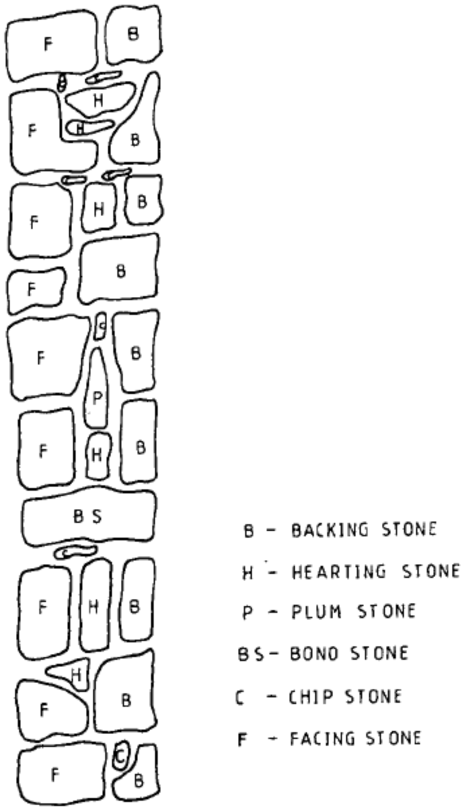


Figure 2.6: Types of Stones used in RRM

Source: Chandrakeerthy [15]

2.5.3 Dressing of Stones

Stones shall be hammer dressed to enable them to come into close proximity with the neighboring stone. The Bushing (irregularities of the face of the wall) in the face shall not project more than 40mm on an exposed face and 10mm on a face which is to be plastered.

2.5.4 Mortar

ICTAD specifications [14] doesn't carry special recommendation for Random Rubble Masonry and it specifies general mortar designations (1:5, 1:6 and 1:8 cement: sand proportions) for RRM as well. However general practice is to adopt 1:5 mortar designation as most of local applications of RRM are in contact with water.

2.5.5 Mortar joints

Different joint finishes can be created depending upon the desired aesthetic requirement. Most commonly used joints in RRM are illustrated in Figure 2.7.

Mortar Joints in RRM shall be within 6mm to 20mm. When joints are more than 20mm, they should be well packed with chip stones to limit the joint thickness within the limits.

When plastering or pointing is not required to be done, the joints shall be struck flush and finished at the time of laying. Otherwise, joints shall be racked to a minimum depth of 20mm by a racking tool during the progress of work, when the mortar is still green.

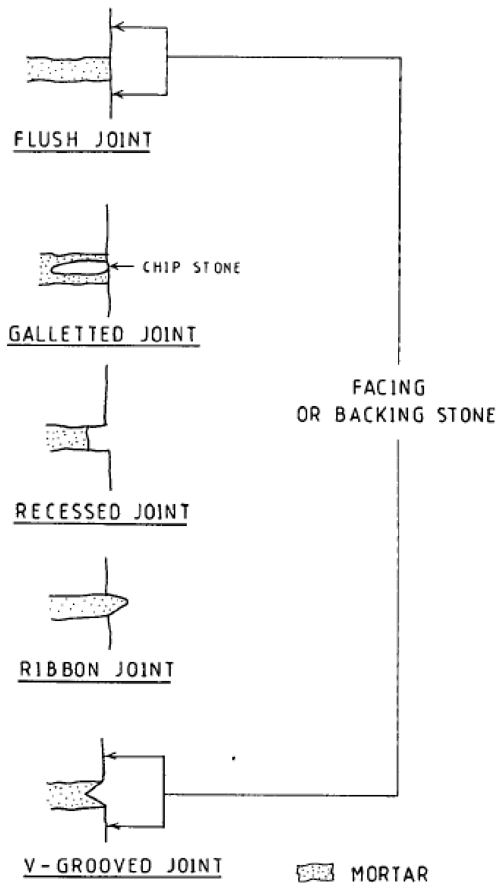


Figure 2.7: Types of Joints used in RRM

Source: Chandrakeerthy [15]

2.5.6 Laying

All stones shall be clean, free of dust and shall be wetted prior to use. Every stone shall be carefully fitted to the adjacent stones, so as to form neat and close joints. A sufficient number of Through stones (see Figure 2.5) shall be adapted to bond adjacent stones together. At least one through stone shall be built into the wall at the intervals of 1.8m horizontally and 0.6m vertically. Such stones shall be at least 150mm square at the face and run through the thickness of the walls up to 600mm. In case of walls exceeding 600mm in thickness, more than one stone may be used to run through the full thickness of the wall with overlaps of not less than 150mm. Where through stones of suitable lengths are not available, concrete block of grade 15 shall be used.

2.5.7 Curing

Masonry work shall be kept constantly moist on all faces for a minimum period of 7 days.

2.6 Previous Experimental Investigations on Material properties of Random Rubble Masonry

2.6.1 Compressive Strength

Chandrakeerthy[5,15] has investigated the compressive strength of RRM for 1:5 and 1:8 mortar designations. For both cases 300mm thick 600mmx600mm wall panels built with Chanockites stones, have been tested.

All test panels have been built according to general guidelines (sizing, dressing, laying, mortar thicknesses, curing etc.) specified in ICTAD specifications [14]. Testing of panels (age of testing, load application etc.) have been carried out in accordance with BS 5628-1 [16]. Test results reported in these research publications are tabulated in Table 2.2 and Table 2.3.

Table 2.2: Characteristic Compressive Strength of RRM for Mortar Designation of 1:5

Mortar Designation	Mortar Mix (cement:sand)	Compressive Strength of Stones (N/mm^2)						
(iii)	1:5	20	30	40	50	60	80	100
		1.07	1.60	1.84	2.08	2.31	2.31	2.31

Source: Chandrakeerty [5]

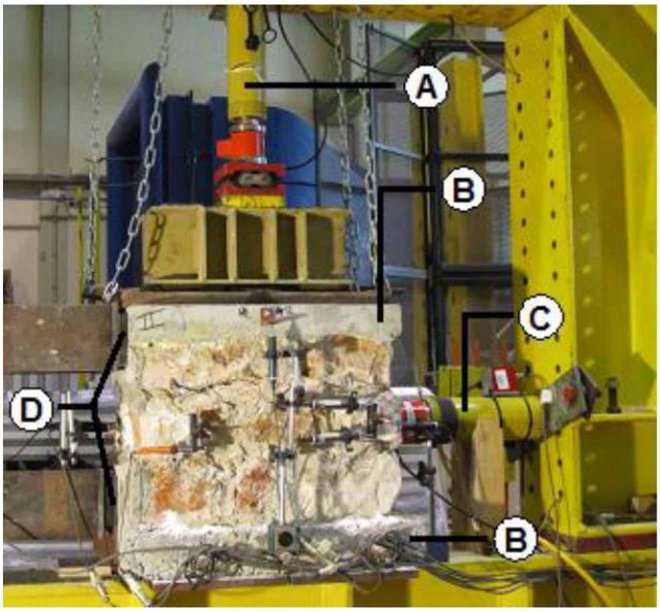
Table 2.3: Comparison of Characteristic Compressive Strength of RRM and Brickwork for Mortar Designations of 1:5 and 1:8

Mortar Designation (Cement: Sand)	Characteristic Compressive Strength (N/mm^2)			
	Brick Work		Random Rubble Masonry Flush Jointed	
	Brick Compressive Strength : $2.0N/mm^2$	Brick Compressive Strength: $2.8N/mm^2$	Compressive Strength of Stone: $40N/mm^2$	Compressive Strength of Stone: $60N/mm^2$ and above
iii (1:5)	1.00	1.40	1.84	2.31
iv (1:8)	0.88	1.23	0.68	0.86

Source: Chandrakeerty [15]

2.6.2 Shear Strength

An experimental study was carried out by Milosevic et al. [17] to investigate the shear strength of rubble masonry. In this study, rubble triplets having 400mm X 600mm X 400mm dimensions were built with three stone layers. These layers were intentionally formed to allow for the middle stone layer to slide between the top and bottom layers, when it was subjected to a shear load as shown in Figure 2.8, while the top and bottom layers were latterly restrained.



A- Pre compression Load
B- Concrete Slab
C- Shear Load
D- Lateral restrains to top & bottom layers

Figure 2.8: Triplet setup by Milosevic et al. [17]

In this test setup, failure surfaces were imposed to be parallel with the stone layers. Hence, this test doesn’t represent the actual shear failure of Random Rubble Masonry of which well-defined failure surfaces cannot be expected.

Figure 2.9 shows the relationship between normal stress and shear stress of samples. Following abbreviations have been used in this figure:

HM : Hydraulic Mortar; and
AM : Air lime Mortar.

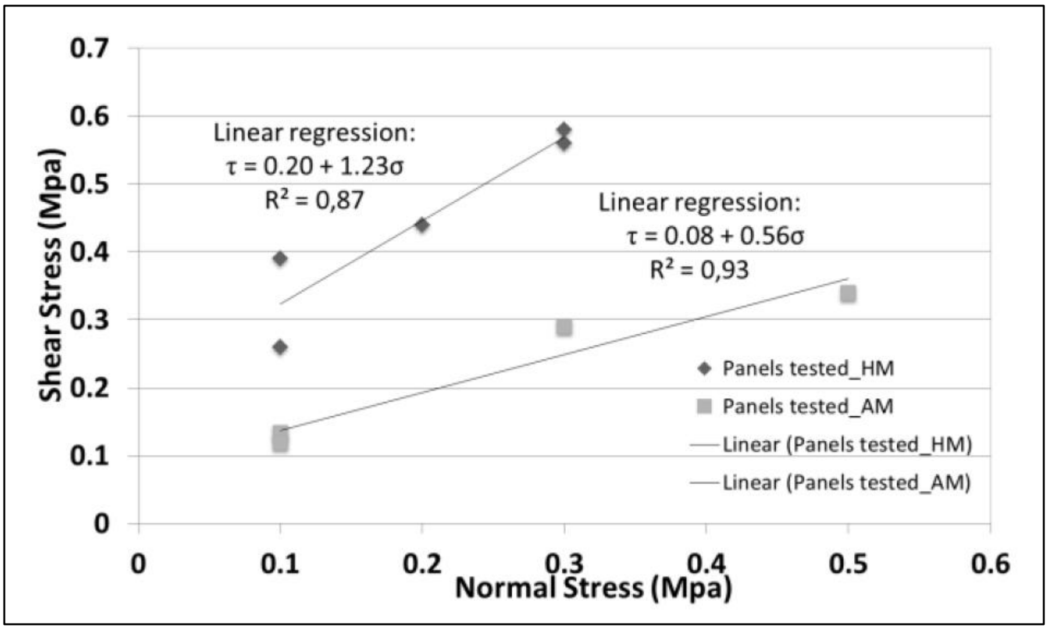


Figure 2.9 : Relationship between Normal stress and Shear stress of samples

Source : Milosevic et al. [17]

According to their analysis, Initial shear strength (shear strength when normal stress equal to zero) of masonry samples built with hydraulic mortar (HM) can be considered as 0.2 Mpa. However, it was not reported any information on mortar designation, strength of stones or type of stones which have been used for the investigation.

2.7 Experimental Investigation on Masonry- Concrete Interface

Shear behavior at the Masonry – Concrete interface relevant to Sri Lankan brick masonry has been investigated by Premadasa et al. [4]. In this investigation, tests were carried out using a series of Brick- Concrete block couplets bonded with mortar. The brick-concrete couplets were tested in accordance with ASTM C 952-02 [18].

For preparation of testing specimens, a wire cut brick of standard size (215mm x 105mm x 65mm) and a concrete block of the same size cast with Grade 25 concrete have been used. Testing has been carried out for 1:5, 1:6, 1:8 mortar designations and 10mm, 15mm mortar thickness. Test specimens have been cured for 28 days prior to testing.

During testing, concrete portion of the specimen has been fixed and shearing load was applied to the Brick through a hydraulic jack. This is illustrated in Figure 2.10.

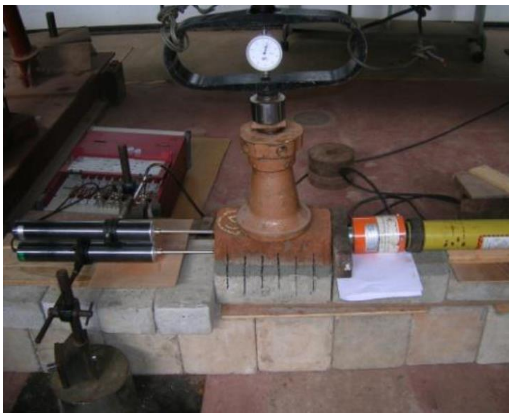


Figure 2.10 (a)

Figure 2.10 (b)

Figure 2.10 : (a) - Test Setup ; (b) – Shear deformation while applying the load

Source : Premadasa et al. [4]

According to the results, average shear strength at the interface for 10mm mortar joint with 1:5 mortar designations was 0.2 N/mm^2. For other mortar designations and for increased mortar thickness lesser strength values have been observed.

Chapter 03

DIFFERENT APPROACHES USED FOR DESIGN OF RANDOM RUBBLE MASONRY

3.1 Introduction

In order to study the effects of Tie back and flexural strength of masonry on the design of RRM gravity retaining wall, a 3m high RRM retaining wall was designed considering following 3 cases:

1. Design of Conventional Retaining wall using Conventional Method;
2. Design of Retaining wall assuming RRM will not fail due to flexure; and
3. Design of Retaining wall with the Tie back effect.

Following Design Information were assumed;

- Density of the backfill : 17 kN/m^3
- Density of the RRM : 22 kN/m^3
- Surcharge pressure : 5 kN/m^2
- Characteristic values of the shear strength parameters for the backfill,
- C = 0 , $\emptyset = 30°$
- Angle of friction between the base and the foundation soil , $\delta = 30°$
- Allowable bearing capacity of underneath soil is 250 kN/m^2 and water table is below the base of Retaining wall.

3.2 Case 1 - Design of RRM Retaining Wall Using Conventional Method

The loads act on the retaining wall for Case 1 is shown in the Figure 3.1.

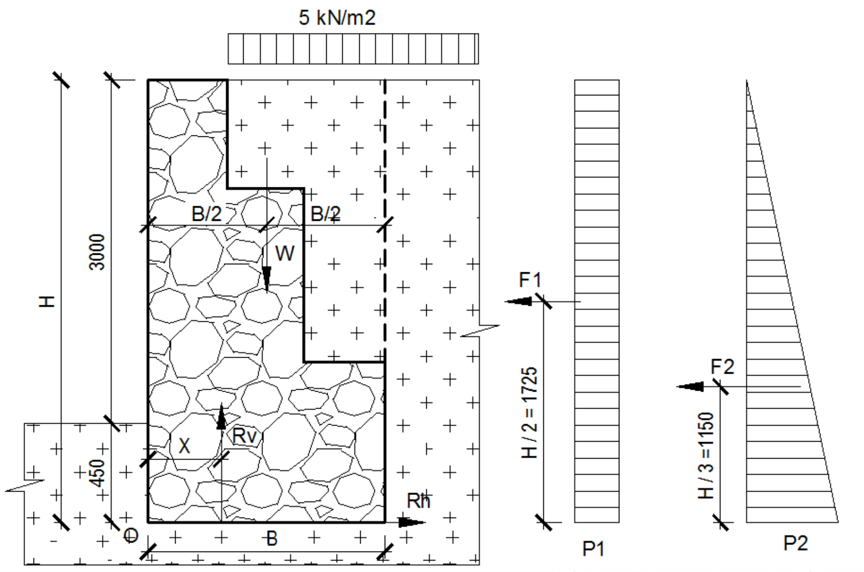


Figure 3.1: Loads acting on Retaining wall for Case 1

In this case, a value of 2.1m was assumed for base width.

The weight of soil above the retaining wall contributes to the stability of the wall. In order to simplify the analysis, the retaining wall can be treated as a rectangular one with a base width of B. The average density of the wall (γ_{wall}) is considered as 20kN/m^3

Applying Rankin theory for evaluating lateral soil pressure, the active pressure coefficient,

$$K_a = \frac{1 - sin\emptyset}{1 + sin\emptyset}$$

$$K_a = \frac{1 - sin30°}{1 + sin30°}$$

$$K_a = 0.33$$

Lateral soil pressure at the bottom,

$P2 = K_a \times H \times \gamma_{soil}$

$= 0.33 \times 3.45 \times 17$

$= 19.3\ kN/m^2$

Lateral pressure due to surcharge,

$P1 = K_a \times 5$

$= 0.33 \times 5$

$= 1.7\ kN/m^2$

The resultant forces due to lateral pressures can be calculated as,

$F1 = P1 \times 3.45$

$= 5.9\ kN/m$

$F2 = P2 \times 3.45/2$

$= 33.3\ kN/m$

The weight of simplified retaining wall,

$W = B \times H \times \gamma_{wall}$

$= 2.1 \times 3.45 \times 20$

$= 144.9\ kN/m$

Considering vertical and horizontal equilibrium,

$$W = R_v = 144.9\ kN/m \tag{3.1}$$

$R_h = F1 + F2$

$= 33.3 + 5.9$

$= 39.2\ kN/m$

By taking moments about the toe of the wall (O),

$$R_v \times x = W \times \frac{B}{2} - F1 \times 1.725 - F2x1.15$$

Substituting the results of eqation 3.1 and values of F,

$$144.9 \times x = 144.9 \times \frac{2.1}{2} - 5.9 \times 1.725 - 33.3x1.15$$

$x = 0.71m$

Eccentricity of R_v can be wrtten as,

$$e = \frac{B}{2} - x$$

$$= \frac{2.1}{2} - 0.71$$

= 0.34 m

Stability analysis was done based on Craig, R.F [10].

Check for the minimum base presure

$$p_{min} = \frac{R_v}{B}\left(1 - \frac{6e}{B}\right)$$

$$= \frac{144.9}{2.1}\left(1 - \frac{6 \times 0.34}{2.1}\right)$$

$= 2.0\ kN/m^2 > 0,$ hence check for minimum pressure is satisfactory.

i) Check for the Maximum Base Presure

$$p_{max} = \frac{R_v}{B}\left(1 + \frac{6e}{B}\right)$$

$$= \frac{144.9}{2.1}\left(1 + \frac{6 \times 0.34}{2.1}\right)$$

$= 136.0\ kN/m^2 < 250kN/m^2$, hence check for maximum pressure is satisfactory.

ii) Check for Sliding

Factor of safety against sliding,

$$F_s = \frac{R_v\ tan\delta}{R_h}$$

$$= \frac{144.9\ tan30°}{39.2}$$

$$= \frac{83.6}{39.2} = 2.1 > 1.5$$

Hence check for sliding is satisfactory.

These three checks covered the stability analysis as described in Craig R.F[10]

Check on minimum bearing pressure ensures the overturning criterion of the retaining wall. This is an indirect approach to check the overturning stability and the factor of safety against overturning has to be calculated in order to find out the actual status of the overturning stability.

By using the method specified by Cheng Liu [11], Factor of Safety against Overturning can be computed as follows;

$$F.S\ (Overturning) = \frac{\text{Total righting moment about toe}}{\text{Total overturning moment about toe}}$$

$$= \frac{W \times B/2}{F1 \times 1.725 + F2 \times 1.15}$$

$$= \frac{144.9 \times 1.05}{33.7 \times 1.725 + 5.9 \times 1.15}$$

$$F.S\ (Overturning) = 3.1 > 1.5$$

This shows that the factor of safety is well beyond the minimum requirement and base width could have been further reduced while keeping the required minimum Factor of Safety of 1.5.

However, this will lead to a negative bearing pressure, which means tension between base and soil.

In reality, soil cannot furnish any tensile resistance. Hence equations derived based on flexural formula are no longer valid to compute the soil pressure at this stage.

Further, when bearing pressure becomes negative, flexural stresses develop within the retaining wall material itself. It can be assumed that both take place at the same time since same equation is used to compute the bearing pressure based on the flexural formula which is applicable to stress distribution of retaining wall material at the base level as well. In case of masonry retaining walls, development of tension or flexure is generally avoided.

These are the reasons for keeping the minimum bearing pressure greater than zero even with a higher factor of safety against overturning.

Though it is assumed that masonry does not possess any flexural strength, indeed, it has some flexural strength. In a retaining wall, if it is allowed to develop small flexural stress which is less than the flexural strength, while keeping the required factor of Safety against overturning, retaining wall section can be optimized. In this case, flexural formula cannot be used to compute the soil pressure distribution. This is illustrated in the next design example.

3.3 Case 2 - Retaining Wall Assuming RRM will not Fail Due to Flexure

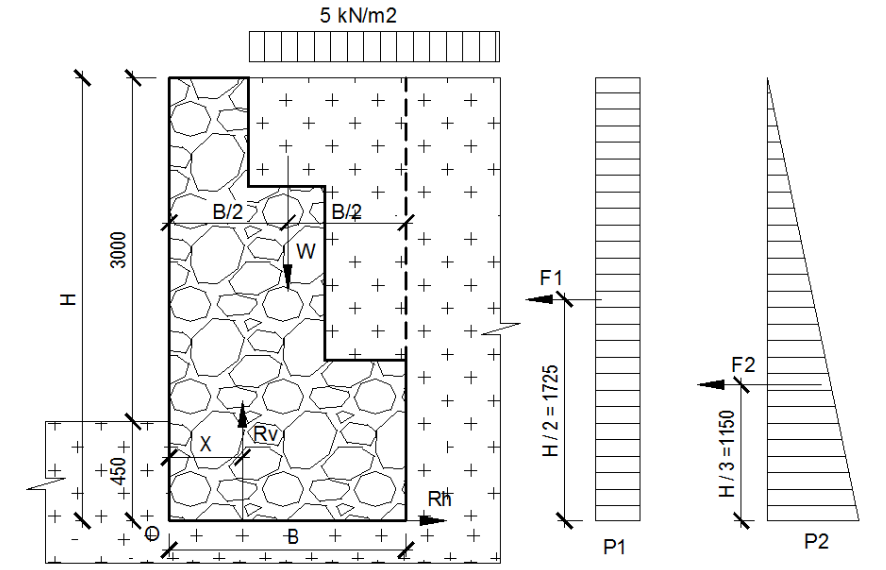


Figure 3.2: Loads acting on Retaining wall for Case 2

In this case try for B=1.5m

Lateral soil pressures and R_h are the same as in the previous case. Hence,

$P1 = 1.7\ kN/m^2$

$P2 = 19.3\ kN/m^2$

$F1 = 5.9\ kN/m$

$F2 = 33.3kN/m$

$R_h = 39.2\ kN/m$

The weight of retaining wall,

$$W = B \times H \times \gamma_{wall}$$

$$= 1.5 \times 3.45 \times 20$$

$$= 103.5 kN/m$$

Considering vertical and horizontal equilibrium,

$$W = R_v = 103.5\ kN/m \qquad (3.2)$$

By taking moment about toe of the wall (O),

$$R_v \times x = W \times \frac{B}{2} - F1 \times 1.725 - F2x1.15$$

Subtituding results of Eqation 3.2 and values of F1 anf F2

$$103.5 \times x = 103.5 \times \frac{1.5}{2} - 5.9 \times 1.725 - 33.3 \times 1.15$$

$$x = 0.28m$$

Eccentricity of R_v can be wrtten as,

$$e = \frac{B}{2} - x$$

$$= \frac{1.5}{2} - 0.28$$

$$= 0.47m$$

$$\frac{B}{6} = 0.25m < e\ ,$$

Henec, base reaction doesn't lie within one-third of the base width.

i) Check for Maximum Base Presure

Base reaction doesn’t lie within the one- third of the base width; hence base pressure will not be positive throughout the base.

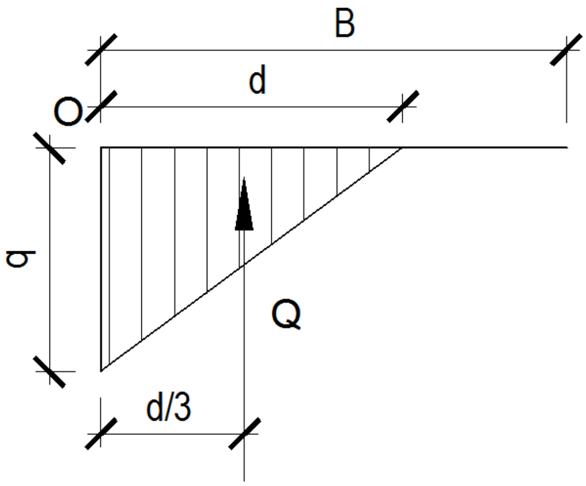


Figure 3.3: Assumed Base Pressure Variation for Case 2

The flexural formula is not applicable in this situation. Instead, soil pressure can be calculated using basic equations of statics.

Since R_v represents the resultant of base pressure,

$$Q = \frac{q \times d}{2} = Rv = 103.5\ kN/m \qquad (3.3)$$

$$x = \frac{d}{3} = 0.28$$

$d = 0.84m$, subsituding d in the equation 3.3,

$$\frac{q \times 0.84}{2} = 103.5$$

$q = 246\ kN/m^2 < 250\ kN/m^2$, hence check on maximum base pressure is satisfactory.

ii) Check for Sliding

Factor of safety against sliding,

$$F_s = \frac{R_v \tan\delta}{R_h}$$

$$= \frac{103.5 \tan 30°}{38.9}$$

$$= \frac{59.7}{39.2} = 1.52 > 1.5$$

Hence check for sliding is satisfactory.

iii) Check for Overturning

$$F.S\,(Overturning) = \frac{\text{Total righting moment about toe}}{\text{Total overturning moment about toe}}$$

$$= \frac{W \times B/2}{F1 \times 1.725 + F2 \times 1.15}$$

$$= \frac{103.5 \times 0.75}{33.3 \times 1.15 + 5.9 \times 1.725}$$

$$F.S\,(Overturning) = 1.60 > 1.5$$

Hence check for overturning is satisfactory.

Since all three checks are satisfactory, the stability of retaining wall can be considered as satisfactory.

However, in this case, flexural stresses develop at the heel of the wall and it was assumed that flexural strength of masonry is capable of resisting it.

The magnitude of flexural stress developed can be estimated by using the flexural formula. The same equation used to compute the minimum base pressure in Chapter 2 (eq. 2.2), can be used to evaluate the magnitude of the flexural stress developed in the base.

If the Flexural Stress at the heel is t,

$$t = \frac{R_v}{B}\left(1 - \frac{6e}{B}\right)$$

$$= \frac{103.5}{1.5}\left(1 - \frac{6 \times 0.47}{1.5}\right)$$

$$= -60.7\ kN/m^2 = -0.061\ N/mm^2$$

If RRM can resist $0.061\ N/mm^2$ flexural stress, retaining wall design is satisfactory with the proposed base width of 1.5m.

3.4 Case 3 – Design of Retaining Wall with the Tie-Back Effect

In this case, the effect of a Tie back on the design of a retaining wall was investigated. The Tie back effect is expected to get through the 225mm x 225mm Reinforced Concrete (RC) beam cast on the retaining wall. This RC tie beam has to be laterally restrained at certain intervals through tie beams cast perpendicular to the retaining walls or through cross walls.

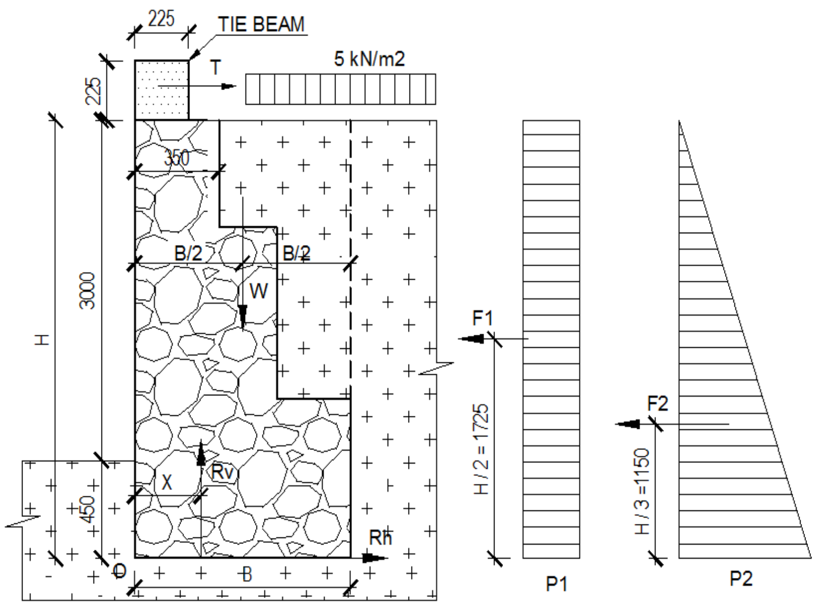


Figure 3.4: Loads acting on Retaining wall for Case 3

Try for B=1.5m

In this case, lateral movement of the retained soil is restrained than previous two cases due to the tie back at the top of the wall. In order to mobilize the active pressure on the wall, the lateral deformation of the soil mass has to reach a certain level. In general, for cantilevered retaining walls, it is assumed that this minimum deformation is reached [8].

If the lateral deformation of soil is zero, the corresponding lateral pressure is called at –rest earth pressure and it is greater than the active earth pressure.

In this case, lateral deformation is not zero since lateral deformation is restrained by a RC tie beam which has no significant lateral stiffness compared to the Retaining walls. Hence, assuming a lateral pressure in between at- rest and the active pressure is more appropriate.

At rest soil pressure coefficient,

$K_0 = 1 - sin\emptyset$

$= 1 - sin30°$

$= 0.5$

Active pressure coefficient,

$K_a = 0.33$

Pressure coefficient K= 0.4 is assumed which is in between Ko and Ka.

Lateral soil pressure at the bottom,

$P2 = K \times H \times \gamma_{soil}$

$= 0.4 \times 3.45 \times 17$

$= 23.5\ kN/m^2$

Lateral pressure due to surcharge,

$P1 = K \times 5$

$= 0.4 \times 5$

$= 2.0\ kN/m^2$

The resultant forces due to lateral pressure can be calculated as,

$F1 = P1 \times 3.45$

$= 6.9\ kN/m$

$F2 = P2 \times 3.45/2$

$= 40.5\ kN/m$

R_v and W are the same as the previous case (neglecting the weight of the tie beam).

Hence,

$W = R_v = 103.5\ kN/m$

Considering horizontal equilibrium,

$Rh = F1 + F2 - T$

$Rh = 47.4 - T$ ———————————— (3.4)

By taking moment about toe of the wall (O),

$Rv \times x = W \times \frac{B}{2} - F1 \times 1.725 - F2x1.15 + T \times H$

Substituting values for W, B, and F,

$103.5 \times x = 103.5 \times \frac{1.5}{2} - 6.9x1.725 - 40.5 \times 1.15 + T \times H$

$103.5 \times x = 19.1 + T \times 3.45$ ———————————— (3.5)

There are three unknowns and only two equations were available. Therefore, for diffent values of T, these two equations were resolved until get acceptable values for other parameters.

As a first trial T= 10kN was assumed.

Then $x = 0.52m$ and $Rh = 37.4\ kN/m$

Eccentricity of Rv can be wrtten as,

$e = \frac{B}{2} - x$

$= \frac{1.5}{2} - 0.52$

$= 0.23m\ (< \frac{B}{6} = 0.25m)$, hence, base pressure will be positive throughout.

Stability checks can be done as follows.

i) Check for Minimum Base Pressure

$$p_{min} = \frac{R_v}{B}\left(1 - \frac{6e}{B}\right)$$

$$= \frac{103.5}{1.5}\left(1 - \frac{6 \times 0.23}{1.5}\right)$$

$= 5.5\ kN/m^2 > 0$, hence check for minimum pressure is satisfactory.

ii) Check for Maximum Base Pressure

$$p_{max} = \frac{R_v}{B}\left(1 + \frac{6e}{B}\right)$$

$$= \frac{103.5}{1.5}\left(1 + \frac{6 \times 0.23}{1.5}\right)$$

$= 132.5\ kN/m^2 < 250 kN/m^2$, hence check for maximum pressure is satisfactory.

iii) Check for Sliding

Factor of safety against sliding,

$$F_s = \frac{R_v\ tan\delta}{R_h}$$

$$= \frac{103.5\ tan 30°}{37.4}$$

$$= \frac{59.7}{37.4} = 1.6 > 1.5$$

iv) Check for Overturning

$$F.S\ (Overturning) = \frac{\text{Total righting moment about toe}}{\text{Total overturning moment about toe}}$$

$$= \frac{W \times \frac{B}{2} + T \times 3.45}{F1 \times 1.725 + F2 \times 1.15}$$

$$= \frac{103.5 \times 0.75 + 10 \times 3.45}{40.5 \times 1.15 + 6.9 \times 1.725}$$

$$F.S\ (Overturning) = 1.92 > 1.5$$

Hence check for overturning is satisfactory.

Hence check for sliding is satisfactory and T = 10 kN.

In this case overall stability can be maintained without allowing flexural stresses to develop within the masonry. However, the capability of transferring tie back force to the RC tie beam through the masonry- Concrete interface and through the narrowest masonry section at the top of the retaining wall, as shown in the Figure 3.5, need to be investigated.

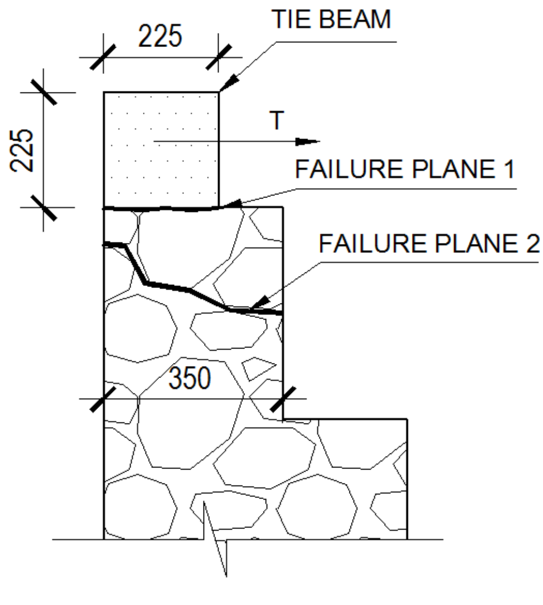


Figure 3.5: Possible Shear Failure Planes of RRM for Case 3

The expected shear stress at the two shear planes are computed as follows,

Shear stress at Concrete- Masonry interface,

$$\frac{10\ kN/m}{225 \times 1000mm^2} = 0.044\ N/mm^2$$

Shear stress at Masonry- Masonry interface (shear stress of RRM masonry),

$$\frac{10\ kN/m}{350 \times 1000mm^2} = 0.028N/mm^2$$

3.5 Summary of the Results Obtained from Three Cases

The results obtained from three cases are summarized in Table 3.1.

Table 3.1: Summary of Results Obtained from Three Cases

Parameter	**Case 1**-Conventional Retaining wall	**Case 2**- Retaining wall assuming RRM will not fail due to flexure	**Case 3**- Retaining wall with a Tie back
Base Width /m	2.1	1.5	1.5
Max. Bearing Pressure / kNm^{-2}	136.0	246	132.5
Min. Bearing Pressure / kNm^{-2}	2.0	0	5.5
Flexural stress at the heel/ Nmm^{-2}	-	0.061	-
Shear stress at Concrete - Masonry interface/ Nmm^{-2}	-	-	0.044
Shear stress at the narrowest masonry section/ Nmm^{-2}	Negligible	Negligible	0.028
Factor of Safety - Overturning	3.1	1.60	1.92
Factor of Safety - sliding	2.1	1.52	1.6
Percentage of base width reduction / %	-	28	28

This study shows that with the effect of flexural strength of RRM and tie back, the conventional retaining wall design can be optimized to a considerable level. According to analysis, approximately, one-third of the base width can be reduced if those effects were taken into consideration.

However, the flexural effect (Case2) can be considered for ground conditions with higher bearing capacities only.

In relation to conventional gravity wall, the tie- back effect further reduces the maximum bearing pressures and the difference between maximum to minimum soil pressures. The combination of both effects would be able to adopt for further optimized solutions.

Chapter 04

EXPERIMENTAL STUDY

4.1 General

This chapter summarizes the details of the experimental investigation carried out in the research study. The main objectives of the experimental study are:

a) To determine the flexural strength of RRM;
b) To determine the shear strength of RRM;
c) To determine the compressive strength of RRM; and
d) To determine the shear strength at the concrete- masonry interface.

To determine above strength parameters of RRM, tests were carried out in accordance with the following codes of practice.

- Flexural Strength : BS EN 1052-2: 1999, Methods of test for masonry - Part 2: Determination of flexural strength [2].
- Shear Strength : BS EN 1052-3: 2002 , Methods of test for masonry- Part 3: Determination of initial shear strength [3]
- Compressive Strength : BS EN 1052-1: 1999, Methods of test for masonry- Part 1: Determination of compressive strength [1].

All these codes of practice deal with masonry constructed with regular size units which are laid in a specified bonding pattern. Standard test methods to determine the above strength properties of RRM are not specified in any of these codes of practice. Therefore, this investigation had to be based on these codes due to unavailability of any other standard specification.

The investigation on shear strength at Concrete- RRM interface was based on the experimental approach adopted by Premadasa et al. [4] to determine the shear behaviour at the Brick – Concrete interface.

4.1.1 Preparation of Test Specimens

- All RRM panels were made to 300mm thick which is the minimum practically possible thickness to be constructed.
- 9″x6″ size of rubble were used since this is the most commonly used size for constructing retaining walls
- 1:5 (cement: sand) mortar mix was used for all panels as it is the widely used mortar mix for RRM.
- Thickness of Mortar joints were maintained within 6-20mm and flush joints were used.
- Curing: All specimens were closely covered with polythene sheets and maintained them undisturbed until testing.
- Specimens for Flexure and shear tests were pre-compressed by placing concrete test cubes on the top surface of the specimen, which is equivalent to $3.6 \times 10^{-3}\ N/mm^2$ pre- compression.

Figure 4.1:(a) shows the preparation of specimens and the pre-compressed specimens are shown in Figure 4.1:(b)

Figure 4.1: (a) – Preparing samples for Flexural Strength Test

Figure 4.1: (b) – Pre-compressed Specimens

4.2 Experimental Set-up

4.2.1 Testing for Flexural Strength of RRM

Two types of test specimens were constructed for the following two cases;

1) When the plane of bending is vertical - (see Figure 4.2(a))
2) When the plane of bending is horizontal - (see Figure 4.2(b))

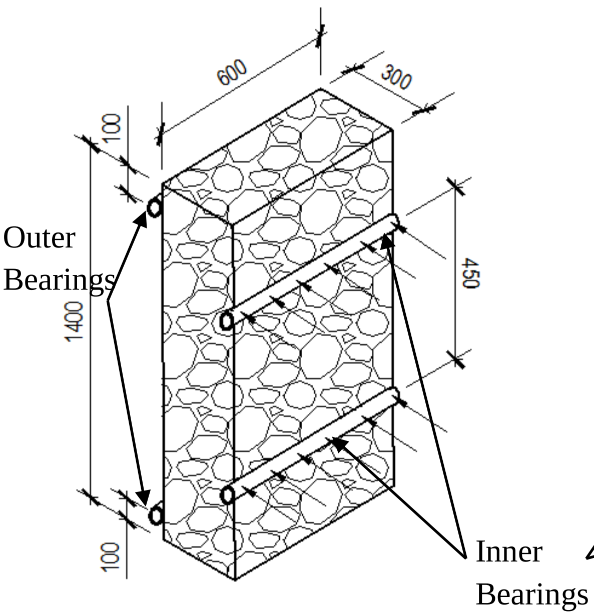


Figure 4.2 (a): The Plane of Bending is Vertical

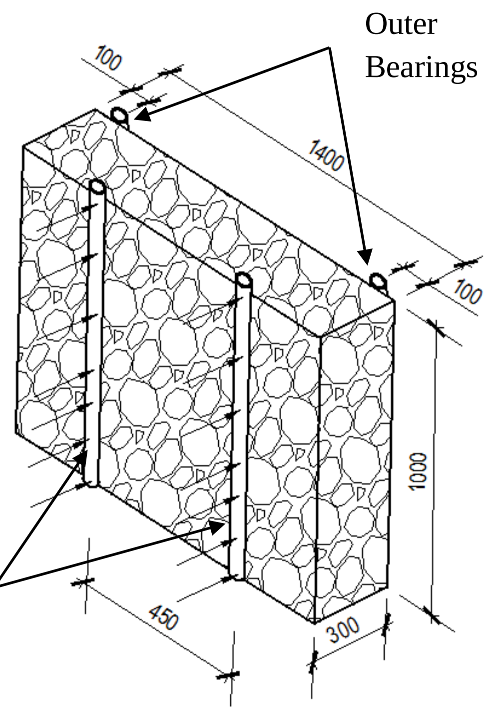
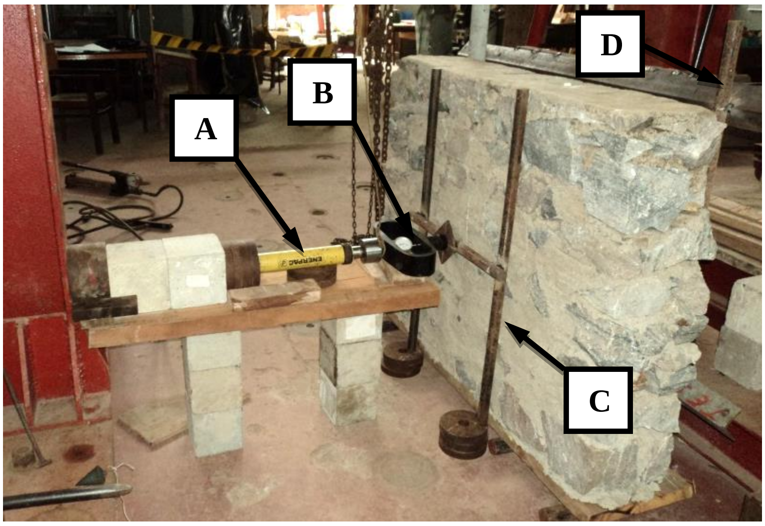


Figure 4.2 (b): The Plane of Bending is Horizontal

Three samples were tested under four point loading for each type of test. Loads were applied to samples through two inner bearings while two outer bearings were used to support them laterally.

A gradually increasing load was applied using a hydraulic jack (Manufacturer: EVERPAC) on inner bearing via Proving ring (Manufacturer : MARUTO). Proving ring with 100 kN (10 Tons) load capacity had been used for all 6 specimens. The test set-ups are shown in Figure 4.3 and Figure 4.4.



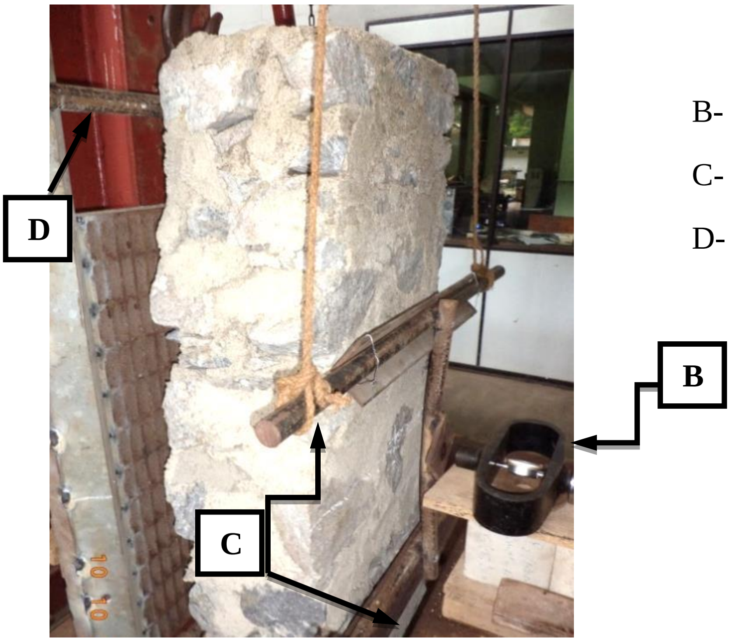
A – Hydraulic jack

B- Proving ring

C- Inner bearings

D- Outer bearings

Figure 4.3: The Test set up When the Plane of bending is horizontal



B- Proving ring

C- Inner bearings

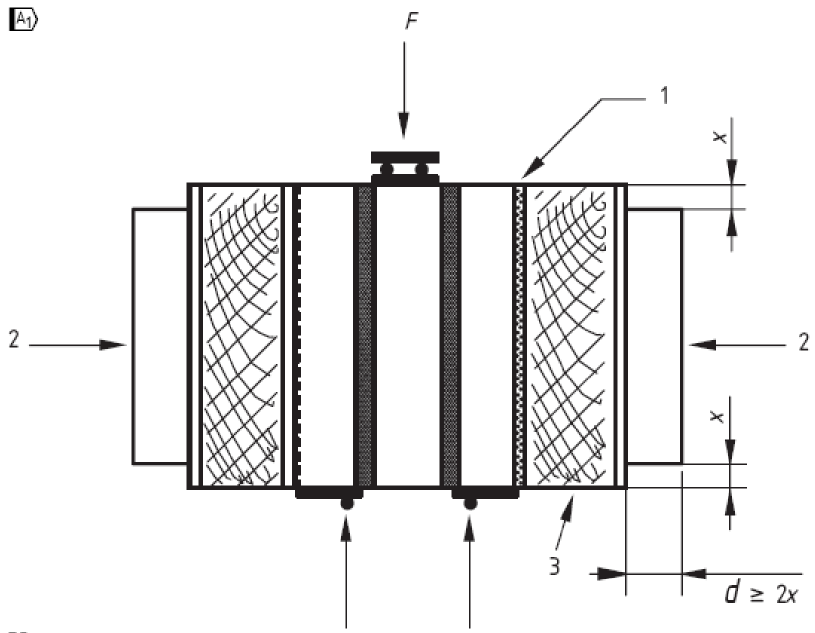
D- Outer bearings

Figure 4.4: The test set up for Specimens when the Plane of Bending is Vertical

The dimensions of the samples to the nearest 5mm and maximum dial gauge reading for all cases were recorded. Appendix A includes all data relevant to results of the experiment. Only planes of failure occurred within the inner bearings were identified as satisfactory failure modes as this was the region when pure bending was expected.

4.2.2 Testing for Shear Strength of RRM

BS EN 1052-3: 2002 [3] specifies triplet test for investigating shear strength of masonry units which are laid in a specific bond pattern. This test provides two straight failure planes which are parallel to the masonry units. Figure 4.5 indicates the set-up for triplet test.



Key

1 Strawboard/softboard/gypsum plaster

2 Precompression

3 Loading beam

d depth of loading beam

x length by which the loading beam extends beyond the end of the platen

Figure 4.5: Set up for Triplet tests as in [3]

Source: BS EN 1052-3: 2002 [3]

In the case of RRM, this type of straight failure surfaces cannot be expected. Further, the shear failure of a retaining wall provides single plane shear failure. Hence, testing set-up shown in the Figure 4.6 was adopted. In this set-up, supporting plates and loading plates were positioned so that only single failure plane is obtained.

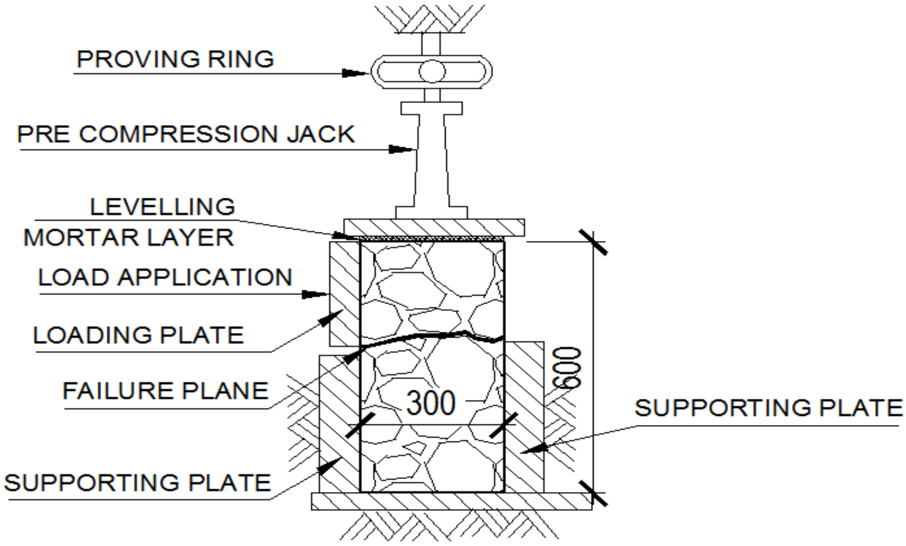
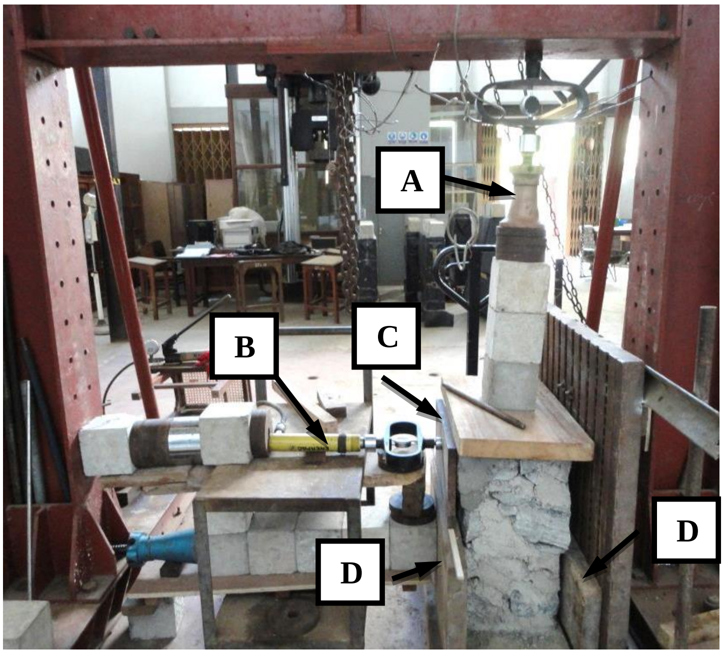


Figure 4.6: Set up adopted for Shear Strength Test

The pre- compression was applied with a hydraulic jack through a proving ring of 100kN capacity. The shear load was applied with the hydraulic jack through the proving ring of 29 kN (3 Tons) capacity.

Six samples of 600mm × 300mm ×450mm were tested. Three were tested for zero pre- compression and the rest were tested with 4.5 kN, 6.0 kN, 9.0 kN pre-compression forces. These forces were selected as they reflect the brick wall weights of typical heights, built on the retaining walls. The pre- compression of 6kN is equivalent to the compression due to the weight of 10′ high 9″ thick brick wall. However, during the application of shear loading, pre-compression load increased due to the slight inclination of the specimen. The pre-compression load at the time of failure was recorded as the pre- compression load.

Figure 4.7 shows the typical loading arrangement for shear strength test. Dimensions of the samples to the nearest 5mm and maximum dial gauge readings for all cases were recorded and included in Appendix B.



A – Screw Jack

B- Hydraulic Jack

C- Loading Plate

D- Supporting Plates

Figure 4.7: Test set up for Shear Strength Test

4.2.3 Testing for Shear Strength at Concrete- RRM Interface.

Six numbers of test specimens were prepared by casting a 225mm × 225mm Concrete beam on 300mm thick 450mm × 450mm RRM samples. When preparing specimens, concrete mix was directly poured on the top surface of the RRM samples. The concrete mix of 1: 1.5: 3 (19mm) was used. Figure 4.8 illustrates Set up for investigating shear strength at Concrete-RRM interface test

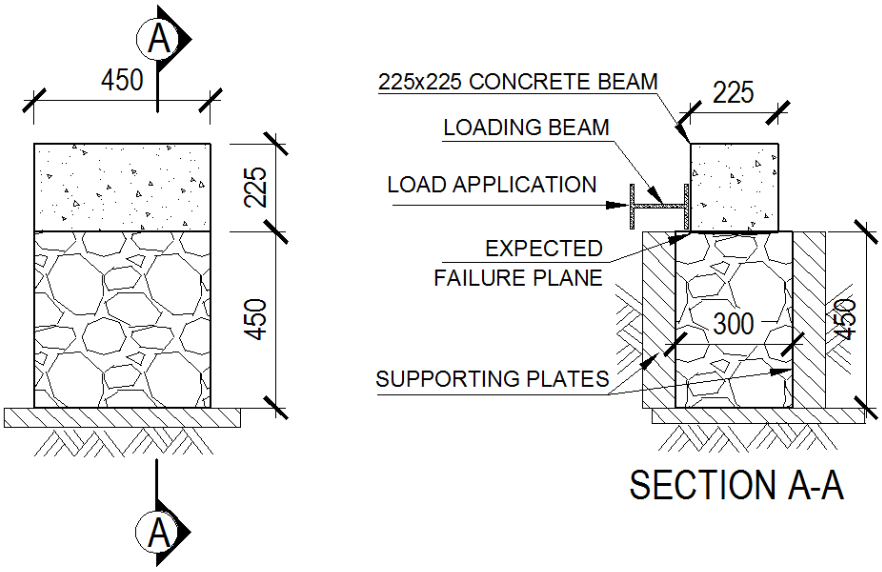
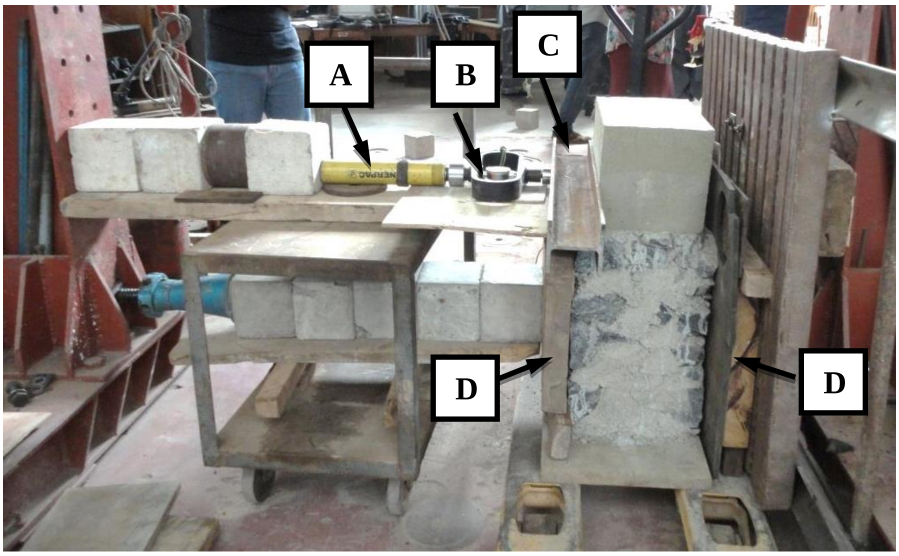


Figure 4.8: Set up for investigating Shear Strength at Concrete-RRM Interface Test

The shear load was applied with the hydraulic jack through the proving ring of 29kN (3 Tons) capacity. Dimensions of the RRM samples and concrete tie beam to the nearest 5mm and maximum dial gauge readings for all cases were recorded and included in Appendix C. Figure 4.9 illustrates the set up for Shear strength at RC-RRM interface test.



A – Hydraulic Jack

B- Proving Ring

C- Loading Beam

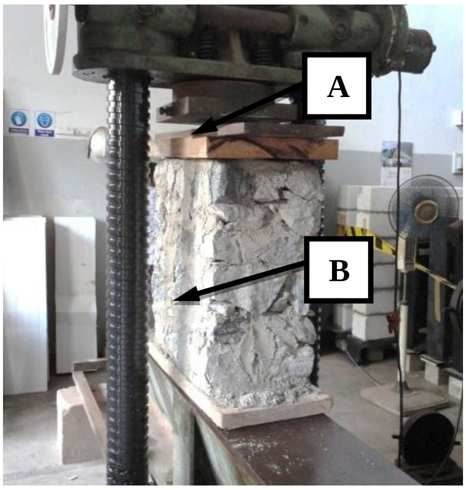
D- Supporting Plates

Figure 4.9: The set up for Shear strength at RC- RRM interface test

4.2.4 Testing for Compressive Strength of RRM

Three numbers of 300 thick 600mm×600mm RRM samples were used for the investigation. The sizes of samples are in conformity with the BS EN 1052-1: 1998 and also are similar to those tested by Chandrakeerthy[5,15].

The specimens were loaded by means of a 200 Ton-Amsler compression testing machine. Steel and timber platens were provided to the top surface of the sample in order to have a uniform pressure distribution over the surface. The steady loading rate of 0.15N/(mm^2.min) was maintained for all samples. The test set-up for Compressive Strength of RRM is shown in Figure 4.10.



A – Compression Testing Machine

B – Loading Platens

Figure 4.10: Set up for Compressive Strength Test

Dimensions of the RRM samples to the nearest 5mm and maximum dial gauge readings for all cases were recorded. Refer Appendix D for the results.

Chapter 05

ANALYSIS OF TEST RESULTS

5.1 Flexural Strength

5.1.1 Experimental Results

a) When the Plane of Bending is horizontal

The Dimensions of the test panels and test results when the plane of bending is horizontal are given in the Table 5.1.

Table 5.1: Results of the test on Flexural Strength (When the Plane of Bending is horizontal)

Specimen	**Dimensions/ (mm)**			**Load /(N)**	**Comments on Failure Pattern**
	Average Width	**Average Height**	**Average Thickness**		
1	1378	1000	300	19,078	Not satisfactory, Failure at an outer bearing
2	1400	1000	300	37,278	Satisfactory
3	1366	1000	300	39,874	Satisfactory

The failure patterns of the test specimens are shown in Figures 5.1(a) – (c).

Figure 5.1 (a) – Specimen 01

Figure 5.1 (b) – Specimen 02

Figure 5.1 (c) – Specimen 03

Figure 5.1 (c) – Specimen 03

Figure 5.1: Failure patterns of Specimens (When the Plane of Bending is horizontal)

b) When the Plane of Bending is vertical

The Dimensions of the test panels and test results when the plane of bending is vertical are given in the Table 5.2.

Table 5.2: Results of Flexural Strength Test (When the Plane of Bending is Vertical)

Specimen	Dimensions/ (mm)			Load/ (N)	Comment on Failure Pattern
	Average Width	Average Height	Average Thickness		
4	560	1400	300	5182	Satisfactory
5	565	1390	300	5182	Satisfactory
6	560	1380	300	5182	Satisfactory

The failure patterns of the test specimens 4, 5 and 6 are shown in Figures 5.2(a) – (c).

Figure 5.2 (a) – Specimen 4

Figure 5.2 (b) – Specimen 5

Figure 5.2 (c) – Specimen 6

Figure 5.2: Failure Patterns of Specimens (When the Plane of Bending is Vertical)

5.1.2 Evaluation of Results

The flexural strength of each specimen can be calculated by using following formula [2].

$$f_{xi} = \frac{3F_{i,max}(l1 - l2)}{2bt_u^2}\ N/mm^2$$

Where,

f_{xi} - Flexural strength of an individual masonry specimen, (N/mm^2)
$F_{i,max}$ - Maximum load, (N)
$l1$ - Spacing of outer bearings, (mm)
$l2$ - Spacing of inner bearings, (mm)
b - Width of the section in the plane of bending, (mm)
t_u - Width of the masonry unit, (mm)

The Flexural Strength of Specimen 2,

$$f_{xi} = \frac{3 \times 37278 \times (1200 - 450)}{2 \times 1000 \times 300^2}\ N/mm^2$$

$$= 0.47\ N/mm^2$$

Similarly, flexural strengths of all specimens were evaluated and tabulated in Table 5.3. The result of specimen 1 is discarded due to unsatisfactory failure mode.

Table 5.3: Flexural Strength of RRM Specimens

Plane of Bending	Specimen No	f_{xi} (N/mm^2)
Horizontal	1	-
	2	0.47
	3	0.50
Vertical	4	0.12
	5	0.11
	6	0.12

The characteristic flexural strength of masonry units can be calculated by using the following formula [2].

$$f_{xk} = \frac{f_{mean}}{1.5}$$

Where,

f_{xk} - Characteristic flexural strength of masonry,(N/mm^2)

f_{mean} - Mean flexural strength of masonry specimens, (N/mm^2)

This formula is specified for five specimens, however due to non-availability of results of five specimens for each Plane of Bending case and the deviation of the results is very small, the Mean Flexural Strength is calculated using the available test results.

When the Plane of Bending is horizontal, the Characteristic Flexural Strength of RRM is:

$$f_{xk} = \frac{(0.47 + 0.5)/2}{1.5}$$

$$= \mathbf{0.32\ N/mm^2}$$

When the Plane of Bending is vertical, the characteristic flexural strength of RRM is:

$$f_{xk} = \frac{(0.12 + 0.11 + 0.12)/3}{1.5}$$

$$= \mathbf{0.08\ N/mm^2}$$

The characteristic flexural strength values obtained for each case are tabulated in Table 5.4.

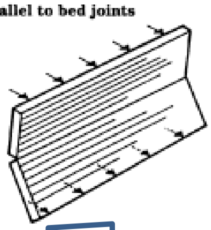
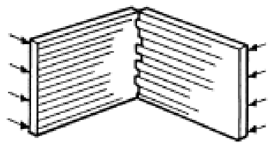
Table 5.4: Characteristic Flexural Strength of RRM

Plane of Bending	Characteristic Flexural strength (N/mm^2)
Horizontal	0.32
Vertical	0.08

5.1.3 Comparison of Test Results with Flexural Strength of Brick/ Block Masonry.

Table 5.5 indicates the flexural strength values given in BS 5628-1:1992.

Table 5.5: Flexural strength of Brick and Block Masonry as per BS 5628-1:1992 [16]

	Plane of failure parallel to bed joints 			Plane of failure perpendicular to bed joints 		
Mortar designation	(i)	(ii) and (iii)	(iv)	(i)	(ii) and (iii)	(iv)
Clay bricks having a water absorption						
less than 7 %	0.7	0.5	0.4	2.0	1.5	1.2
between 7 % and 12 %	0.5	0.4	0.35	1.5	1.1	1.0
over 12 %	0.4	0.3	0.25	1.1	0.9	0.8
Calcium silicate bricks	0.3		0.2		0.9	0.6
Concrete bricks	0.3		0.2		0.9	0.6
Concrete blocks (solid or hollow) of compressive strength in N/mm^2:						
2.8 3.5 7.0 — used in walls of thickness[a] up to 100 mm	0.25		0.2		0.40 0.45 0.60	0.4 0.4 0.5
2.8 3.5 7.0 — used in walls of thickness[a] 250 mm	0.15		0.1		0.25 0.25 0.35	0.2 0.2 0.3
10.5 14.0 and over — used in walls of any thickness[a]	0.25		0.2		0.75 0.90[b]	0.6 0.7[b]

The mortar mix of 1:5 (cement: sand) corresponds to mortar designation (iii) (highlighted area). The Table 5.4 shows that characteristic flexural strength result of RRM (when the Plane of Bending is horizontal) which is comparable with those of block masonry. The test result of RRM (when the Plane of Bending is vertical) is very much smaller when compared with the brick and block masonry.

5.2 Shear Strength

5.2.1 Experimental Results

The Dimensions of the test panels and results for the Test on Shear Strength are given in the Table 5.6.

Table 5.6: Results of the Test on Shear Strength

Specimen	Dimensions/ (mm)			Load (N)	Pre-Compression Load/ (N)	Failure Pattern
	Average Width	Average Height	Average Thick.			
1	455	600	300	8092	4834	Satisfactory
2	455	600	300	13841	9527	Satisfactory
3	450	600	300	4640	12133	unsatisfactory
4	450	600	305	5273		Satisfactory
5	450	605	310	8954		Satisfactory
6	450	600	310	3374		Satisfactory

The failure patterns of the test specimens are shown in Figures 5.3(a) – (f).

Figure 5.3(a) – Specimen 01

Figure 5.3(b) – Specimen 02

Figure 5.3 (c) – Specimen 03

Figure 5.3 (d) – Specimen 04

Figure 5.3 (e) – Specimen 05

Figure 5.3 (f) – Specimen 06

Figure 5.3: Failure Patterns of Specimens 1 – 6

5.2.2 Evaluation of Results

The shear strength and the pre- compressive stress on a sample can be calculated by using following formulae [3].

$$f_{voi} = \frac{F_{i,max}}{A_i}\ N/mm^2$$

$$f_{pi} = \frac{F_{pi}}{A_i}\ N/mm^2$$

Where,

f_{voi} - Shear strength of an individual sample, (N/mm2)

f_{pi} - Pre-compressive stress of an individual sample, (N/mm2)

$F_{i,max}$ - Maximum shear force, (N)

F_{pi} - Pre-compressive force, (N)

A_i - Cross sectional area of a sample parallel to the shear load, (mm)

The shear strength of specimen 1,

$$f_{voi} = \frac{8092}{455 \times 300}\ N/mm^2$$

$$f_{voi} = 0.059\ N/mm^2$$

The pre-compressive stress on specimen 1,

$$f_{pi} = \frac{F_{pi}}{A_i}\ N/mm^2$$

$$= \frac{4834}{455 \times 300}\ N/mm^2$$

$$= 0.035\ N/mm^2$$

Similarly, shear strength results for all specimens are calculated and presented in Table 5.7

Table 5.7: Shear Strength results for different Pre-Compressive Stresses

Specimen No	Shear Strength (f_{voi}) Nmm^{-2}	Pre-compressive Stress (f_{pi}) Nmm^{-2}
1	0.059	0.035
2	0.101	0.070
3	0.034	0.090
4	0.038	0.000
5	0.064	0.000
6	0.024	0.000

Figure 5.4 shows graphically the variation of Shear Strength with Pre-Compressive Strength.

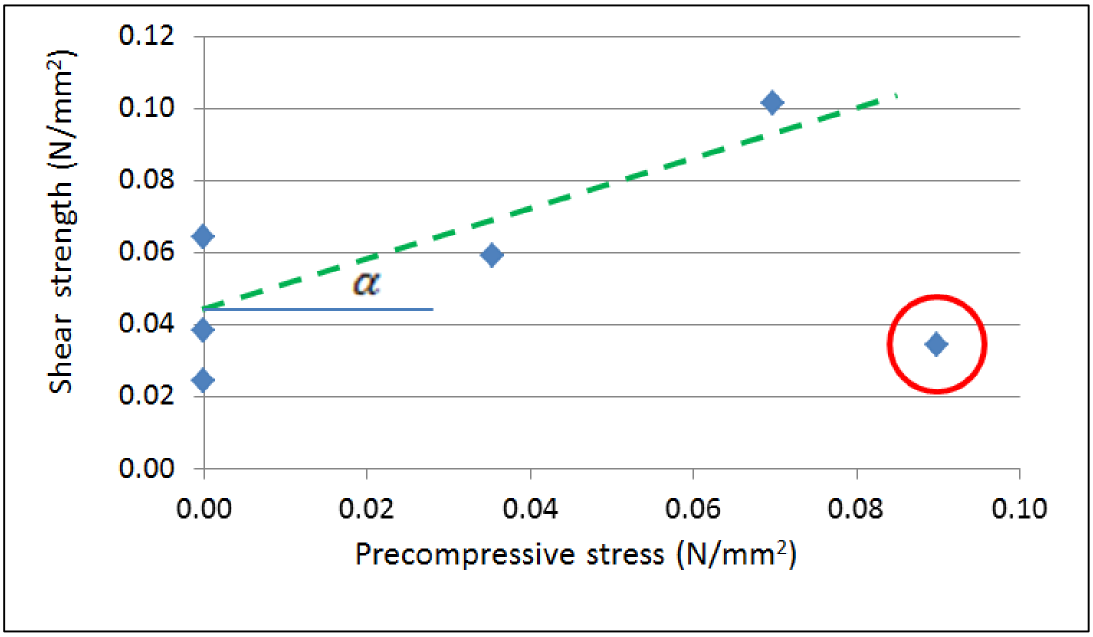


Figure 5.4: Variation of Individual Shear Strength values with the Pre-Compressive Stresses

When plotting the variation of shear strength with pre-compressive stresses, the results correspond with specimen 3 (which is highlighted) was discarded since its failure mode was not satisfactory.

As specified in BS EN 1052-3:2002 [3], shear strength parameters can be evaluated as follows,

Referring to Figure 5.4,

Initial shear strength,

f_{vo} = 0.042 N/mm^2 (Shear strength when pre-compression is zero)

Angle of Internal Friction,

$$\alpha = \tan^{-1}\left[\frac{(0.1 - 0.042)}{0.08}\right]$$

$$= \tan^{-1}[0.725]$$

$$= 35.9°$$

The Characteristic Initial Shear Strength,

$$f_{vok} = 0.8 f_{vo}$$

$$= 0.8 \times 0.042$$

$$= \mathbf{0.033\ N/mm^2}$$

The Characteristic Angle of Internal Friction,

$$\alpha_k = \tan^{-1}[0.8 tan\alpha]$$

$$= \tan^{-1}[0.8 \times 0.725]$$

$$= \mathbf{30.1°}$$

The Characteristic frictional coefficient,

$$\mu_k = \tan \alpha_k$$

$$= \tan 30.1$$

$$= \mathbf{0.57}$$

5.2.3 Comparison of Results with previous research findings.

The investigation carried out by J.Milosevic et al. [17] reported the Characteristic Shear Strength parameters of RRM based on the triplet test. According to Milosevic's et al. [17] findings,

Characteristic initial shear strength of RRM = 0.16 N/mm^2
Characteristic frictional coefficient ($\tan \alpha$) = 0.98

Compared to above values, the results obtained for local RRM in this research study are extremely low.

The samples used for Milosevic's investigation were built in three stone layers and the middle layer was pushed through other two layers. Hence, this test doesn't make a fully representation of the actual shear strength of Random Rubble Masonry of which well-defined failure surfaces cannot be expected.

There can be differences of strength parameters due to dissimilar mortar designations and the difference of workmanship. However, the mortar designation which had been used for making samples was not mentioned in the research paper.

5.3 Shear Strength at Concrete- RRM interface.

5.3.1 Experimental Results

Experimental results for all 6 specimens are summarized and shown the Table 5.8. Failure modes of all samples are illustrated subsequently in Figures 5.5(a)-(f).

Table 5.8: Results of Shear Strength Test

Specimen	Dimensions of Tie beam/ (mm)			Dimensions of Masonry Panel/ (mm)			Load /(N)	Failure Surface
	Length	Height	Width	Width	Height	Thick.		
1	450	225	225	450	600	300	3201	Masonry-Masonry
2	450	225	225	450	600	300	5675	Masonry-Masonry
3	450	225	225	450	600	300	12117	Concrete-Masonry
4	430	225	225	430	600	300	6366	Concrete-Masonry & Masonry-Masonry
5	450	225	225	450	600	300	14128	Concrete-Masonry
6	450	225	225	450	600	300	5503	Masonry-Masonry

Figure 5.5 (a) – Specimen 1

Figure 5.5 (b) – Specimen 2

Figure 5.5 (c) – Specimen 3

Figure 5.5 (d) – Specimen 4

Figure 5.5 (e) – Specimen 5

Figure 5.5 (f) – Specimen 6

Figure 5.5: Failure Pattern of Specimens Used for Shear Test,

(a) - 1, (b) -2, (c) -3, (d) - 4, (e) -5, (f)-6

5.3.2 Evaluation of Results

In some cases, failure has not been occurred at the Concrete- Masonry interface as expected. Instead, it has occurred at Masonry- Masonry interface.

Only samples failed at Concrete- Masonry interface can be used for evaluating the shear strength at the concrete-masonry interface. Indeed, the other results with Masonry- Masonry interface failure was due to the shear failure of RRM. The results of the sample (sample 4) of which failure occurred across both failure surfaces were discarded.

The shear strength at the Concrete- Masonry interface,

$$f_{voti} = \frac{F_{i,max}}{A_{ti}} \ N/mm^2$$

Where,

f_{voti} - Shear strength at the Con.- Masonry of an individual sample, (N/mm2)

$F_{i,max}$ - Maximum shear force, (N)

A_{ti} - Cross sectional area of the tie beam parallel to the shear load, (mm)

The shear strength at Concrete- Masonry interface of specimen 3,

$$f_{voti} = \frac{12117}{225 \times 450} \ N/mm^2$$

$$= 0.120 \ N/mm^2$$

The shear strength of RRM can be evaluated by using the same equation as in 5.2.2.

The shear strength of specimen 2,

$$f_{voi} = \frac{F_{i,max}}{A_i} \ N/mm^2$$

$$= \frac{5675}{450 \times 300} \ N/mm^2$$

$$= 0.042 \ N/mm^2$$

Similarly, shear strength results for all specimens are calculated and presented in Table 5.9.

Table 5.9: Results of test carried out for Shear Strength at Concrete- Masonry Interface

Specimen	Shear strength at Con.- Masonry (f_{voti}) /Nmm^{-2}	Shear strength of RRM (f_{voi}) /Nmm^{-2}
1	-	0.024
2	-	0.042
3	0.120	-
4	-	-
5	0.140	-
6	-	0.041

The Average shear strength at Concrete- Masonry interface is 0.13 N/mm^2

As evaluated in section 5.2.2, Characteristic shear strength at Concrete- Masonry interface,

$$f_{votk} = 0.8 \times f_{vot}\ N/mm^2$$

$$= 0.8 \times 0.13\ N/mm^2$$

$$= 0.104\ N/mm^2$$

5.3.3 Comparison of Results with previous research findings on Shear strength at Concrete- Masonry interface

According to the findings of Premadasa et al.[4] average shear strength at the interface of 10mm mortar joint with 1:5 mortar designations was 0.2 N/mm^2. For other mortar designations and for increased mortar thickness lesser strength values of shear strength have been observed.

In this study, the average shear strength at the interface of Concrete- RRM was 0.13N/mm^2 which is slightly lower than the Brick- Concrete interface.

5.4 Compressive Strength.

5.4.1 Experimental Results

The Dimensions of the test panels and results for the Test on Compressive Strength are given in the Table 5.10.

Table 5.10: Results of Compressive Strength Test

Specimen	Dimensions/ (mm)			Load/ (N)
	Average Width	Average Height	Average Thickness	
1	595	600	300	147150
2	570	600	300	128511
3	600	600	300	209934

5.4.2 Evaluation of Results

5.4.2.1 Compressive Strengths of each Sample

The Compressive strength of each sample can be calculated by using following equation [1].

$$f_i = \frac{F_{i,max}}{A_i} \ N/mm^2$$

Where,

f_i - Compressive strength of an individual sample, (N/mm2)

$F_{i,max}$ - Maximum load on a sample, (N)

A_i - Loaded cross sectional area of an individual sample, (mm)

The compressive strength of specimen 1,

$$f_i = \frac{147150}{595 \times 300} \; N/mm^2$$

$$= 0.82 \; N/mm^2$$

Similarly, compressive strength of other specimens were calculated and tabulated in Table 5.11.

Table 5.11: Compressive Strength Results of Each Sample

Specimen	Compressive strength (f_i) /Nmm^{-2}
1	0.82
2	0.75
3	1.17

5.4.2.2 Mean Compressive Strength

Mean compressive strength,

$$f = \frac{(f_1 + f_2 + f_3)}{3} \; N/mm^2$$

$$= \frac{(0.82 + 0.75 + 1.17)}{3} \; N/mm^2$$

$$= 0.91 \; N/mm^2$$

5.4.2.3 Characteristic Compressive Strength

Characteristic Compressive strength as specified in [1],

$$f_k = \frac{f}{1.2} \; or \; f_{i,min} \qquad \text{Whichever is the smaller}$$

$= 0.76\ N/mm^2$ or $0.75 N/mm^2$, whichever is the smaller.

$$f_k = 0.75\ N/mm^2$$

5.4.3 Comparison of Results with previous research findings on Compressive Strength of RRM

Chandrakeerthy[5,15] has investigated on the compressive strength of RRM for Sri Lankan rubble stones using 300mm thick 600mm X 600mm panels. Test results reported in the study are given in Table 5.12.

Table 5.12: Characteristic Compressive Strength of RRM for Mortar designation of 1:5

Mortar Designation	Mortar Mix (cement: sand)	Compressive Strength of Stones (N/mm^2)						
		20	30	40	50	60	80	100
(iii)	1:5	1.07	1.60	1.84	2.08	2.31	2.31	2.31

Source: Chandrakeerty [5]

It is reported that, tests have been carried out for various strength grades of stones. Characteristic compressive strength of RRM with stones of 20 N/mm^2 compressive strength was slightly greater than the results obtained in this research study.

5.5 Summery of Test Results obtained by the Experimental Study

Table 5.13 gives the summery of Test results obtained by this research study.

Table 5.13: Summery of Strength Parameters of RRM

Strength Parameter	**Characteristic Strength / (N/mm^2)**
Flexural Strength (When plane of bending is horizontal)	0.32
Flexural Strength (When plane of bending is vertical)	0.08
Shear Strength	0.033
Shear Strength at Concrete- RRM interface.	0.104
Compressive Strength	0.75

Chapter 06

CONCLUSIONS AND RECOMMENDATIONS

6.1 Use of Experimental Results for the improvements of RRM Retaining wall Design

In chapter 3, three different design approaches were considered. The results obtained for those three cases are summarized in Table 6.1.

Table 6.1: Results obtained through different Design Approaches

Parameter	**Case 1**-Conventional Retaining wall	**Case 2**- Retaining wall assuming RRM will not fail due to flexure	**Case 3**- Retaining wall with a Tie back
Base Width /m	2.1	1.5	1.5
Max. Bearing Pressure / kNm^{-2}	135.3	246	132.8
Min. Bearing Pressure / kNm^{-2}	2.7	0	5.2
Flexural stress at the heel/ Nmm^{-2}	-	0.061	-
Shear stress at Concrete - Masonry interface/ Nmm^{-2}	-	-	0.044
Shear stress at the narrowest masonry section/ Nmm^{-2}	Negligible	Negligible	0.028
Factor of Safety - Overturning	3.12	1.59	1.92
Factor of Safety - sliding	2.12	1.51	1.6
Percentage of base width reduction / %	-	28	28

6.1.1 Flexural Strength.

According to the above summery, if RRM is able to withstand 0.061N/mm^2 flexural stress, the base width of the conventional design can be reduced by 28%. However, the maximum bearing pressure is increased up to 246 kN/m^2

The results indicated that, characteristic flexural strength of RRM (when the plane of bending is vertical) is 0.08 N/mm^2. Hence there is a possibility for an optimization of RRM retaining wall design considering the effect of flexural strength of RRM, However it depends on the bearing capacity of the soil.

6.1.2 Effect of Tie back

If the concrete- masonry interface can transfer 0.044 N/mm^2 shear stress and RRM can withstand 0.028 N/mm^2 shear stress, the base width selected for the conventional design could be reduced up to 28%.

According to the experimental results, characteristic shear strengths at the Concrete-Masonry interface and of RRM are 0.104 N/mm^{2,} 0.033 N/mm^2 respectively. Hence reducing the base width up to 28% with aid of the Tie back effect is possible.

6.2 Adopting Results of the Study for Other Retaining Heights

The extent of optimization achievable for some other retaining heights is illustrated in the Table 6.3. The same design information which are listed in chapter 3, are assumed for all retaining heights.

Table 6.2: Extent of Optimization for 1-3m Retaining Heights

Soil Retaining Height	**Parameter**	**Case 1**-Conventional Retaining wall	**Case 2**- Retaining wall assuming RRM will not fail due to flexure	**Case 3**- Retaining wall with a Tie back
1	Base Width /m	1	0.75	0.6
	Percentage of base width reduction / %	-	25	40
2	Base Width /m	1.55	1.15	1
	Percentage of base width reduction / %	-	26	35
3	Base Width /m	2.1	1.5	1.5
	Percentage of base width reduction / %	-	28	28

From the results, it can be concluded that base width of the retaining wall can be optimized considerably considering the effect of tie-back and allowing flexural stress, for retaining heights bellow 3m.

6.3 Suggestions for Future Works

1) Carry out investigation for other mortar designations.

In this investigation, all samples were made with 1:5 (cement: sand) mortar mix. ICTAD specification [14] specifies mortar mixes of 1:5, 1:6, 1:7 and 1:8 for RRM. 1:5 mortar mix was selected for this investigation, as it is the most commonly used mortar mix for outdoor and below ground level applications [5]. However there are some instances where 1:6 mortar mix is also adopted for this type of applications. Hence, it is suggested to carryout investigation by using 1:6 mortar mix as well.

2) Investigation of the effect of strength of Rubble to the strength parameters of RRM.

It is a known fact that not only the strength of mortar, but also the strength of the masonry unit affects the strength parameters of Masonry.
However, in this investigation, the compressive strength of rubble samples was not investigated. Therefore it is recommended to investigate the influence of strength of different rubble stones on different strength parameters of RRM.

3) To increase the number of test specimens

During the experimental investigation, there were several instances where failure patterns of the specimen were not acceptable. Hence results of those specimens had to be discarded. Also there can be damages during handling of specimens.

Therefore it is necessary to test more samples to get more appropriate characteristic strength values.

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Appendix A: Flexural Strength -Experimental Data and Results

A.1 Flexural Strength When Plane of Bending is Horizontal

A.1.1 Experimental Data

Date of making samples : 12-July-2013

Date of Testing : 2-September-2013

Distance between Outer bearings : 1200mm

Distance between inner bearings : 550mm

A.1.2 Experimental Results

Table A.1: Results of flexural strength test (When plane of bending is horizontal)

Specimen	Dimensions/ (mm)			Dial gauge reading	Load* /(N)	Comments on failure pattern
	Average Width	Average Height	Average Thickness			
1	1378	1000	300	110	19,078	Not satisfactory, Failure at an outer bearing
2	1400	1000	300	215	37,278	Satisfactory
3	1366	1000	300	230	39,874	Satisfactory

* The Dial gauge reading was converted into load in N, referring to the calibration Table of the Proving ring (Appendix- E)

A.2 Flexural Strength When Plane of Bending is Vertical

A.2.1 Experimental Data

Date of making samples: 16-Jul-2013

Date of Testing : 10-Oct-2013

Distance between Outer bearings : 1200mm

Distance between inner bearings : 550mm

A.2.2 Experimental Results

Table A.2: Results of flexural strength test (When plane of bending is vertical)

Specimen	Dimensions/ (mm)			Dial gauge reading	Load* / (N)	Comment on failure pattern
	Average Width	Average Height	Average Thickness			
4	560	1400	300	30	**5182**	No clear crack pattern
5	565	1390	300	30	**5182**	No clear crack pattern
6	560	1380	300	30	**5182**	No clear crack pattern

* The Dial gauge reading was converted into load in N, referring to the calibration Table of the Proving ring (Appendix- E)

Appendix B: Shear Strength -Experimental Data and Results

B.1 Experimental Data

Date of making samples: 31-Jul-2013 to 2-Aug-2013

Date of Testing : 24-Oct-2013

B.2 Experimental Results

Table B.1: Results of Shear Strength Test

Specimen	Dimensions/ (mm)			Dial gauge reading	Load* /(N)	Pre-Compression		Failure pattern
	Average Width	Average Height	Average Thick.			Dial gauge reading	Load (N)	
1	455	600	300	140	8092	28	4834	Satisfactory
2	455	600	300	240	13841	55	9527	Satisfactory
3	450	600	300	80	4640	70	12133	unsatisfactory
4	450	600	305	91	5273			Satisfactory
5	450	605	310	155	8954			Satisfactory
6	450	600	310	58	3374			Satisfactory

* The Dial gauge reading was converted into load in N, referring to the calibration Table of the Proving ring (Appendix- E)

Appendix C: Shear Strength at Concrete Masonry Interface - Experimental Data and Results

C.1 Experimental Data

Date of making samples : 27-July-2013

Date of casing of RC beam : 30- July-2013

Date of Testing : 29-Oct-2013

C.2 Experimental Results

Table C.1: Results of Shear strength at concrete masonry interface

Specimen	Dimensions of Tie beam/ (mm)			Dimensions of Masonry panel/ (mm)			Dial gauge reading	Load * /(N)	Failure surface
	Length	Height	Width	Width	Height	Thick.			
1	450	225	225	450	600	300	55	3201	Masonry- Masonry
2	450	225	225	450	600	300	98	5675	Masonry- Masonry
3	450	225	225	450	600	300	210	12117	Concrete- Masonry
4	430	225	225	430	600	300	110	6366	Concrete- Masonry & Masonry- Masonry
5	450	225	225	450	600	300	245	14128	Concrete- Masonry
6	450	225	225	450	600	300	95	5503	Masonry- Masonry

* The Dial gauge reading was converted into load in N, referring to the calibration Table of the Proving ring (Appendix- E)

Appendix D: Compressive Strength -Experimental Data and Results

D.1 Experimental Data

Date of casting: 17-July-2013

Date of Testing: 24-Oct-2013

D.2 Experimental Results

Table D.1: Results of Compressive strength test

Specimen	Dimensions/ (mm)			Dial gauge reading /(Ton)	Load*/ (N)
	Average Width	Average Height	Average Thickness		
1	595	600	300	15	147150
2	570	600	300	13.1	128511
3	600	600	300	21.4	209934

* The Dial gauge reading was converted into load in N, referring to the calibration Table of the Proving ring (Appendix- E)