Static Behaviour and Strength of Adhesively Bonded Anchors in Concrete and Masonry

Submitted by

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Abstract

This study deals with the static tensile behaviour and strength of epoxy-bonded anchors embedded in hollow clay brick, hollow concrete block, concrete and limestone. A total of 234 specimens were tested in direct tension, or under combined tension, shear and bending to determine their bond characteristic, mode of failure and strength. Other test variables included anchor diameter, embedment length, anchor inclination (vertical and 45-degree), concrete compressive strength and edge distance.

Results of the investigation show six typical failure modes: smooth pullout; anchor yielding and rupture; combined cone-bond pullout; base material splitting; breakout cone and brick web cracking. The anchor inclination led to a major reduction of the tensile capacity of the anchors. Using these observations, the experimental data, and some finite element analysis results, a design method is proposed for adhesively bonded anchors.

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List of Symbols

α	Strength factor for anchor due to fracture
β	Coefficient for bond strength
θ	Angle inclination, measured from the normal to the base material surface
φ	Average cone angle
ψ	Strength reduction factor dependent on prevailing design safety requirements or codes
ϕ	Strength reduction factor for base material
ϕ_c	Strength reduction factor for concrete
ϕ_s	Strength reduction factor for steel
ϕ_e	Strength reduction factor for epoxy
σ_r	Radial stress in base material (MPa)
σ_t	Tangential stress in base material (MPa)
σ_y	Yield stress of anchor (MPa)
τ	Bond stress along the anchor (MPa)
τ_{crack}	Minimum required bond stress to cause crack in base material (MPa)
γ	Angle between the principal compressive bond stress and the axis of the anchor
ν	Poisson ratio
σ_1	Major principle stress (MPa)
σ_2	Intermediate principle stress (MPa)

σ_3	Minimum principle stress (MPa)
σ_{xx}	Axial stress (MPa)
τ_{xy}	Shear stress (MPa)
λ	Length of the anchor (mm)
A	Cross-sectional area (size) of anchor rod (mm ²)
A_s	Effective area of anchor (mm ²)
A_e	Projected area of failure cone (mm ²)
A_c	Effective projected area of concrete cone failure (mm ²)
A_{no}	Actual projected area of failure cone or cones
A_n	Projected area of a single cone unaffected by edges
BR	Hollow clay brick (shallow embedment length)
HBR	Hollow clay brick (deep embedment length)
CMU	Hollow side concrete block
CMUE	Hollow edge concrete block
CON2	20.6 MPa solid concrete block
CON3	34.5 MPa solid concrete block
LS	solid limestone block
c	Distance from center of a fastener to edge of concrete (mm)
d	Diameter of anchor (mm)
d_o	Hole diameter (mm)
d_u	Diameter of anchor head (mm)
E	Young's modulus

r	Radius of anchor (mm)
k	Factor for post-install fasteners
k_1	Anchor inclination factor
k_2	Size effect for cone strength
k_3	Strength factor for cracking failure
$\tau_{avg.}$	Average bond strength (MPa)
f'_c	Concrete compressive strength measured on 150×300 mm cylinder (MPa)
f'_{cc}	Concrete compressive strength measured on 200 mm cube (MPa)
f_y	Nominal yield stress of anchor rod (MPa)
f'_b	Compressive strength of base material (MPa)
f_u	Specified ultimate tensile strength of anchor (MPa)
f	Shape factor for plastic modulus of cross section
f_t	Tensile strength of base material (MPa)
F_u	Ultimate tensile force of anchor steel (kN)
F_x	Component of ultimate tensile force of steel anchor in X coordinate (kN)
F_y	Component of ultimate tensile force of steel anchor in Y coordinate (kN)
h_{ef}	Effective anchor embedment (mm)
h_c	Failure cone depth (mm)
h_b	Effective bond length in cone-bond failure mode (mm)
L	Moment arm of bending stress applied on the anchor (mm)
e	Nearest edge distance from anchor to base material edge (mm)

$e_{critical}$	Critical edge distance to prevent cracking of base material (mm)
N_{crack}	Minimum force to cause crack line in base material (N)
N	Maximum tensile force for crack of base material failure (N)
N_u	Ultimate concrete tensile capacity (N)
s	Distance between fastens (spacing)
S	Elastic section modulus of the rod cross-section (mm ³)
T	Ultimate tensile cone-bond capacity (N)
T_1	Tensile capacity of bonded anchor (N)
T_2	Tensile capacity of bonded anchor (N)
T_3	Tensile capacity of bonded anchor (N)
T_{cone}	Ultimate cone capacity (N)
T_{bond}	Ultimate bond capacity (N)
P	Pullout (bond) resistance (N)
F	Ultimate tensile force of steel (N)
P_u	Failure load without edge effect (N)
P_{ue}	Failure load with edge effect (N)
u_e	Edge effect factor
Z	Plastic modulus of the cross section (mm ³)

Chapter 1

Introduction

1.1 General

Anchors used in concrete and masonry today can be divided into two general categories: cast-in-place and post-installed. Cast-in-place anchors are installed during the construction while post-installed anchors are placed in existing structures. Much research work has been carried out on cast-in-place anchors, while most post-installed anchors have not been adequately studied. Post-installed anchors include undercut anchors, expansion anchors, grouted anchors and adhesive anchors. Due to the advent of high-strength polyester, vinylester and epoxy adhesives and due to their lower cost, and ease and speed of installation, adhesive anchors are widely used in structural repair and retrofits works.

An adhesive anchor consists of a threaded steel rod or reinforcing bar inserted into a drilled hole, with the chemical adhesive injected into the hole acting as the bonding agent between the base material and steel. The strength of such an anchor depends on the strength of the rod, the base material, the rod-adhesive interface, the adhesive-base material interface, the embedment length, the anchor size, the distance of the anchor from the free edge of the base material (edge distance), the strength of the adhesive, and the type and rate of loading. Therefore, due to the many parameters which can affect the strength of adhesive anchor, one cannot develop a simple expression for calculating the strength of adhesively bonded anchors.

In the past two decades, extensive research has been conducted to determine the effect of the preceding parameters on the strength of adhesively bonded anchors in concrete. However, very little work has been done to determine the behaviour and strength of adhesively bonded anchors in such base materials as stone, clay brick and concrete block masonry. Yet, the latter materials are extensively used in construction and it may be necessary to embed anchors in them. For example, many building facade walls are made of the preceding masonry products and it may be necessary to anchor window frames to these types of walls. Currently, no design guidelines are available for the installation of adhesive anchors in masonry.

The aim of this study is to investigate the static behaviour and tensile strength of adhesively bonded anchors set in hollow clay brick, hollow concrete block, solid concrete and limestone. Based on the test data, and limited numerical and theoretical analyses, design guidelines will be developed to determine the strength of adhesively bonded anchors.

1.2 Research significance and problem definition

Adhesive anchors are increasingly used as fasteners to connect structural or non-structural elements to existing hardened concrete and masonry elements because they are easy to install and are economical. However, the behaviour of post-installed anchors is somewhat less understood than that of cast-in-place anchors. For instance, if post-installed anchors and chemical bonding agents are used for attaching windows frame to concrete or masonry elements, the following questions arise: (1) How strong should the anchor be? (2) How many anchors should be used? (3) How would the inclination of the

anchor rod relative to the surface of the structural element affect its strength? (4) What should the anchor embedment length be? (5) How close can the anchor be to the free edge of the structural element? In some applications, forces due to impact, blast and earthquake may need to be resisted by anchors, in which the dynamic response and strain rate effects must also be considered.

Currently, no design standard for adhesively bonded anchors exists in Canada or elsewhere and designers must rely on the design tables presented by anchor manufacturers. However, these tables only deal with the static behaviour of adhesive anchors installed in concrete and no further information is provided by the manufacturer to assist designers in other situations. Therefore, there is a clear need for more in-depth investigation.

1.3 Objectives and scope

The objectives of this study are:

- (1) To determine experimentally the load-displacement behaviour and strength of single adhesively bonded anchors installed in hollow clay brick, hollow concrete block, solid concrete and limestone under static tensile loading.
- (2) To perform linear elastic theoretical and numerical analyses to determine the stress distribution profile in the vicinity of the anchor in the base material.
- (3) To develop design guidelines for adhesively bonded anchors subjected to tension force, bending moment and shear force. The guideline shall be based on the experimental results and analysis.
- (4) To verify the proposed guideline by the data from the experimental work.

The test variables will include:

- (1) Different base materials including concrete, limestone, hollow concrete block and hollow clay brick
- (2) Anchor diameter
- (3) Embedment length of anchor
- (4) Edge distance of anchor (i.e. distance of anchor from the base material free edge)
- (5) Anchor inclination with respect to the horizontal surface of the base material.

The anchor system used in this study, including the chemical adhesive, is propriety. It is believed that they are representative of similar anchor systems available in the market, but the current results cannot be applied to other anchor systems. Verification of the applicability of the current results to other adhesive anchor systems is beyond the scope of the current study.

Chapter 2

Literature Review

2.1 Introduction

This chapter provides a detailed of background to the behaviour and strength of adhesive anchors in concrete and masonry. The recent research work on this topic can be traced back to Daws (1978), Usami et al. (1980, 1981), Cannon et al. (1981), Hunziker (1983, 1986), Labes (1985), Luke (1983) and Eligehausen et al. (1984).

With the increasing use of adhesive anchors in civil engineering, extensive work was conducted by Rehm et al (1988), Collin et al. (1989), Doerr et al. (1989), Eligehausen R. and Balogh T. (1995), Zavliaris et al. (1996) and Jennifer et al. (2001). Based on the worldwide database of test results from single or group adhesive anchors, Cook et al. (1998) summarized five typical failure modes of adhesive anchors and proposed different design procedures that could be used to analyze the capacity of adhesive anchors in concrete. In addition, parameters affecting the behaviour and strength of adhesive anchors were identified by Bergvall, B. and Johnson, A. (1975), Lang (1979), Hittenberger R. (1988), Rehm R. (1985), Collins et al. (1989) and Cook et al. (1998). However, limited experimental work is available regarding the behaviour and strength of adhesively bonded anchors in masonry and limestone.

2.2 Adhesive anchor system

The adhesive anchor consists of a chemical adhesive and a threaded rod, which is inserted in a drilled hole. The chemical adhesive is injected into the hole and it acts as a bonding agent between the anchor and the base material. Cook et al. (1998) shows the different types of available adhesive anchors as illustrated in Figure 2.1.

Typically, the hole diameter for adhesive anchor is about 20% larger than the diameter of threaded rod and the hole is drilled by a universal hammer drill or boring machine. Usually, the effective embedment length is about 10 times the diameter of the threaded rod. Two types of adhesive anchor systems currently exist, each with a slightly different installation procedure as described below:

(a) Capsule anchor system - It consists of a cylindrical glass capsule, which contains synthetic resin, accelerator and quartz aggregates. The capsule is inserted into a clean-drilled hole and mixed directly by rotation and hammering.

(b) Injection anchor system - Injection anchor is mainly used for anchorages in base material with a low compressive strength (lightweight concrete, aerated concrete) and in perforated brick or hollow block. A common injection anchor system consists of a plastic cartridge, filled with premeasured amounts of resin and hardener. The components are mixed during installation process by a special mixing nozzle. The resin is injected into a clean-drilled hole first, and then the threaded rod is inserted into the hole by hand. Although theoretically cementitious material can be used to bond the anchor to the base material, in practice, chemical adhesive are more commonly applied.

Figure 2.1: Types of adhesive anchor system (Cook et al., 1998)

2.3 Mechanism of load transfer in adhesive anchors

An adhesive is bonded to the threaded rod both by adhesion and the mechanical interlock in base material. Figure 2.2 shows that the tensile load is transferred through the adhesive to the base material along the entire embedded portion of the anchor. According to Daws (1978), the load transfer of an adhesive anchor depends on the following factors:

- (1) Mechanical interlock at the adhesive-base material interface
- (2) Chemical bond along the adhesive-base material interface
- (3) Mechanical interlock at the adhesive-anchor interface
- (4) Chemical bond along the adhesive-anchor interface

Anchor pullout failure may occur along either interface. Chemical bond failure may occur if the adhesive is improperly cured (Daws, 1978) or if the adhesive does not have adequate bond strength, while mechanical interlock failure may occur if the hole is not properly cleaned (Luke, 1985), or if the base material lacks adequate strength.

Figure 2.2: Adhesive anchor load transfer components (ITW Ramset/Red Head, 2002)

2.4 Variables affecting the behaviour of adhesive anchors

In addition to the strength of the anchor rod, the parameters that generally determine the load-carrying capacity of fastening system are:

- (1) Bond strength: Anchors set in epoxy or vinyl ester develop higher bond strength than those set in polyester resin (Lang, 1979). Anchors set in cementitious grouts have lower strength compared to those set in polymers resins.
- (2) Temperature effect and creep: All polymer resins are sensitive to high temperature. Cook et al. (1998) reported that the tensile load capacity of organic

resins under elevated temperature may decrease substantially while creep becomes much more important under normal temperature

- (3) Hole cleaning: The importance of hole cleaning on bond strength has been demonstrated by Collins (1989), who reported an average 30% decrease of bond strength for injection anchor M12 set in uncleaned holes as compared to the similar anchors set in holes that were blown out with compressed air and brushed with a stiff brush. Rehm et al. (1985) reported a maximum reduction of up to 80% in the case of holes drilled downwards with a large amount of the concrete material left in the hole.
- (4) Embedment length: Adnan Colak (2001) reported that the pullout strength of steel rods increased with increasing embedment length. It was noted that this increase was not linear and there was little increase in strength once a certain embedment length was reached. The nonlinearity implies that the concept of an average bond stress along the embedded length may not always be suitable.
- (5) Concrete strength: Cook et al. (1998) found that the majority of adhesive anchors exhibited a slight increase in bond strength when the concrete compressive strengths increased from 20 MPa to 40 MPa, but decreased slightly thereafter for strength up to 60 MPa. This implies that a 40 MPa concrete has adequate bond strength to prevent failure at the concrete-adhesive interface and that an increase in the concrete strength beyond this value is unnecessary. The observed slight decrease may be due to two reasons. First, the increase in concrete strength also increases its elastic modulus, which may affect the distribution of stresses at the concrete-adhesive interface as well as at the adhesive-anchor interface. This effect

may increase in shear stresses at the adhesive-anchor interface, causing a reduction in pullout strength. Alternatively, the strength reduction may be due to the natural variations in the properties of concrete, adhesive and their interfaces.

- (6) Moisture: If the drilled hole is flooded with water and not dried before the injection type anchor is set, the initial bond strength is markedly reduced. Mauthe et al. (1987) reported that in damp masonry a thin layer of moist material could be formed on the hole wall during drilling which was difficult to remove. Under worse circumstance, this layer works as a “lubricant”, which can reduce anchor capacity by as much as 60%.
- (7) Drilling method: Bergvall and Johnson (1975) showed that holes drilled by percussion tools (hammer drills) equipped with an effective dust removal device were rougher and differed from holes produced with core drills (diamond drilling). Therefore, adhesive anchors set in holes drilled with percussion tools have higher strength than those in holes produced by core drills. Collins (1989) compared the strength of injection-type anchors, M12 and M16, set in holes made with carbide bits, hammer drills and diamond core drills. Holes were cleaned in the same way with compressed air in each case. An average strength reduction of 7% was noted in the case of holes drilled with the diamond core drills.
- (8) Mechanical condition of the base material (cracked or uncracked): The load-bearing capacity of adhesive anchor is significantly reduced with increasing crack width in base material. Eligehausen and Ralph (1995) reported that the anchor strength in cracked concrete could only be one-third of its expected value in uncracked concrete.

Ammann W.J. (1991), Zavliaris et al. (1996), Lin et al. (1996), Adnan Colak (2001) and Cook et al. (2001) have reported that the following parameters also affect the behaviour and strength of adhesive-bonded anchors:

- (9) Strength and type of base material (concrete and masonry)
- (10) Type of loading (static, fatigue, shock or earthquake)
- (11) Nature of loading (tension, shear, combined tension and shear with or without bending of the fastening element)
- (12) Distance between adjacent fasteners or between fastener and free edge
- (13) Hole Size
- (14) Curing time of adhesive

It is clear that the development of any design method must properly consider the effect of these parameters. By the same token, testing of anchors must also reflect the role of each parameter before the results can be considered conclusively and generally.

2.5 Failure modes of adhesive anchors

The Comité Euro-International du Béton (CEB, 1994) document states that for injection anchors in masonry, the modes of failure are essentially the same as in concrete. Cook (1993) concluded that adhesive-bonded anchors in concrete and masonry typically display five failure modes.

1) Splitting of concrete and masonry (Figure 2.3): Zavliaris et al. (1996) showed that splitting failure occurred when the ratio of the anchor diameter to distance from the edge of the base material element was large.

2) Concrete and masonry breakage (Figure 2.4): A cone-like piece breaks away in concrete, solid brick or bricks with small cavities. As noted by Luke et al. (1985), this type of failure was observed in anchors with shallow embedment length.

3) Bond or pullout failure (Figure 2.5): This type of failure can occur in two ways, either by failure at the interface between the resin and the base material or by failure within the resin if it lacks adequate strength. Doerr et al. (1989) showed that this failure occurred when the top portion of the embedment length was debonded about 50mm; Cook et al. (1991) also produced this failure by performing tension tests.

4) Anchor steel failure (Figure 2.6): This failure is characterized by yielding and subsequent fracture of the steel rod. This occurs for larger anchor embedment and strong adhesive. The minimum embedment for steel failure load represents the basic installation length of the anchor.

5) Combined cone-bond failure (Figure 2.7): Luke et al. (1985) reported that in the case of anchors with diameter of 19 mm and for embedment between 50 and 100 mm, the commonly observed failure mode was the combined cone-bond failure mode, pullout with a shallow cone attached to the top of the anchor. Luke et al. (1985) also found that in this type of failure, bond failure occurred first, followed by cone failure. Subsequently, Cannon et al. (1981) suggested that a shallow cone formed before failure of the anchor. Collin et al. (1989) measured the displacement at the surface of the concrete adjacent to the anchor and at the embedded end of anchor and noted that the cone and bond failure occurred concurrently. Cook et al. (1993) also reported a combined cone and bond failure occurring together, as well as stating that the load-deformation response of the anchor was unreliable once bond failure occurred and concluded that the erratic behaviour of the

anchor after bond failure was due to mechanical interlock failure between the adhesive layer and the concrete interface.

Figure 2.3: Splitting failure

Figure 2.4: Cone failure

Figure 2.5: Pullout failure

Figure 2.6: Steel Failure

Figure 2.7: Combined Cone-Bond Failure

In all these failure modes, the strength of the adhesive anchor is influenced by the bond distribution along the anchor. Uniform, linear and nonlinear bond stress distributions have been suggested by Luke et al. (1985), Collin et al. (1989) and Doerr et al. (1989).

2.6 Load-displacement behaviour of adhesive anchors

Collins et al. (1989) investigated the load – displacement behaviour of adhesive anchors. Five-eighths inch diameter ASTM A193 Grade B7 threaded rods were installed in concrete with a specified compressive strength of 25 MPa (actual 31 MPa – 46.5 MPa). The embedment ranged from 178 mm to 305 mm and the anchors were allowed to cure for either seven or 24 days. The adhesive anchors were subjected to unconfined testing (the forces required to keep the concrete base fixed were kept away from the anchor). During the test, four failure modes were experienced: yield and fracture of the steel with anchor slip; yield and fracture of the steel without another slip; failure of the bond between the adhesive and the concrete or the failure of the bond between the adhesive and the steel. Their results showed that just before the peak load, the anchors apparently resisted the loading until they reached a maximum bond stress; beyond that point, the anchor and adhesive began to slip out as a unit. Little or no anchor slip was detected before bond failure. After bond failure, residual anchor strength was apparently due to the mechanical interlock between the adhesive and the concrete. Figure 2.8 shows some of their test results. Collins et al. (1989) also found that the failure of the bond between the adhesive and the concrete occurred when loading ranged from about one-third of the anchor steel capacity to the full capacity of the anchor steel.

Figure 2.8: Load-displacement diagram from Collin et al. (1989)

Doerr et al. (1989) researched the behaviour and the bond stress distribution of structural adhesive anchors. Data from their experiments was used to develop a design equation that, given certain characteristics of the anchoring system, can predict anchor capacity. Five-eighths inch diameter ASTM A193 Grade B7 threaded rods (with ultimate tensile strength of 690 - 862 MPa) were installed in 25 MPa concrete and subjected to unconfined testing. The anchors tested were either fully bonded (adhesive covered the entire embedded portion of the anchor) or partially bonded (top 50 mm of the embedded portion of the anchor was treated with a debonding agent) and the embedment ranged from 100 to 200 mm. Their test results showed that the fully bonded anchors failed either by fracture of the steel or by the formation of a shallow (about 25 to 50 mm) concrete cone, accompanied by pullout of the adhesive. The partially bonded anchors failed either

by rupture of the steel or by the pullout of an adhesive cone. Concrete cone failures did not occur with the partially bonded anchors. Test data also revealed that fully bonded anchors had only slightly higher capacities than the partially bonded anchors for the same embedment. Figure 2.9 shows some of their tension tests for 19 mm diameter anchors and 178 mm embedment that were either fully bonded or partially bonded (over the upper 50 mm) and subjected to both confined and unconfined testing. Later, Cook et al. (1991) verified these test results.

Figure 2.9: Fully and partially bonded tension tests (Doerr et al., 1989)

Eligehausen et al. (1984) conducted axial tension tests on adhesive-bonded anchors in uncracked concrete. The threaded rods ranged from M8 to M24 and the embedment was 9 times the nominal diameter of the anchor. Concrete compressive strength (cube strength) varied between approximate 15 MPa and 40 MPa. Their tests showed that the anchor failure modes were: pullout of the anchor from the hole without damage to the concrete; conical breakout of the concrete when anchors were closely spaced or when they were located close to an edge; splitting of the concrete or occurrence of a splitting crack extending to the edge or between adjacent anchors; rupture of the

anchor rod. Based on the assumption that the tensile strength of concrete is proportional to the square root of its cube compressive strength $\sqrt{f'_{cc}}$ and on the experimental results for concrete with $f'_{cc}=25$ MPa, the following expressions for calculating the anchor failure load were proposed:

$$P_u = 0.85\sqrt{f'_{cc}}h_{ef}^2 \quad (2.1)$$

$$\text{or} \quad P_u = 0.92\sqrt{f'_c}h_{ef}^2 \quad (2.2)$$

where P_u = Failure load without edge effect (N)

f'_{cc} = Concrete compressive strength measured on 200 mm cube (MPa)

f'_c = Concrete compressive strength measured on 150×300 mm cylinder (MPa)

h_{ef} = Effective depth of anchor (mm)

As part of their work Eligehausen et al. (1984) were able to evaluate the axial tension behaviour of individual anchors close to the edge of base material. The equation to the predict failure load as a function of edge distance was given by

$$P_{ue} = u_e P_u \quad (2.3)$$

$$\text{where} \quad u_e = \frac{e}{e_{critical}} \leq 1$$

e = Nearest distance from anchor to base material edge

$e_{critical}$ = Critical edge distance to prevent cracking of base material

P_{ue} = Failure load with edge effect

In order to be able to use Equation (2.3), one must first determine the critical edge distance. This quantity can be experimentally determined for a given anchor system and

each type of base material. Eligehausen and Ralgh (1995) compared the behavior of adhesive anchors subjected to tension loading in cracked and uncracked concrete. They found that the a crack in concrete would run around the anchor and destroy the bond between the base material and the bonding agent. Therefore, the initial part of the load-displacement curve of bonded anchors in cracked concrete was less stiffer than in uncracked concrete. Their pullout tests in cracked concrete also showed that the scatter of the failure load was large due to the irregular crack patten along the anchor depth and around the anchor perimeter.

Zavliaris et al. (1996) determined the load-displacement relationship for monotonically increasing load up to failure of adhesive anchors in concrete and the strain distribution in all the components of the anchor system (steel anchor and resin). Three types of anchors were investigated: threaded rod, ribbed and plain reinforcing bar, all 12 mm in diameter. The embedment length varied between 5 to 11.25 times the anchor diameter. Edge distance effect was also considered by placing two anchors at a distance of 75 mm from the edge of the block, but symmetrically displaced with respect to the center of the specimen. The bond agent was a very low viscosity epoxy resin with 2 mm thickness, but in some samples the thickness was increased to 4 mm, 8 mm and 10 mm. The concrete block in which the anchors were set had dimensions of $600 \times 600 \times 200$ mm with compressive strength, f'_{cc} , ranging between 19 MPa and 63 MPa. The holes in concrete blocks were made with an electrically driven percussion, except for one speciment, which was made by drilling with a diamond drilling bit. They used a Linear Voltage Displacement Transducer (LVDT) mounted on a special frame to continuously record the movements of the free part of the anchor in relation to the concrete surface.

Anchor strains were measured by means of three special strain gauges with 12 mm gauge length inserted along the anchor length. The concrete strains were measured by means of strain gauges glued onto precast mortar cubes ($30 \times 30 \times 30$ mm) of approximately the same strength, prepared with the same sand and the same type of cement. The observed failure modes of their tests (Zavliaris et al., 1996) were: failure of bond between the resin and the concrete and of the resin itself; splitting of base material; internal double cone cracking of the concrete; typical bond failure and pullout and resin failure at the bottom of the anchor. They drew the following conclusions from their tests: (1) The ultimate pullout load may be normalized with respect to the concrete strength by dividing by $f_{cc}^{0.36}$. In general, the normalized pullout load increased with both embedment and diameter. (2) The ultimate pullout load increased sharply with the thickness of the resin shell surrounding the anchor for thickness up to about 2mm. Further moderate increases in pullout load occur for thickness up to about 4 mm, but thereafter little further capacity developed.

Usami et al. (1980) investigated the behavior of epoxy bonded anchors under cyclic shear. They tested anchors with 20 mm diameter, hole diameter $d_0 = 40$ mm and embedment $h_{ef} = 10$ d. All anchors were embedded into concrete block ($f'_c = 20.5$ MPa) with a large edge distance. Loading was varied with either one loading and unloading or reversed loading. Two types of specimens were used (one shear plane and two shear planes). All anchors failed by rupture. Similar tests on capsule type bonded anchors M20 (polyester resin) were performed. Anchors with an embedment of $h_{ef} = 5$ d were pulled out, while anchors with embedment length $h_{ef} = 10$ d failed by rupture of the steel.

Collins et al. (1988) tested 12 specimens of different types of anchors including adhesive anchors under cyclic (fatigue) loading. The anchors were subjected to 1 million cycles at approximately 17 Hz, using a stress range of 48 MPa, just below the endurance limit of the anchor steel, and using a maximum stress of 60% of the minimum specified yield value. All anchors were embedded at depths previously found sufficient to produce steel failure (yielding and fracture of the anchor shank). The maximum load during cycling loading was about 30% to 40% of the concrete cone failure. The following conclusions can be drawn from their study: (1) High-cycle fatigue loading had no effect on anchor strength when anchors were embedded sufficiently to develop the full tensile capacity of anchor steel under static loads and the load amplitude was low enough to exclude a fatigue failure of the steel or a concrete cone failure. (2) Required embedment for all types of anchors in their study subjected to fatigue loads could be estimated using the same criteria as for anchors under static loads. (3) Fully embedded anchors, including adhesive anchors, could be expected to experience no significant reduction in stiffness and strength due to low level fatigue loading. (4) These results would also apply to anchors embedded insufficiently to develop the full tensile capacity of the anchor steel, if no fatigue failure of steel, concrete or load transfer mechanism occurred. However, a decrease of anchor stiffness during the fatigue loading may occur.

As part of the study by Collins et al. (1988) impact loads were also applied in tension to fully embedded anchor bolts of various types, including bonded anchors. Loads were applied to the anchors using a triangular pulse of approximately 0.25 seconds. Three pulses were applied to the anchor at a maximum load equal to 60% of the minimum specified yield load. If the anchor behaved satisfactorily, three more pulses were applied

up to a maximum load of 80% of specified yield loading, followed by a final set of 3 pulses at 100% of the specified yield loading. As in the static tests in uncracked concrete, all anchors failed by rupture of the steel. Their test conclusions regarding anchor behavior under impact loading were: (1) Impact loading to yield had no effect on anchor strength when anchors were embedded sufficiently to develop the full tensile capacity of anchor steel under static load. Required embedment lengths for all types of anchors in their study, subjected to impact loads, could be estimated using the same criteria as for anchors under static loads. (2) Fully embedded bonded anchors showed no reduction in secant stiffness up to the yield-level impact loads.

Hunziker (1983, 1986) reported results for bonded anchors inserted into cracked concrete and subjected to impact loading. Bonded anchors were M8 which had successfully passed tests under monotonic loading. These fasteners were installed in a concrete block ($f'_c = 25$ MPa) which was previously cracked in bending with crack width equal to 1.0 mm. Two impact loads one after another were applied on each fastener at an axial load equal to about the permissible service load under static loading. If the anchors survived these impact loadings, then two more impact loads were applied with an impact load equal to twice the admissible load. The behavior of the fastener under impact loading was considered satisfactory when the sum of the pullout displacement of the fastener after two impact loadings was not larger than 5.0 mm.

The CEB (1994) states that under cyclic (fatigue) loading an anchor can fail at a load level less than that would fail it under a monotonically increasing tensile load. The fatigue behaviour of connections depends on the fatigue behaviour of the anchor steel itself, on the fatigue behaviour of the load transfer mechanism between the anchor and

the base material and on the fatigue behavior of the concrete base. If anchors are sufficiently embedded to develop the tensile strength of the anchor steel, the fatigue strength of the anchor is independent of the load transfer mechanism and depends only on the fatigue strength of the anchor steel. The CEB (1994) also states the following regarding adhesive anchors failing by the formation of a concrete cone or by pullout under impact loading:

(1) The peak load and the displacement at peak load under impact loading are at least equal to or higher than under static loading

(2) If an anchor does not fail after one or several impulse loadings, the failure load measured in a subsequent static test is not much affected by the previous impact loading

(3) Bonded anchors may be pulled out when installed in wide cracks and loaded with a dynamic load with a magnitude equal to the allowable load for uncracked concrete

(4) Displacement should be taken into account when evaluating the strength of a fastening assembly under impact loading.

2.7 Code design method for chemically bonded anchor

2.7.1 Comité Euro-Internationale du Béton (CEB- Fastenings to Concrete and Masonry Structures, Euro-International Concrete Committee, 1994)

The CEB code adopts the Concrete Capacity Design (CCD) method proposed by Fuchs, Eligehausen and Breen (1995) for calculating the anchor tensile capacity. Under the tensile loading, the concrete capacity of a single anchor is calculated assuming an inclination angle between the failure surface and concrete surface as about 35 degrees.

For a single anchor in uncracked concrete, without edge influence or over-lapping failure volume, the maximum tensile load that can be resisted by the concrete surrounding the anchor is given by

$$N_u = k\sqrt{f'_c}h_{ef}^{1.5} \quad (2.4)$$

where N_u = Ultimate concrete tensile capacity (lb)

$k = 35$, for post-install fasteners

f'_c = Concrete compressive strength measured on 6 by 12 in. cylinders (psi)

h_{ef} = Effect embedment length of anchors (in.)

2.7.2 Japan Concrete Institute (JCI – State of the Art Report in Post Installed Anchors, 1994)

JCI states that the tensile resistance of adhesively bonded anchors is the smallest of the three values given by Equations (2.5), (2.6) and (2.7), which are based on the tensile yield strength of the anchor rod, the concrete cone strength and the ultimate bond strength of the anchor-concrete interfaces, respectively. It is noted that Equations (2.6) and (2.7) only apply to the concrete being the base material.

$$T_1 = f_y A_s \quad (2.5)$$

$$T_2 = 0.23\sqrt{f'_c} A_c \quad (2.6)$$

$$T_3 = \tau_{avg} dh_{ef} \pi \quad (2.7)$$

where T_1, T_2, T_3 = Tensile capacity of bonded anchor (N)

f_y = Nominal yield stress of anchor rod (MPa)

f'_c = Concrete compressive strength (MPa)

h_{ef} = Effective embedment length of anchor (mm)

d = Diameter of anchor (mm)

$\tau_{avg.} = 10\sqrt{\frac{f'_b}{21}}$ (MPa), average bond strength (mm^2)

A_c = Effective projected area of concrete cone failure

$$= h_{ef}\pi(h_{ef} + d)$$

The applicability of these equations to the anchors tested in the current study will be discussed later in the document.

2.8 Summary

This literature review reveals that the strength of adhesive anchors is governed by the properties of some key parameters. However, the values of the parameters depend on the particular anchor system. It is also evident from the review that most of the research was focused on anchors embedded in concrete, while anchors in masonry and stone received scant attention. The lack of adequate information on adhesive anchors in general and adhesive anchors in masonry and stone in particular is reflected by the absence of clear design guidelines for adhesive anchors in Canada and the US. It appears that only the CEB and Japan Concrete Institute have developed design provisions for adhesive anchors, but these rules are applicable only to anchors embedded in concrete only.

Chapter 3

Experimental Program

3.1 Introduction

The present experimental program involved testing 234 specimens to determine the load-displacement behaviour, mode of failure and the strength of adhesive-bonded anchors under different test conditions. All the tests were performed at the Civil Engineering Laboratory at Carleton University. In this chapter, the test parameters, test specimens and their construction, the loading system, instrumentation, data acquisition system, test procedure and results are described.

3.2 Test parameters

The test parameters studied in this investigation are summarized in Table 3.1. The five main parameters are described in column 2, and the range and types of variation are described in column 4. To eliminate statistical anomalies in each case, several repeat specimens were manufactured and tested, resulting in a total number of 234 specimens. The particulars of each specimen are given in the following section.

3.3 Specimen designation

For easy reference, each test specimen is identified using a mnemonic notation. According to the notation and scheme shown in Figure 3.1, each specimen designation consists of three basic parts BM-N-SA, where BM denotes the base material, N represents the specimen number and SA designates the anchor slope or inclination.

Table 3.1: Test parameters and their variation

Test Parameter No.	Test Parameter	Number of Variation	Detail
1	Base material	5	Hollow clay brick
			Hollow concrete block
			20.6 MPa concrete
			34.5 MPa concrete
			Limestone
2	Anchor rod diameter	2	6.4 mm or 9.5 mm
3	Anchor embedment length		
	a) Hollow clay brick	3	28 mm, 40 mm, 63 mm
	b) Hollow concrete block	4	35 mm, 49 mm, 55 mm, 77 mm
	c) Concrete	2	88 mm, 114 mm
	d) Limestone	2	88 mm, 114 mm
4	Edge distance		
	a) Concrete	2	40 mm, 60 mm
	b) Limestone	2	40 mm, 60 mm
5	Anchor slope	2	90° or 45°

Note: The number of tests includes repeat or duplicate specimens used for statistical purposes. The full details are given in Table 3.2.

Figure 3.1 Test designations

The complete details of all the test specimens are summarized in Table 3.2, which includes anchor diameter, anchor slope, embedment, base material, edge distance and number of tests. For example, BR-1,2,3,4,5-N designates the five (1,2,3,4,5) hollow clay brick specimens with vertical (90°) anchors and shallow embedment (28.6 mm).

Table 3.2: Specifications of each test specimen

Test Specimen Designation	Anchor Diameter (mm)	Embedment Length (mm)	Edge Distance (mm)
BR-1,2,3,4,5-N	6.4	28	30
BR-11,12,13,14,15-N	9.5	28	30
BR-1,2,3,4,5-45°	6.4	40	30
BR-21,22,23,24,25-45°	9.5	40	30
BRH-1,2,3,4,5-N	6.4	63	30
BRH-11,12,13,14,15-N	9.5	63	30
CMU-1,2,3,4,5-N	6.4	35	-
CMU-11,12,13,14,15-N	9.5	35	-
CMU-1,2,3,4,5-45°	6.4	49	-
CMU-11,12,13,14,15-45°	9.5	49	-
CMUE-1,2,3,4-N	6.4	55	-
CMUE-11,12,13,14-N	9.5	55	-
CMUE-1,2,3,4,5-45°	6.4	77	-
CMUE-11,12,13,14,15-45°	9.5	77	-
LS-1,2,3,4-N	6.4	88	-
LS-15,16,17,18-N	9.5	88	-
LS-21,22,23,24-N	6.4	114	-
LS-35,36,37,38-N	9.5	114	-
LS-1,2,3,4,-45°	6.4	88	-
LS-15,16,17,18-45°	9.5	88	-
LS-21,22,23,24-45°	6.4	114	-
LS-31,32,33,34-45°	9.5	114	-
LS-41,42,43-N	6.4	88	40
LS-44,45,46-N	6.4	88	60

Table 3.2 (Continued)

Test Specimen Designation	Anchor Diameter (mm)	Embedment Length (mm)	Edge Distance (mm)
LS-47,48,49-N	6.4	114	40
LS-50,51,52-N	6.4	114	60
LS-53,54,55-N	9.5	88	40
LS-56,57,58-N	9.5	88	60
LS-59,60,61-N	9.5	114	40
LS-62,63,64-N	9.5	114	60
CON2-1,2,3,4-N	6.4	88	-
CON2-5,6,7,8-N	6.4	114	-
CON2-9,10,11,12-N	9.5	88	-
CON2-13,14,15,16-N	9.5	114	-
CON2-1,2,3,4-45°	6.4	88	-
CON2-5,6,7,8-45°	6.4	114	-
CON2-9,10,11,12-45°	9.5	88	-
CON2-13,14,15,16-45°	9.5	114	-
CON2-17,18,19-N	6.4	88	40
CON2-20,21,22-N	6.4	88	60
CON2-23,24,25-N	6.4	114	40
CON2-26,27,28-N	6.4	114	60
CON2-29,30,31-N	9.5	88	40
CON2-32,33,34-N	9.5	88	60
CON2-35,36,37-N	9.5	114	40
CON2-38,39,40-N	9.5	114	60
CON3-1,2,3,4-N	6.4	88	-
CON3-5,6,7,8-N	6.4	114	-
CON3-9,10,11,12-N	9.5	88	-
CON3-13,14,15,16-N	9.5	114	-
CON3-1,2,3,4-45°	6.4	88	-
CON3-5,6,7,8-45°	6.4	114	-
CON3-9,10,11,12-45°	9.5	88	-
CON3-13,14,15,16-45°	9.5	114	-
CON3-17,18,19-N	6.4	88	40
CON3-20,21,22-N	6.4	88	60
CON3-23,24,25-N	6.4	114	40
CON3-26,27,28-N	6.4	114	60

Table 3.2 (Continued)

Test Specimen Designation	Anchor Diameter (mm)	Embedment Length (mm)	Edge Distance (mm)
CON3-29,30,31-N	9.5	88	40
CON3-32,33,34-N	9.5	88	60
CON3-35,36,37-N	9.5	114	40
CON3-38,39,40-N	9.5	114	60

Note: 1) BR – hollow clay brick (shallow embedment length)
 2) BRH- hollow clay brick (deep embedment length)
 2) CMU – hollow regular concrete block
 3) CMUE – hollow plain end concrete block
 4) CON2 – 20.6 MPa solid concrete block
 5) CON3 – 34.5 MPa solid concrete block
 6) LS – solid limestone block

3.4 Base materials

As mentioned earlier, five base materials were used for anchor embedment. In the case of concrete, a low strength and a medium strength concrete were utilized. The relevant properties of these materials are described in the following sections.

3.4.1 Hollow clay brick

Ontario 64.5 Brick per Square Meter (W60 × L213 × H102 mm) hollow core brick, with 38 mm core diameter, was used as base material in this series of tests. The geometry and dimensions of the brick are shown in Figure 3.2. From auxiliary tests performed according to ASTM C67 (ASTM 1990), the average compressive strength of hollow clay brick was found to be 38.5 MPa. The actual compressive strength values for the five test specimens are given in Table 3.3, which show reasonably consistent results.

The compressive strength of hollow clay brick was the only basic material property that was determined through auxiliary tests in this investigation.

Figure 3.2: Dimensions and geometry of hollow clay brick

Table 3.3: Compressive strength of hollow clay brick

Sample Number	Test Type	Strength (MPa)	Average Strength (MPa)
1	Compressive	35.8	38.5
2	Compressive	36.6	
3	Compressive	37.3	
4	Compressive	39.7	
5	Compressive	42.9	

3.4.2 Hollow concrete block

The hollow concrete blocks used in this study were either two core regular stretcher with frogged ends (Figure 3.3) or end units with one plain end (Figure 3.4). Auxiliary tests were performed according to the procedure described in ASTM C140 (ASTM 1980) to determine the compressive strength of concrete material of these blocks. Based on these tests, their average compressive strength was found to be 4.3 MPa. The

variations in the compressive strength from five specimens are given in Table 3.4. Generally speaking, the results appear to be relatively consistent, although the first specimen strength is somewhat lower than expected.

Figure 3.3: Dimensions and geometry of standard hollow side concrete block

Figure 3.4: Dimensions and geometry of breaker hollow edge concrete block

Table 3.4: Compressive strength of hollow concrete block

Sample Number	Test Type	Strength (MPa)	Average Strength (MPa)
1	Compressive	3.6	4.3
2	Compressive	4.1	
3	Compressive	4.3	
4	Compressive	4.6	
5	Compressive	4.9	

3.4.3 Solid concrete

All solid concrete specimens were cast using ready-mix concrete with specified cylinder compressive strengths (f'_c) equal to 20.6 MPa (3000 psi) and 34.5 MPa (5000 psi), which are representative of concrete strengths in existing buildings. Twelve sets of long cylinders were cast in each pour and tamped with a rebar to remove the air pockets, and then they were, troweled to a smooth finish and covered with polyethylene sheets to cure. The mixture proportions for each concrete mix are shown in Table 3.5. In addition to the long cylinders, in each pour, twelve standard control concrete cylinders (150 × 300 mm) were cast and cured under laboratory humidity and temperature conditions.

Table 3.5: Concrete mixture proportions

Concrete	Concrete mixture proportions						
	Cement (kg/m ³)	Coarse aggregate (kg/m ³)	Fine aggregate (kg/m ³)	Water (kg/m ³)	Admixture (ml/kg)	Air entraining additives (%)	Slump (mm)
20.6 MPa	224	1056	1005	132	6.25	2.5	80
34.5 MPa	398	1014	895	144	2.60	1.5	75

The specimens were demoulded 24 hours after casting and the specimens were cured under ambient conditions. Before anchor installation, each long concrete cylinder was cut into 300 mm long cylindrical test specimens.

To determine the compressive and tensile strength of the two concrete mixes, eight control cylinders (150 × 300 mm) of each mix were tested after 28 days; five in compression and three in splitting tension. From these tests, the characteristic cylinder compressive strength, f_c and split tensile strength, f_t were obtained as indicated in Tables 3.6 to 3.9. Subsequently, at the time of testing the anchors embedded in concrete, four cylinders of each specified concrete strength were tested to find their compressive strength at that age. In addition, using the displacement control test method, the full stress-strain relationship of each concrete cylinder in compression was recorded as shown in Figures 3.5 to 3.8. Though the softening portion of all the curves was not as well captured as the ascending part, this descending, or softening, branch of the stress-strain curve of concrete cylinders is not of particular significance in the context of this research. It is, however, important when failure is due to the failure of concrete in compression. In the present anchor tests, we do not anticipate that failure would be due to the crushing of concrete.

Results from these tests show that for two mixes, the average measured 28-day cylinder compressive strengths are roughly 21.1 MPa and 34.0 MPa, while their corresponding split cylinder tensile strengths are 2.4 MPa and 3.1 MPa. The measured compressive strength values are very close to their specified values. Meanwhile, the average cylinder compressive strengths of the two mixes at the time (approximate 10

months after pouring) of anchor testing increased to 21.7 MPa and 37.3 MPa, respectively, which shows a relatively small strength gain with aging.

Table 3.6: 28-day strength of concrete cylinders for the 20.6 MPa specified concrete

Cylinder Number	Test Type	Strength (MPa)	Average Strength (MPa)
1	Compressive	20.0	21.1
2	Compressive	20.2	
3	Compressive	20.3	
4	Compressive	22.0	
5	Compressive	23.0	
6	Splitting	2.3	2.4
7	Splitting	2.4	
8	Splitting	2.5	

Table 3.7: Strength of concrete cylinders for the 20.6 MPa specified mix at time of testing of relevant anchors

Cylinder Number	Test Type	Strength (MPa)	Average Strength (MPa)
1	Compressive	20.3	21.7
2	Compressive	21.8	
3	Compressive	22.4	
4	Compressive	22.4	

Table 3.8: 28-day strength of concrete cylinders for the 34.5 MPa specified concrete

Cylinder Number	Test Type	Test Results (MPa)	Average Strength (MPa)
1	Compressive	32.0	34.0
2	Compressive	34.2	
3	Compressive	34.2	
4	Compressive	34.3	
5	Compressive	35.3	
6	Splitting	3.1	3.1
7	Splitting	3.1	
8	Splitting	3.1	

Table 3.9: Strength of concrete cylinders for the 34.5 MPa specified mix at time of testing of relevant anchors

Cylinder Number	Test Type	Test Results (MPa)	Average Strength (MPa)
1	Compressive	42.1	37.3
2	Compressive	33.8	
3	Compressive	37.1	
4	Compressive	36.2

Figure 3.5: stress-strain relationship of concrete cylinders ($f'_c = 20.6$ MPa) at 28 Days

Figure 3.6: Stress-strain relationship of concrete cylinders ($f'_c = 20.6$ MPa) on anchor testing day

Figure 3.7: Stress-strain relationship of concrete cylinders ($f'_c = 34.5$ MPa) at 28 days

Figure 3.8: Stress-strain relationship of concrete cylinders ($f'_c = 34.5$ MPa) on anchor testing day

3.4.4 Limestone

Indiana limestone blocks (L305 × W305 × H230 mm) were obtained directly from the manufacturer and were used as base material. To determine the stone strength, cylindrical specimens for compressive strength testing were cored from the limestone. The samples had an averaged length of 100 mm and a diameter of 50 mm, resulting in a 2:1 length-to-diameter ratio that meets the ASTM C170 (ASTM 1987) requirement (Standard Test Method for Compressive Strength of Dimension Stone). Loading was applied perpendicular to the limestone bedding or rift. Using displacement control method, the loading rate was 0.0012 mm per second, which is the same as in concrete cylinder compressive strength tests. The stress-strain relationships of five limestone cylinders are presented in Figure 3.9. Table 3.10 shows the compressive and tensile strengths of each cylinder as well as the corresponding average strengths for all the cylinders. We observe relatively wide variability among the compressive strength for all the cylinders, ranging from a low value of 27.4 MPa to a high value of 39.9 MPa; with the average of the five being 32.5 MPa. On the contrary, there is significantly less variation among the tensile strength values and they are all practically equal. The average tensile strength is 7.1 MPa. It is interesting to note that the compressive to tensile strength ratio for limestone is smaller than the corresponding ratio for the concrete. Therefore, it is expected that anchor failure modes initiated by splitting of limestone would be less likely.

Table 3.10: Strength of limestone

Cylinder Number	Test Type	Strength (MPa)	Average Strength (MPa)
1	Compressive	29.2	32.5
2	Compressive	33.9	
3	Compressive	39.9	
4	Compressive	32.2	
5	Compressive	27.4	
6	Splitting	7.1	7.1
7	Splitting	7.1	
8	Splitting	7.2

Figure 3.9: Compressive stress-strain relationship of limestone cylinders

3.5 Anchor rod

Threaded rods meeting ASTM A-193 B7 (ASTM, 1990) specifications with either 6.4 mm nominal diameter or 9.5 mm nominal diameter were used for anchors in this study. Prior to placing the anchors in the base material, they were cut to the design length, using a band saw, were cleaned with a wire-brush, wiped and their sharp wedges were removed with a grinding wheel.

Subsequently, auxiliary tests were performed to determine the tensile and direct shear strengths of the anchor rods and their tensile stress-strain relationship. These results are summarized in Table 3.11 and Table 3.12, which give the yield stress and ultimate strength of the anchors in tension as well as their direct shear strength. The shear strength of the threaded rods was determined using the double shear test method as illustrated in Figure 3.12. During the shear testing, the rod was tightly inserted into the solid steel block and fixed in position by washers and heavy nuts on both sides. A vertical load was applied to the solid block that loaded the rod in shear. The results indicated that the 6.35 mm anchor has an average yield strength of 874 MPa, average ultimate tensile strength of 1138 MPa and an average shear strength of 607 MPa, while the corresponding values for the 9.53 mm anchor are 1030 MPa, 1177 MPa and 665 MPa, respectively. Here, the calculated strengths of anchors are based on the root (minor) diameter of the threaded rod (www.sdot.wa.gov). The stress values given in Table 3.11 and Table 3.12 are also based on the net cross-sectional area, which does not reflect the effect of the threaded portion of the cross-section.

Table 3.11: Direct tensile yield and ultimate strength of the anchor rods

Sample Number	Test Type	Diameter (mm)	Ultimate Tensile Capacity (N)	Average Ultimate Tensile Capacity (N)	Yield Strength (MPa)	Average Yield Strength (MPa)	Ultimate Strength (MPa)	Average Ultimate Strength (MPa)
1	Tensile	6.35	20712	19952	874	874	1181	1138
2	Tensile	6.35	20680		914		1179	
3	Tensile	6.35	20712		863		1181	
4	Tensile	6.35	20680		888		1179	
5	Tensile	6.35	17007		853		970	
6	Tensile	9.53	54282	51922	1090	1030	1231	1177
7	Tensile	9.53	50644		1003		1149	
8	Tensile	9.53	49004		995		1111	
9	Tensile	9.53	53854		1039		1221	
10	Tensile	9.53	51786		1020		1174	

Table 3.12: Direct shear strength of the anchor rods

Sample Number	Test Type	Diameter (mm)	Ultimate Shear Capacity (N)	Average Ultimate Shear Capacity (N)	Ultimate Strength (MPa)	Average Ultimate Strength (MPa)
1	Shear	6.35	9659	10642	551	607
2	Shear	6.35	9089		518	
3	Shear	6.35	11622		663	
4	Shear	6.35	12382		706	
5	Shear	6.35	10483		578	
6	Shear	9.53	22112	29317	501	665
7	Shear	9.53	28317		642	
8	Shear	9.53	32740		742	
9	Shear	9.53	31100		705	
10	Shear	9.53	32383		734	

In comparing the average tensile and shear strength of each anchor, we observe that the shear-to-tensile strength ratios are $607/1138 = 0.53$ and $665/1177 = 0.56$, for 6.4 mm and 9.5 mm rods, respectively. These values are larger than the theoretical values of 0.5, based on the shear to tensile yield stress ratio. The reason is that the shear stresses in Table 3.11 and Table 3.12 are based on an assumed uniformly distributed shear stress, which is not actually true, but this issue is not germane to the current discussion. In practices, the shear strength is assumed equal to half of the yield stress.

The stress-strain relations in Figures 3.10 and 3.11 show that the anchor rods have the typical characteristics of higher strength steel, without a well-defined yield point or a yield plateau. They experienced a maximum deformation of almost 10% at rupture. The yield stresses shown in Tables 3.10 and 3.11 were calculated using the 0.2% offset method.

Figure 3.10: Tensile stress-strain relationship for the 6.4 mm diameter threaded rod

Figure 3.11: Tensile stress-strain relationship for the 9.5 mm diameter threaded rod

Figure 3.12: Threaded rod double-shear-strength-test setup

3.6 Adhesive

Adhesive Epcon G5, meeting ASTM C881 (ASTM, 1990) requirement, manufactured by ITW Ramset/Red Head, was used for this series of tests. This adhesive product is a two-component ceramic-filled epoxy (Figure 3.13) contained in a two-chamber cartridge, one containing epoxy and the other one containing the hardener. The two components are blended in static mixer element within the nozzle (Figure 3.14). When properly blended the adhesive is light grey in color as dispensed using a hand-operated caulking-gun type applicator (Figure 3.15). For a typical application, the adhesive product is initially discharged on a paper towel and is not applied until a uniform color is confirmed. Detailed properties of this adhesive product as provided by the manufacturer are presented in Appendix A. According to the manufacturer, the Epcon G5 adhesive has a bond strength of 23 MPa (3330 psi) after 2 days, and a tensile strength of 30.1 MPa (4335 psi) after 7 days. Its compressive strength and compressive modulus after 7 days are 71.58 MPa (10344 psi) and 4.6 GPa (665.25 ksi), respectively. Some of the other chemical and mechanical properties of this adhesive are also given in Appendix A.

Figure 3.13: Adhesive G5 (Epcon)

Figure 3.14: Mixing nozzle

Figure 3.15: "Gun"- Applicator

3.7 Hole preparation

It is suggested by the adhesive anchor manufacturer that the optimum hole diameter for the particular anchor be 1.6 mm larger than the anchor rod diameter. Hole preparation consisted of the following steps:

- (1) Holes for adhesive anchors were drilled within a drill press (Figure 3.16) with an 8.0 mm carbide bit for 6.4 mm threaded rods and 11.1 mm carbide bit for the 9.5 mm threaded rods. The holes in the solid concrete block samples were drilled with a rotary hammer drill (Figure 3.17). During hole preparation, laboratory assistants carefully observed the drilling to ensure proper alignment and to maintain a vertical or 45-degree alignment. Hole depths were measured after cleaning the dust from the bottom of the hole.
- (2) After drilling, the dust was blown out from the holes commencing from the bottom.
- (3) Subsequently, the hole was cleaned with a nylon brush (Figure 3.18) to remove loose particles along its sides, and was vacuumed with an attachment long enough to reach the bottom of the hole.
- (4) Finally, the hole was blown out with compressed air with a plastic tube attachment that enabled the compressed air to clean the bottom and walls of the hole until dust leaving the holes was no longer noticeable.

Figure 3.16: Drill press

Figure 3.17: Rotary hammer drill

Figure 3.18: Nylon brush

3.8 Placement of anchors

All the anchors were installed according to the manufacturer's recommendations. The adhesive was placed from the bottom of the hole to the top with slowly moving the "gun" applicator up until the hole was 1/3 to 1/2 full. Care was taken to avoid trapped air bubbles. The anchor was then slowly pushed into the hole while rotating it to ensure maximum surface adhesion.

3.8.1 Placement of anchors in hollow brick and hollow concrete block

To prevent the unrestricted flow of injected adhesive into the hollow part of the hollow brick and hollow block masonry, a nylon screen (Figure 3.19) was used for the 9.5 mm anchors and a stainless steel screen (Figure 3.20) was used for the 6.4 mm anchors.

Figure 3.19: Nylon screen

Figure 3.20: Stainless steel screen

With reference to Figures 3.21 to 3.27, which show sample pictures of anchor configurations in hollow clay brick and hollow concrete block, the following installation procedure was used:

- (1) The nylon and stainless steel screens were cut to size, ensuring that they fit snugly into the holes.
- (2) The static mixing nozzle was inserted all the way inside the screen and the tube was completely filled with epoxy adhesive. The adhesive filled screen was then inserted into the hole.
- (3) The thread rod was inserted inside the screen by a slow twisting motion to ensure that maximum surface adhesion was achieved.
- (4) The excess adhesive was removed from the specimen surface.

Figure 3.21: Stainless steel screen, 6.4 mm diameter threaded rod with 28 mm vertical embedment in hollow clay brick

Figure 3.22: Nylon screen, 9.5 mm diameter threaded rod with 63 mm vertical embedment in hollow clay brick

Figure 3.23: Nylon screen, 9.5 mm threaded rod with 40 mm embedment 45-degree inclined anchor in hollow clay brick

Figure 3.24: Stainless steel screen, 6.4 mm diameter threaded rod with 35 mm vertical embedment in hollow regular concrete block

Figure 3.25: Nylon screen, 9.5 mm diameter threaded rod with 49 mm embedment and 45-degree inclined anchor in hollow regular concrete block

Figure 3.26: Stainless steel screen, 6.4 mm diameter threaded rod with 55 mm vertical embedment in hollow plain end concrete block

Figure 3.27: Stainless steel screen, 6.4 mm diameter threaded rod with 77 mm embedment and 45-degree inclined anchor in hollow plain end concrete block

Notice that the adhesive filled screens provide extra bond length for the anchor as long as the screens bearing against the base material can sustain the stress or the base material can sustain the stress due to the screens bearing against it.

3.8.2 Placement of anchors in solid concrete and limestone

Figure 3.28 shows some of the typical concrete test specimens after anchor installation. Additional sample pictures of concrete and limestone specimens are presented in Figures 3.29 and 3.30. The installation procedures of the anchors in solid concrete and limestone can be described as follow:

- (1) Holes were half filled with adhesive by inserting the dispenser nozzle to the bottom of the hole and slowly withdrawing it as the adhesive filled the hole.
- (2) Anchors were inserted immediately afterwards and slowly pushed to the bottom of the holes with a clockwise/counter clockwise twisting motion by hand, displacing the adhesive to the top of the hole.
- (3) The excess adhesive was wiped from the specimen surface.

Once the anchors were installed, they were left in the laboratory to cure for at least 72 hours before any pullout tests were performed.

Figure 3.28: Adhesive Anchors Being Installed by Author Using “Gun” – Applicator

Figure 3.29: Anchors in solid concrete samples

Figure 3.30: Anchors in limestone samples
(Note Uncut Concrete Samples in Background)

3.9 Test setup and procedure

The testing procedure for all the specimens was the same and all the tests were conducted in the Civil Engineering Laboratory at Carleton University.

3.9.1 Test Setup

The basic test setup is schematically shown in Figure 3.31. The test specimen was sandwiched between two steel plates. Figure 3.32 shows photograph of a typical specimen in the testing machine. The two plates were held together by means of four large threaded rods (25 mm diameter) to the moving platen of the test machine. The test anchor was attached to the load cell by means of an adaptor that allowed free rotation of the top of the anchor rod in all directions. The load cell was rigidly attached to the fixed top of the machine frame.

Note that in the case of the solid concrete and limestone specimens, the load was applied to the specimen via a steel annular plate placed between the top surface of the base material and the top steel plate. To avoid undesirable confinement of the base material, the distance from the anchor rod center to the inner radius of the annular plate was almost twelve times larger than the anchor diameter. Furthermore, the bearing area of the annular plate on the base material surface was designed to be sufficiently large to prevent bearing failure of the base material during loading.

All the tests were carried out using a 90 kN Instron Universal Testing Machine. Two different adaptors were used to connect the anchor and load cell, one for vertical anchors and the other for anchors inclined at 45°. The adaptors were screwed directly onto the threaded anchor rod and then connected to the load cell by a 20 mm

3.31: Schematic of loading and data acquisition systems for anchor pullout tests

Figure 3.32: Loading and data acquisition systems

diameter pin connection. The vertical pullout load was applied through a universal jack using displacement control to capture the post-peak load response of the anchor. The loading rate was relatively slow and was within the static regime.

Anchor slip was measured using a Linear Voltage Differential Transformer (LVDT). The LVDT was mounted on a steel strip that was firmly attached to the anchor as schematically indicated in Figure 3.31. The LVDT measured the anchor rod elongation along its unembedded length and any slip due to pullout. It must be pointed out that in most cases the unembedded length of the anchor rod was quite small and therefore any

large deformations measured during the test were primarily due to pullout. Figures 3.33 and 3.34 show the actual LVDT.

3.9.2 Instrumentation

A load cell with maximum capacity of 90 kN was used to measure the applied load on the test anchor. As shown in Figure 3.31, the load cell was placed between the adaptor and the loaded end of the anchor. The anchor displacement was also measured and the measurement scheme depended on the specimen geometry. For the hollow brick and concrete masonry blocks, the LVDT was placed directly beneath the free-end of the anchor as indicated in Figure 3.35. Figure 3.36 shows an actual hollow clay brick in the test machine and the LVDT stroke. In the case of the solid concrete and limestone specimens, as indicated earlier, the LVDT was placed on a steel strip attached to the anchor at a location very close to the top of anchor rod.

The LVDT had a 25.4 mm stroke. The displacement measurements in the solid concrete and limestone specimens give the total movement of the loaded end of anchor, including the anchor axial deformation and the anchor slip in the base material. The displacements in the hollow clay and concrete blocks give the relative movement between the anchor and base material, which is actually the slip.

The data acquisition system that was used is shown in Figures 3.31 and 3.32, where a desktop computer is connected to a data acquisition system. Voltage outputs from the load cell and the LVDT were sent through analog-to-digital processors, stored on "floppy" disks, converted to engineering units and plotted on an X-Y diagram format (load-displacement recording) on the computer screen.

Figure 3.33: Location of LVDT for vertical anchors

Figure 3.34: Location of LVDT for anchors at 45-degrees

Figure 3.35: Schematic drawing of LVDT location in hollow clay brick

Figure 3.36: Location of LVDT in hollow clay brick

3.9.3 Test procedure

Each specimen was placed in the testing frame, which essentially consisted of top and bottom steel plates and four large threaded rods. After centering the specimen, the load was applied gradually. Each specimen was loaded to failure under displacement control. The loading rate was set at 0.508 mm per minute. Load and displacement readings were taken automatically from start to finish, and measurements were taken at a rate of one reading per second. Once the load dropped dramatically, the test was stopped. The collected test data and results are presented in the following chapter.

Chapter 4

Test Results and Observations

4.1 Introduction

In this chapter, the failure mode and load-displacement curve for each specimen are discussed. In addition, factors affecting the behaviour and strength of adhesive anchor are discussed in detail, while other observations are discussed in brief...

Slip, in this discussion, is defined as the relative displacement between the top surface of the base material and the adjoining anchor rod section, or as the displacement of the unloaded end of the anchor. The former applies to anchors in solid concrete and limestone, while the latter applies to anchors in hollow concrete block and hollow clay brick. Anchor stiffness is defined as the initial slope of the anchor load-displacement curve. The effective embedment was measured from the surface of base material to the embedded end of the anchor rod along its centerline.

The complete data and the load-displacement curves of all the 234 test specimens are given in Appendixes B to G. Appendix B reports the following properties for each test specimen: anchor diameter, edge distance, effective anchor embedment, peak load, slip at peak load, ultimate load, ultimate slip, failure mode, average peak load, standard deviation of the average peak load, average slip at peak load and the standard deviation of the average slip at peak load. The average and standard deviation are given for results obtained from repeated tests. Appendixes C to G show the axial load-displacement curves of all the test specimens. The curves are categorized in terms of the base material; thus

they are separated into anchors in hollow clay brick (Appendix C), in hollow concrete block (Appendix D), in 20.6 MPa concrete cylinders (Appendix E), in 34.5 MPa concrete cylinders (Appendix F), and in limestone (Appendix G). It will be noticed that some of the load-displacement curves are rather unsmooth and jagged. The reason is the slip-stick behaviour of some anchors, which produces this type of response. Some of the non-smoothness is due to specimen seating and so-called shake down. However, despite these complexities, the quantities reported in the table in Appendix B can be reliably obtained from these figures. In particular, the maximum or peak anchor load and the maximum deformations are easy to read from these curves.

4.2 Load-displacement behaviour and failure mode

As mentioned previously in Chapter 3, an LVDT was used to measure the displacement of the anchor. In the case of anchors in solid concrete and limestone, the measured displacement is due to the extension of the unembedded portion of the anchor and the slip in the embedded portion. However, the elastic extension of the unembedded portion is generally quite small relative to any slip; therefore, unless the anchor yielded, it was assumed that the measured displacement was primarily due to slip. In the case of anchors in hollow concrete block and hollow clay brick, the displacement was measured at the free end of the anchor; therefore, it represents the true slip.

The results presented are based on the observed failure mode. Since in practice, the design would be based on the acceptable failure mode, it would be appropriate to present the results and to develop design equations based on this criterion.

4.2.1 Anchor rod failure

Anchor rod failure occurred in hollow concrete block, concrete and limestone test specimens. Typical load-displacement curves for this failure mode are presented in Figures 4.1 and 4.2. Some vertical anchors behaved nearly linearly up to failure, as shown in Figure 4.1 (a), and they fractured in a sudden and brittle manner; while others experienced considerable yielding prior to failure as shown in Figure 4.1(b).

As stated, the load-displacement curves of vertical anchors failing due to the rupture of the anchor rod exhibit two distinct shapes as in Figures 4.1(a) and (b). Figure 4.1(a) represents the behaviour of those anchors that experienced negligible pullout or loss of bond between the substrate and the anchor. Accordingly, the maximum displacement is quite small and significant change in slope occurred only at yielding of the anchor, followed closely by its rupture. Notice that the maximum displacement in Figure 4.1(a) is only 1.2 mm. On the other hand, Figure 4.1 (b) represents the behaviour of vertical anchors that experienced substantial slip, but did not pullout before the anchor ruptured. In this case, the maximum anchor displacement is primarily due to slip and is generally large, with the anchor experienced more than 10 mm displacement before rupture. The plateau in this figure actually represents the combined post-yield extension and slip of the rod. From the practical point of view, the combined yielding and pullout mode would be a very desirable failure mode under blast and other dynamic loads because the area under the load-displacement curve represents the energy absorption capacity of the anchor system. If we compare the area under the curves in Figure 4.1(a) and (b), we observe that the anchor in (a) absorbed approximately 26 kN.mm energy while the one in (b) absorbed approximately 155 kN.mm, roughly six times that in (a).

(a)

(b)

Figure 4.1: Typical load-displacement behaviour for vertically installed anchors exhibiting anchor failure

On the other hand, the anchor in (a) resisted a total force of almost 40 kN, while the one in (b) resisted only 16 kN. Hence if the anchor response were governed by pressure rather than impulse, anchor (a) would resist a higher blast force before failure while if the behaviour was governed by impulse, anchor (b) would resist a higher impulse before failure.

For inclined anchors, slippage occurred early and their behaviour is characterized by a pseudo-yield response as in Figure 4.2. In fact, the dramatic change in the slope of the curve in Figure 4.2 at an applied axial load of approximately 3 kN is due to slip at the anchor-concrete interface. From this load onward most of the measured displacement is due to slip, but despite the relatively large slip, the anchor did not completely pullout before rupture. It must be stated that the inclined anchors in the current tests were subjected to combined axial, shear and bending loads; therefore, they failed at relatively lower loads compared to the vertical applied anchors that were subjected to pure axial load. Notice that whenever failure is due to yielding and fracture of anchor, the failure load is relatively higher than that in the other failure modes. As shown in Figure 4.3, the steel usually fractured in the region outside the block and at a section very close to the block surface.

Three anchor specimens (CMU-1-45°, CMU-2-45° and CMUE-1-45°) that were embedded in hollow concrete blocks failed by steel fracture. These anchor specimens were 6.4 mm in diameter and were installed at 45-degree inclination. Their average failure load was approximately 10.7 kN.

Figure 4.2: Typical load-displacement behaviour for 45-degree installed anchors exhibiting anchor failure

Figure 4.3: Typical anchor failure

Steel fracture was a common failure mode for the 6.4 mm diameter anchors embedded at 45-degree inclination in concrete and their average failure load was approximately 6.7 kN compared to 18.5 kN for similar vertical installed anchors. Test results showed that concrete strength (i.e. 20.6 MPa versus 34.5 MPa), did not have a significant effect on the failure load of the 6.35 mm diameter anchors which failed due to steel fracture. It seems that due to adequate concrete strength and embedment depth, the pullout resistance only depended on the anchor property. On the other hand, failure due to steel fracture in the 9.5 mm diameter anchors set in concrete was only observed in the case of inclined anchors or in the case of vertical anchors with deep embedment. The average failure load was 48.9 kN for vertical anchors and 25.1 kN for inclined anchors.

For anchors set in limestone, whenever failure occurred due to rupture of steel, there was no obvious difference in failure load between these anchors and those embedded in concrete. This signifies that both materials had adequate strength to fully develop the strength of the anchor rod.

4.2.2 Anchor pullout (bond) failure

Anchor pullout failure occurred in hollow brick, hollow concrete block, concrete and limestone. This failure mode was due to insufficient strength along the embedded length of the anchor. Figures 4.4 shows typical load-displacement curves for vertically installed anchors that failed by pullout. The curves in Figures 4.4 (a) and (b) represent the typical characteristic behaviour of anchors experiencing pullout failure. The first part of these curves (i.e. from zero to the peak or maximum load) generally indicates small or negligible slip while the second part after the peak load exhibits significant pullout. In

(a)

(b)

Figure 4.4: Typical load-displacement curves for vertically installed anchors exhibiting anchor pullout failure

Figures 4.4(a), the sudden drop after the peak load indicates a sudden pullout, followed by the resistance due to friction or mechanical interactions between the anchor threads and the base material. Figures 4.4(b), on the other hand, exhibits a gradual loss of bond after the peak. In both cases the pullout mechanism dissipates a great amount of energy. Note that the energy absorbed before the peak is relatively small compared to that absorbed after peak load. From the energy absorption standpoint, the behaviour of the anchors represented by Figure 4.4(a) is superior to those which behaviour is represented by Figure 4.4(b). Test specimens LS-44-N and LS-46-N are examples of specimens that exhibited the behaviour indicated in Figures 4.4(a) while specimens CON2-4-N and CON3-1-N are examples of specimens showing the behaviour indicated in Figures 4.4(b). It should be pointed out that the bond between the anchor and the base material is due to a combination of adhesion and mechanical interlock with the threaded surface of the anchor. Generally speaking, up to the peak load both bond resistance mechanisms are effective, while after the peak load, the adhesion is generally lost and only the mechanical bond remains effective.

The inclined anchors in hollow clay brick suffered bond loss under relatively small loads but continued to resist increased load, albeit at the expense of large displacements. After the peak, the load dropped suddenly but it started to increase again and then dropped again and continued to drop at a fast rate. The first drop in the load was due to cracking of the brick, which produced a “pop” sound as it occurred. Figure 4.5 shows this typical load-displacement curve for specimen LS-4-45°. It should be pointed out that the slip-stick type of response in Figure 4.5 is typical of anchors that have lost adhesion but are transferring load by mechanical interlock.

Figure 4.5: Typical load-displacement curve for 45-degree installed anchors
exhibiting anchor pullout failure

Two brick specimens (Br-11-N and Br-15-N) failed by anchor pullout. These anchors were 9.5 mm diameter and vertically installed with 28 mm embedment. Figure 4.6 shows that with these anchors the nylon screen lost the bond with the surrounding brick material and consequently the screen pulled out with the anchor still bonded to it. This means that the weak link in the anchors that failed in this manner was the bond between the nylon screen and the brick. After the loss of the bond, the nylon tube transferred the load by bearing against the bottom of the brick flange. This eventually cracked the brick thereby dropping the load. It would appear that even after the brick cracked, the anchor continued to carry some load, but as the cracks widened, the nylon tube easily pullout. The average failure load for these two specimens was approximately 6.5 kN.

Figure 4.6: Typical anchor pullout failure in hollow brick

In hollow concrete block tests, this failure mode only occurred with the 9.5 mm diameter anchors, where the pullout again included the anchor and the nylon screen as shown in Figure 4.7. The nylon screen was seriously damaged after the pullout failure. However, one of the specimens (CMUE-12-N) exhibiting this failure mode had not cured well and the epoxy inside the screen was still soft, resulting in a lower pullout load (4.1 kN). This clearly indicates that curing of the epoxy has a very important effect on an adhesive anchor behaviour and strength.

Figure 4.7: Typical anchor pullout failure in hollow concrete block

Figures 4.8 and 4.9 show the anchors exhibiting pullout failure in concrete and limestone, respectively. The failure load of these anchors depended on the anchor system. Typically, anchors with larger diameter, large embedment length and vertical installation had higher bond strength, consequently they sustained much larger pullout load. During the tests in concrete and limestone, some specimens (e.g. CON2-15-45°, LS-4-45°, LS-24-45°) experienced bond failure at the anchor-adhesive interface. Meanwhile, other specimens failed at the adhesive-concrete or adhesive-limestone interface, and some crushed or spalled base material around the pulled out anchor.

Figure 4.8: Typical anchor pullout failure in concrete (CON3-9-45°)

Figure 4.9: Typical anchor pullout failure in limestone (LS-4-45°)

4.2.3 Combined cone-bond failure

Combined cone-bond failure occurred in hollow clay brick, hollow concrete block, concrete and limestone specimens. This failure mechanism is characterized by pullout of an adhesive anchor plug with a base material shallow cone attached to it. A typical load-displacement curve for this type of failure mode is presented in Figure 4.10. Before reaching the maximum load, the anchor experienced little pullout and experienced relatively high load. After the cone formation, the attached cone began to slip out while the loading concurrently began to gradually decrease to zero. The residual strength was due to mechanical interlock between the adhesive and the base material. For this failure mode, the pullout resistance depends mainly on the tensile strength of base material and the anchor embedment length.

Figure 4.10: Typical load-displacement curve for anchors exhibiting combined cone-bond failure

It is well known that the axial load or stress in the embedded portion of the anchor is accompanied by radial normal and longitudinal shear stresses at the anchor-epoxy and epoxy-base material interfaces. The radial stresses in turn produce circumferential tensile stresses in the base material. Therefore, the base material may either fail due to diagonal tension or it may experience radial cracks. The cone failure is due to diagonal tension. In the next chapter, this issue will be further explained and discussed.

It should be pointed out that the behaviour represented by Figure 4.10 is desirable because it represents the behaviour of an anchor that can resist both high load and can absorb substantial energy before failure. In the context of blast loads, this means that whether the response of the anchor is governed by impulse or pressure, it is capable of offering substantial resistance before failure.

In hollow clay brick, combined cone-bond failure was observed in two specimens (Br-1-N and Br-3-N), both 6.4 mm anchor, 28 mm embedment and vertical installation. Their pullout cones were 25 mm and 30 mm diameter at the base; and 15 mm and 20 mm thick, respectively. The failure of specimen Br-3-N, clearly showing the attached failure cone, is shown in Figure 4.11. The failure loads of these specimens were 4.9 kN and 3.4 kN, respectively. During the test, the cracking sound was heard at peak load.

The pullout cone in hollow concrete block was between 20 mm to 65 mm in diameter and 10 mm to 30 mm in thickness as shown in Figures 4.12 and 4.13. This failure mode only occurred for the anchors installed vertically and again, loud cracking sound could be heard at peak load.

Figure 4.11: Typical pullout with cone and rod failure in hollow brick

Figure 4.12: Typical combined cone-bond failure in hollow concrete block

Figure 4.13: Close view of pullout cone shape for anchor specimen CMU-15-N

Pullout concrete cone failure was observed for those 9.5 mm anchors which were vertically installed at the center of the block with an embedment length of 88 mm. For the anchors embedded in 20.6 MPa concrete, the pullout cone size was between 120 mm and 200 mm in diameter, between 15 mm and 40 mm in thickness and the average pullout loading was 32.1 kN (see Figure 4.14). In the 34.5 MPa concrete, the pullout cone was 50 mm to 100 mm in diameter, 10 mm to 20 mm in thickness and the average pullout load was 39.5 kN. It was observed that the concrete cone size and the failure load depended on the concrete compressive strength. The higher the concrete compressive strength, the smaller the cone size and the higher the pullout resistance. Therefore, the concrete strength has an important effect on the behaviour and strength of adhesive anchors.

In limestone, combined cone-bond failure was only observed for anchors installed near the limestone block edge. These specimens with the 9.5 mm anchor and 88 mm embedment resulted in pullout cone sizes of 40 mm to 70 mm in diameter and 10 mm to 20 mm in thickness as shown in Figure 4.15. The failure load was between 34.5 kN and

45.7 kN. It may be recalled that limestone has a much higher tensile strength than even the higher strength concrete used in the present study. Therefore, failure due to diagonal tension is less likely.

Figure 4.14: Typical combined cone-bond failure in concrete (CON2-10-N)

Figure: 4.15: Typical combined cone-bond failure in limestone

4.2.4 Cracking and splitting of base material

Failure by cracking and splitting of base material occurred in hollow clay brick, hollow concrete block, concrete and limestone specimens. This failure mode is characterized by a planar cracked extending along the anchor and from the anchor to the free edge of the base material. Two different but typical load-displacement curves for this failure mode are shown in Figure 4.16. In Figure 4.16 (a), initially the relationship (slope) between the applied load and anchor displacement is linear up to the peak load. After that, the load decreases gradually due to the formation of numerous small cracks that were observed during the test. The residual strength of the anchor is due to the mechanical interlock between the adhesive and the base material. Some anchors experienced a reduction in stiffness upon approaching the peak load. In the latter case, the loss of stiffness was sudden, as shown in Figure 4.16 (b). Usually, large and long cracks were observed when this failure mode occurred.

Most anchors set in hollow clay brick exhibited the cracking and splitting failure due to the relatively short distance of the anchor rod from the brick edge. This problem cannot be avoided because of the relatively small width of a brick. As shown in Figure 4.17 (a), a single crack goes through the brick flange and propagates laterally from the anchor interface to the brick body. Figure 4.17 (b) shows another splitting mode in hollow brick. Only one anchor specimen (Br-4-N) experienced this failure mode. This was a 6.4 mm anchor with 28 mm embedment and installed vertically. Its failure load was 3.5 kN. Figure 4.17 (b) clearly shows two cracks at nearly 45-degree inclination run along the anchor embedment and form a wedge. The brick is also broken into two parts at its lower flange. The pulled out

(a)

(b)

Figure 4.16: Typical load-displacement curves for anchors due to cracking and splitting of base material

(a)

(b)

Figure 4.17: Typical cracking and splitting failure of hollow clay bricks

part of the steel screen was peeled apart and no adhesive was found to be attached to the anchor between the threads, indicating that the bond between the steel and the adhesive had failed. This type of failure appears to be similar to the punching shear failure in concrete slabs.

It is very important to point out that the failure mode in Figure 4.17 (a) is due to tensile stresses while that in Figure 4.17 (b) is due to shear stresses, resulting in high diagonal tension on nearly 45° planes. Hence in practice one must consider both failure modes in designing.

In hollow concrete block, splitting lines were observed on the top surface as shown in Figure 4.18. These crack lines went through the block upper flange and extended to its edge laterally. As the cracks fully propagated at peak load, cracking sounds could be heard during the test. Since the failure loads for nominally identical specimens had similar magnitudes, this failure mode appears to be rather predictable.

Figure 4.18: Typical cracking and splitting failure in hollow concrete block (CMU-11-N)

There were three different cracking and splitting failure modes in concrete. One was the same as in hollow concrete block (i.e. crack lines propagated along the threaded rod and extended towards the edge of the block). This kind of failure mostly occurred for anchors installed at 45-degree as shown in Figure 4.19 (a). The concrete cylinder completely split into several parts. At the same time, small pieces of concrete material were found around the anchor, indicating bond failure at the adhesive-concrete interface around the anchor. Another cracking and splitting failure shown in Figure 4.19 (b) indicates that crack lines first appeared on the top and side faces of the base material, and extended from the top to the bottom along the anchor embedment. Finally at peak load, a large and long crack line formed in addition to numerous hairline cracks on the entire concrete surface. These cracks were caused by the high tension load in the concrete. Usually, anchor specimens (e.g. CON2-34-N, CON3-36-N) in which the anchor was installed close to the free edge experienced this failure mode.

Another cracking and splitting failure mode is shown in Figure 4.19 (c). However, only one anchor sample (CON3-12-N) failed this way. This is a failure mode caused by circumferential tensile stresses and is often observed when a good bond exists between the centrally installed anchor and the concrete. As we will see later, this specimen was subjected to very high bond stresses. Its failure load was 44.3 kN and its maximum slip was only 0.7 mm – relatively smaller compared to the other anchors. The cracks formed suddenly at maximum load and spread along the anchor. The concrete specimen was broken into four parts by 70 mm deep crack lines. Each broken part of concrete was easily separated since it was not bonded to the anchor, indicating that the bond between

the adhesive and concrete had completely failed. The bond loss is probably a consequence of the splitting and not vice versa.

(a) CON3-10-45°

(b) CON2-32-N

Figure 4.19 (a, b): cracking and splitting failure in concrete specimens

(c) CON3-12-N

Figure 4.19: Typical cracking and splitting failure in concrete specimens

Cracking and splitting failure in limestone was observed for two anchor specimens (LS-53-N and LS-56-N). These anchors had 9.5 mm diameter, 88 mm embedment length and were installed at 40 mm or 60 mm from the edge of the block. Their failure loads were 45.1 kN and 40.6 kN, respectively. Figure 4.20 shows the crack lines on the limestone surface. These cracks ran vertically along the anchor rod, but they did not go through the block completely. These splitting cracks were due to the propagation of the initial vertical crack in the limestone adjacent to the anchor.

Figure 4.20: Typical splitting failure in limestone

4.2.5 Breakout cone failure

Breakout cone failure was observed in anchors installed off center and near the free edges of the concrete and limestone blocks. This failure mechanism is characterized by a breakout cone radiating from the centerline of anchor to the free edge of the base material. Obata et al. (1998) stated that the breakout cone failure could be explained by the proximity of a free edge that affects the ultimate behaviour of the anchor in two different ways. First, there is a decrease in the lateral pressure acting on the anchor rod, and second, and more importantly, the cutout or breakout mass causes a reduction on the adjacent area that resists the tensile stresses.

Figure 4.21 shows the typical load-displacement curve of breakout cone failure from this research work. According to experimental observations, the critical cracks, which eventually led to the breakout cone, initiated when only 30-40% of the critical load was applied, then they propagated toward each other in a stable way as loading increased.

The initial cracks formed at the anchor and were essentially parallel to the central cross-sectional axes of the block. After loading reached a critical level (i.e. the pullout strength), the cracks unstably propagated vertically and towards the edge and finally formed a breakout cone.

Figure 4.21: Typical load-displacement curve for anchors exhibiting breakout cone failure

For most anchor specimens set in concrete, the breakout cone had a wedge shape as shown in Figure 4.22. The size of the wedge varied with anchor diameter, embedment, edge distance and concrete compressive strength. Generally, the concrete breakout cone depth was equal to the anchor embedment and the wedge base width was equal to the anchor edge distance. Typically the testing resulted in the concrete breakout wedge and the adhesive anchor to completely separate.

Figure 4.22: Typical breakout cone failure in concrete (CON3-36-N)

In limestone, breakout cone failure was only observed for one anchor specimen (LS-55-N). Here the 9.5 mm anchor embedded to 88 mm and at 40 mm edge distance, failed at a load of 45.1 kN. The breakout mass had an irregular shape as shown in Figure 4.23. Crack lines appeared at the anchor and propagated towards the two perpendicular sides of the block in an unsymmetrical fashion. Concurrently, the cracks penetrated through the thickness and eventually a portion of the stone separated at one corner from the body of the block. This failure mode was different from that observed in concrete where the failed or separated portion formed a wedge that was symmetrical with respect to the anchor position. The unsymmetrical or corner failure in the case of the limestone block may be due to the anisotropic properties of the stone. Hence, in practice it may be important to ascertain the position of the anchor with respect to the planes of material anisotropy in natural stone.

Figure 4.23: Typical breakout cone failure in limestone (LS-55-N)

4.2.6 Brick web failure

Brick web failure, as shown in Figure 4.24, was observed in two anchor specimens (Br-23-45° and Br-26-45°). Figure 4.25 shows the typical load-displacement curve of this failure mode, where 9.5 mm anchors with 40 mm embedment were installed at 45-degree inclination. A crack usually started on one of the brick webs on either side of the anchor then propagated to adjacent webs as loading increased. Finally, the brick specimen broke into two halves by a horizontal crack line traversing the webs. However, the anchor and plastic screen remained glued together inside the brick, suffering no obvious damage and sustaining the residual pullout strength. This observation is confirmed by Figure 4.25 where the pullout load dropped sharply after the peak load, but then increased gradually, reaching almost 80% of the peak load. Again, the area under the curve shows a reasonable level of energy

Figure 4.24: Brick web failure (Br-26-45°)

Br-23-45°
Diameter=9.5 mm Depth=40 mm Slope=45° Failure: Brick Web

Figure 4.25: Typical load-displacement curve for anchors exhibiting brick web failure

absorption, although both the ultimate strength and total energy absorption are significantly lower than those of similar anchors embedded in concrete or limestone. The

consequence is that more anchors would be required to secure a non-structural member (e.g. a window frame) to a clay brick masonry wall.

4.3 Discussion of anchor test variables

To facilitate the comparison of the test results based on different anchor parameters, test results are presented and discussed in terms of anchor average slip at peak load, average slip at failure, average peak load and the normalized peak load value. The term normalized peak load value refers to the ratio of average peak load to anchor rod tensile strength from the tension coupon tests. Table 4.1 summarizes these values for all the 234 test specimens. More detailed results for each specimen are presented in Appendix B.

The benefit of expressing the anchor strength in terms of the anchor rod tensile strength is that one can immediately determine the efficiency of the anchor system under consideration. The slip values corresponding to the peak load give an easy indication of the amount of slip in the anchor before it achieved its peak load while the ultimate slip, when compared to the slip at peak load, indicates the amount of energy that the anchor could absorb before complete failure. For dynamic loads acting on an anchor, its energy absorption is extremely important.

The data and results in Table 4.1 pertain to all the tested specimens. From the specimen designation, most of the pertinent anchor properties can be easily gleaned, and it is possible to ascertain the effect of certain parameters on the efficiency of an anchor system. A value between 0.9 and 1.0 or greater than one in the last column of the table indicates a highly effective anchor system while smaller values less than 0.9 indicate a reduction in the system efficiency.

Table 4.1: Summary of anchors test results

Test Specimen	Anchor Diameter (mm)	Embedment (mm)	Average Peak Load (kN)	Average Slip at Peak Load (mm)	Average Slip at Failure (mm)	Normalized Peak Load
BR-1,2,3,4,5-N	6.4	28	4.8	0.5	2.4	0.24
BR-11,12,13,14,15-N	9.5	28	6.6	0.4	2.9	0.13
BR-1,2,3,4,5-45°	6.4	28	3.8	2.4	2.3	0.19
BR-21,22,23,24,25-45°	9.5	28	4.9	3.1	9.0	0.10
BRH-1,2,3,4,5-N	6.4	63	8.2	1.1	2.6	0.41
BRH-11,12,13,14,15-N	9.5	63	13.4	0.5	2.7	0.26
CMU-1,2,3,4,5-N	6.4	35	9.4	0.8	3.7	0.47
CMU-11,12,13,14,15-N	9.5	35	6.0	1.4	4.1	0.12
CMU-1,2,3,4,5-45°	6.4	49	7.9	8.3	8.6	0.40
CMU-11,12,13,14,15-45°	9.5	49	7.9	6.4	7.9	0.15
CMUE-1,2,3,4-N	6.4	55	15.1	0.8	1.4	0.76
CMUE-11,12,13,14-N	9.5	55	6.1	6.1	14.5	0.12
CMUE-1,2,3,4,5-45°	6.4	77	11.0	13.0	13.0	0.55
CMUE-11,12,13,14,15-45°	9.5	77	9.5	6.4	6.4	0.18
CON2-1,2,3,4-N	6.4	88	16.2	3.5	11.2	0.81
CON2-5,6,7,8-N	6.4	114	18.5	4.4	12.2	0.93
CON2-9,10,11,12-N	9.5	88	32.6	2.3	16.4	0.64
CON2-13,14,15,16-N	9.5	114	38.4	2.0	9.3	0.75
CON2-1,2,3,4-45°	6.4	88	6.0	17.7	18.0	0.30
CON2-5,6,7,8-45°	6.4	114	6.8	10.9	12.1	0.34
CON2-9,10,11,12-45°	9.5	88	13.8	10.8	12.3	0.27
CON2-13,14,15,16-45°	9.5	114	16.0	15.0	17.2	0.31
CON2-17,18,19-N	6.4	88	15.4	3.2	11.3	0.77
CON2-20,21,22-N	6.4	88	18.8	3.3	6.4	0.94
CON2-23,24,25-N	6.4	114	17.8	1.9	4.0	0.89
CON2-26,27,28-N	6.4	114	19.5	3.0	3.3	0.98
CON2-29,30,31-N	9.5	88	29.2	1.9	3.3	0.57
CON2-32,33,34-N	9.5	88	31.8	2.5	12.0	0.62
CON2-35,36,37-N	9.5	114	40.8	2.5	10.7	0.79
CON2-38,39,40-N	9.5	114	41.5	2.0	10.9	0.81
CON3-1,2,3,4-N	6.4	88	17.6	4.2	5.2	0.88

Table 4.1 (Continued)

Test Specimen	Anchor Diameter (mm)	Embedment (mm)	Average Peak Load (kN)	Average Slip at Peak Load (mm)	Average Slip at Failure (mm)	Normalized Peak Load
CON3-5,6,7,8-N	6.4	114	18.5	4.5	5.8	0.93
CON3-9,10,11,12-N	9.5	88	40.7	3.2	12.8	0.79
CON3-13,14,15,16-N	9.5	114	54.8	3.4	16.5	1.07
CON3-1,2,3,4-45°	6.4	88	6.6	12.2	13.3	0.33
CON3-5,6,7,8-45°	6.4	114	6.7	10.4	12.0	0.34
CON3-9,10,11,12-45°	9.5	88	20.0	15.4	18.9	0.39
CON3-13,14,15,16-45°	9.5	114	23.1	15.7	17.6	0.45
CON3-17,18,19-N	6.4	88	17.3	2.3	3.3	0.87
CON3-20,21,22-N	6.4	88	17.8	2.4	3.0	0.89
CON3-23,24,25-N	6.4	114	17.0	1.4	2.1	0.85
CON3-26,27,28-N	6.4	114	18.4	1.7	4.5	0.92
CON3-29,30,31-N	9.5	88	26.3	1.7	8.7	0.51
CON3-32,33,34-N	9.5	88	33.5	1.0	8.7	0.65
CON3-35,36,37-N	9.5	114	45.2	2.0	2.7	0.88
CON3-38,39,40-N	9.5	114	48.3	2.5	13.4	0.94
LS-1,2,3,4-N	6.4	88	17.8	1.6	9.5	0.89
LS-15,16,17,18-N	9.5	88	48.3	2.3	9.7	0.94
LS-21,22,23,24-N	6.4	114	18.8	2.2	8.2	0.94
LS-35,36,37,38-N	9.5	114	49.0	2.1	12.7	0.95
LS-1,2,3,4,-45°	6.4	88	9.2	15.2	17.7	0.46
LS-15,16,17,18-45°	9.5	88	17.2	16.5	17.3	0.34
LS-21,22,23,24-45°	6.4	114	9.2	14.4	18.0	0.46
LS-31,32,33,34-45°	9.5	114	23.9	17.0	19.3	0.47
LS-41,42,43-N	6.4	88	16.0	2.3	6.7	0.80
LS-44,45,46-N	6.4	88	16.4	2.3	9.8	0.82
LS-47,48,49-N	6.4	114	17.5	1.3	4.8	0.88
LS-50,51,52-N	6.4	114	18.7	9.9	14.2	0.94
LS-53,54,55-N	9.5	88	42.7	1.7	14.9	0.83
LS-56,57,58-N	9.5	88	40.4	1.2	14.7	0.79
LS-59,60,61-N	9.5	114	44.1	1.9	22.0	0.86
LS-62,63,64-N	9.5	114	47.1	1.7	6.6	0.92

It is recognized that the calculated statistical parameters, such as mean and standard deviation, may not fully convey the significance of the experimental errors and the expected variabilities in the materials properties and fabrication of test specimens because the number of repeat specimens is relatively small. Other statistical analysis methods, such as student's t-distribution may be more appropriate; however, the statistical data are given for information only and are not used to derive any design equations or to make any significant inferences.

4.3.1 Effect of concrete strength

Many aspects of concrete behaviour are related to its compressive strength, including the performance of adhesively bonded anchors in concrete. In this research, two concrete compressive strengths were used; namely 20.6 MPa and 34.5 MPa. To better understand the effect of concrete strength on the behaviour and strength of adhesive anchors, we only compare the behaviour of centrally installed anchors. Furthermore, the comparison is based on the ratio of average peak load to rod tensile strength and the slip at peak load.

Figure 4.26 shows the normalized peak loads for the 6.4 mm anchors. We observe that for vertical anchors with 88 mm embedment the normalized peak load in the 20.6 MPa and the 34.5 MPa concrete differ by about 10%. The difference practically vanishes as the anchor depth increases to 114 mm. This is due to the fact that the anchor pullout resistance increases with increasing in embedment and the failure mode changes from pullout to steel fracture. Figure 4.26 (b) shows that concrete compressive strength does not significantly affect the peak load for anchors installed at 45-degree inclination. The

reason is that all these anchors failed by steel fracture. Further study is needed to investigate the effect of anchor inclination on its strength and stiffness.

(a)

(b)

Figure 4.26: Effect of concrete strength on the behaviour of 6.4 mm diameter anchors

For the 9.5 mm anchors, considering Figure 4.27, the effect of concrete compressive strength is obvious because the peak load increases with increasing concrete strength. The reason is that the combined cone-bond failure mode was the dominant failure mode for the 9.5 mm anchors with shallow embedment and lower concrete strength. When concrete strength increased, the anchor resistance to pullout increased due to the higher concrete strength and the failure mode changed to pullout or steel fracture. From Figure 4.27 (a), it is clear that the larger diameter anchor required both the higher concrete strength and the higher embedment length to achieve its full strength, increasing either quantity alone was not sufficient.

Figure 4.27 (b) also shows the results for the 9.5 mm diameter inclined anchors. It indicates that under the same conditions, anchor pullout resistance in the 34.5 MPa concrete is noticeably larger than in the 20.6 MPa concrete. The reason is that concrete splitting was the major failure mode for the 9.5 mm diameter inclined anchors. Cracking was caused by tensile stresses in the concrete. Since the tensile strength of concrete is a function of its compressive strength, increasing the concrete strength can contribute to the anchor pullout resistance.

From the above discussion, it is clear that the concrete compressive strength has an important effect on the behaviour and strength of adhesive anchors. This is particularly true for large diameter anchors with shallow embedment length. Generally, anchors in weaker concrete would require larger embedment to achieve their capacity.

Figure 4.27: Effect of concrete strength on the behaviour of 9.5 mm diameter anchors

4.3.2 Effect of anchor diameter

Two anchor diameters, 6.4 mm or 9.5 mm, were used to and their effects on the behaviour and strength of adhesive anchors are discussed in the following sections with regard to the base material used.

4.3.2.1 Anchors in hollow clay brick

Figure 4.28 shows the effect of anchor diameter in hollow brick. Each bar on the graph represents the mean of five replicate specimens. It is observed that the normalized peak load decreases when using bigger anchor diameter. The reason is that the nylon screen, used for 9.5 mm diameter anchors, decreases the mechanical interlock and friction between the hollow clay brick and the adhesive, while the stainless steel screen used in conjunction with the 6.4 mm diameter anchors contributed to the friction inside the base material. Figure 4.28 also indicates that the average normalized peak load for the 6.4 mm diameter anchors is approximately two times that for the 9.5 mm diameter anchors.

Overall we can see that with the hollow clay brick the smaller diameter anchors achieved 41% of their ideal strength. This value was even lower with the larger anchors. Also the inclined anchors performed poorly because they attained a small fraction of their ideal strength at the expense of relatively large slip. For instance, the 9.5 mm diameter inclined anchor achieved only 10% of its ideal strength at 3 mm slip. Therefore, in practice we should try to avoid inclined anchors in hollow brick, but if we must use them, then we should use smaller diameter anchors whenever possible.

Figure 4.28: Effect of anchor diameter in hollow clay brick

4.3.2.2 Anchors in hollow concrete block

The effect of anchor diameter in hollow concrete blocks is illustrated in Figure 4.29. Each bar on the graph represents the mean of five replicate specimens. It is observed that the normalized peak load is significantly affected by the anchor diameter – the bigger the anchor diameter, the lower the value of normalized peak load and the larger the anchor slip at peak load. The reason is that the stainless steel screen used for the 6.4 mm diameter anchors increased the mechanical friction between the adhesive and the hollow concrete block, while the plastic nylon screen used for the 9.5 mm diameter anchors did not. Test observations also revealed that the nylon screen was heavily damaged after testing, while the stainless steel screen did not show obvious damage. Most of the 9.5 mm diameter anchors experienced pullout (bond) failure and large displacement.

(a)

(b)

Figure 4.29: Effect of anchor diameter in hollow concrete block

Figure 4.29 (a) shows that with adequate embedment length the small vertically installed anchors achieved nearly 76% of their ideal strength compared to only 12% for the large anchors. The concomitant anchor slip at peak load in the small anchor was also relatively small. On the contrary, the larger anchors performed poorly because they failed at a low load and experienced large displacement.

The same observations could be made with regard to the inclined anchors. Figure 4.29 (b) shows that the smaller anchors with adequate embedment length achieved 55% of their ideal strength, but the corresponding displacement was substantially larger than that experienced by the comparable vertical anchors. It is, however, unexpected that the inclined larger anchors reached a higher proportion of their ideal capacity compared to vertical anchors. Notice that the inclined anchors experienced substantial displacement at peak load.

4.3.2.3 Anchors in concrete

Figures 4.30 and 4.31 show the effect of anchor diameter in 20.6 MPa and 34.5 MPa concrete, respectively. Each bar on the graphs represents the mean of four replicate specimens. The graphs show that in 20.6 MPa concrete, the values of normalized peak load for the 6.4 mm anchors are higher than those for the 9.5 mm diameter anchors, while in 34.5 MPa concrete, the larger diameter anchors are equally efficient.

Figure 4.30 (a) clearly shows that in lower strength concretes one should use smaller diameter anchors. This is not unexpected because the required bond stresses for developing the full strength of the smaller anchor are small compared to the bond strength of higher strength concrete.

(a)

(b)

Figure 4.30: Effect of anchor diameter in 20.6 MPa concrete

(a)

(b)

Figure 4.31: Effect of anchor diameter in 34.5 MPa concrete

The higher strength concrete improves the performance of the inclined anchors, but not substantially. Comparing the results in Figure 4.30 (b) and 4.31 (b), it is evident that once again it is the larger diameter anchor that benefits from the higher strength of concrete. On the other hand, the higher strength does not diminish the amount slip.

4.3.2.4 Anchors in limestone

Figure 4.32 shows the effect of anchor diameter in limestone. Each bar graph represents the mean of four replicate specimens. The graphs in Figure 4.32 (a) indicate that under the same conditions, both anchor sizes installed vertically almost reached their full tensile strength, which means that the limestone has high tensile and shear strength. Notice also that the slip at peak load for both anchor sizes and embedment lengths are practically the same. Hence, limestone is a good medium for the installation of anchors. Since many older structures have facade walls that are made of stone, this means that one can anchor window frames into these walls without fear of complete pullout, even if the embedment depth is not very substantial.

In the case of inclined anchors, once again the anchor inclination has a negative effect on its strength. As Figure 4.32 (b) shows, the inclined anchors only achieved 46% of their ideal strength while experiencing substantial amount of slip. Nevertheless, comparatively, these anchors performed better than the similar anchors in concrete.

(a)

(b)

Figure 4.32: Effect of anchor diameter in limestone

4.3.3 Effect of anchor embedment length

Anchor embedment length in this research work varied with base material. In the following sections, the effect of embedment length on anchor performance in each of the four base materials will be considered.

4.3.3.1 Anchors in hollow clay brick

Embedment of 28 mm and 63 mm were used for vertical anchors in hollow clay brick. Figure 4.33 shows the effect of embedment depth on the anchor behaviour and strength. Each bar graph represents the mean of five replicate specimens. For the 6.4 mm anchors, the value of normalized peak load increased 71% when anchor depth increased from 28 mm to 63 mm, while it increases 101% for the 9.5 mm diameter anchors. It seems that increase in embedment is more effective for the 9.5 mm diameter anchors, but the relationship between normalized peak load and embedment is not linear.

Figure 4.33: Effect of anchor embedment length in hollow clay brick

Obviously the required embedment is inversely proportional to the epoxy-brick material interface strength. However, the relationship is not linear because the bond stresses are not uniformly distributed. The results here, once again point to the fact that one should use smaller size anchors in clay brick because they tend to produce less tension in the brick and therefore the brick would not split at low levels of applied load acting on the anchor.

4.3.3.2 Anchors in hollow concrete block

For the vertical anchors installed in concrete blocks, 35 mm and 55 mm embedment were used. The effect of embedment on the normalized peak load in these anchors is shown in Figure 4.34. Figure 4.34 (a) shows that for vertical anchors, the normalized peak load increased 60% for the 6.4 mm anchors when embedment increased from 35 mm to 55 mm, but the effect on the 9.5 mm anchor was negligible. The latter is somewhat surprising because one would expect some increase in the anchor capacity due to increase in embedment length. On the other hand, if splitting failure occurred, then the increase in embedment length would not be governing the anchor capacity.

In the case of inclined anchors, as Figure 4.34 (b) shows, the increase in embedment from 49 mm to 77 mm increased the efficiency of the 6.4 mm diameter anchor by nearly 38%, but it did little for the 9.5 mm anchor. This is in agreement with the results in Figure 4.34 (a), where the increase in embedment did not contribute much to the strength of the larger anchor.

(a)

(b)

Figure 4.34: Effect of embedment length in hollow concrete block

4.3.3.3 Anchors in concrete

Embedment in concrete was 88 mm or 114 mm. Figures 4.35 and 4.36 show the effect of embedment on normalized peak load in the 20.6 MPa and the 34.5 MPa concrete, respectively. Figures 4.35 (a) and 4.36 (a) show that both the smaller and larger diameter anchors installed vertically required not only the higher strength concrete but also the greater embedment to achieve their higher capacity.

In the case of the inclined anchors, as Figures 4.35 (b) and 4.36 (b) shown, irrespective of the anchor size, embedment length or concrete strength, none of the anchors achieved more than 45% of their ideal capacity. Furthermore, the peak loads were accompanied by large anchor displacements. Therefore, once again one can clearly see the detrimental effect of anchor inclination on its resistance and stiffness.

4.3.3.4 Anchors in limestone

The anchor embedments used were 88 mm or 114 mm. Figure 4.37 presents the effect of anchor embedment length on the normalized peak load and the corresponding displacement. For anchors installed vertically, Figure 4.37 (a) shows that there is no significant difference in normalized peak load when the embedment increased from 88 mm to 114 mm. It seems that due to the high bond strength of limestone, the shorter embedment depth is adequate for both anchor sizes. Similarly, the anchor displacement at peak load is small for the specimens in this series. It may be recalled that the vertical anchors in limestone all failed due to steel fracture.

(a)

(b)

Figure 4.35: Effect of embedment length in 20.6 MPa concrete

(a)

(b)

Figure 4.36: Effect of embedment length in 34.5MPa concrete

(a)

(b)

Figure 4.37: Effect of embedment length in limestone

For anchors installed at 45-degree, results in Figure 4.37 (b) show that the normalized peak load for the 9.5 mm anchors increased 38% when the embedment length increased from 88 mm to 114 mm. On the other hand, the 6.4 mm diameter anchors did not exhibit any capacity gain due to the increase in embedment. It would appear from the results for the smaller anchor that in the case of the inclined configurations, their capacities are limited by their inclination, irrespective of the embedment length or base material strength.

4.3.4 Effect of anchor inclination

Anchors were installed either vertically or at 45-degree inclination in this research work. The effect of anchor inclination on the normalized peak load and the corresponding anchor displacement in each of the base materials is discussed in the following section.

4.3.4.1 Anchors in hollow clay brick

Figure 4.38 shows the effect of anchor inclination on the capacity of the anchors embedded in hollow clay brick. Each bar graph represents the mean of five replicate specimens. The graph shows that the anchor inclination does not have a major effect on the anchor normalized peak load, but it does strongly affect its displacement. We can observe that for both anchor sizes, the displacement at peak load dramatically increased as the anchor inclination changed from 90° to 45°. Since in hollow clay brick, splitting of the brick web was the dominant failure mode, it means that anchor inclination does not determine the failure load in this base material.

Figure 4.38: Effect of anchor inclination in hollow clay brick

4.3.4.2 Anchors in hollow concrete block

The effect of anchor inclination on the strength of anchors embedded in hollow concrete blocks is shown in Figure 4.39. Once again, as in the case of hollow clay brick, the anchor inclination does not have a major effect on the normalized peak load, but it has a dramatic effect on the anchor displacement. For the smaller anchors there is a relatively small drop in the anchor peak load when the inclination changed from 90° to 45°, but for the larger anchors the reduction is even smaller. These small differences in the anchor peak loads are within the normal variations one would expect and therefore cannot be attributed solely to the anchor inclination.

Figure 4.39: Effect of anchor inclination in hollow concrete block

It would appear that in both the hollow clay brick and the hollow concrete block, the splitting of the material, which is caused by combined stresses due to shear at the epoxy-base material interface and the bearing stresses due to the epoxy filled screens in the hollow part, determine the anchor capacity.

4.3.4.3 Anchors in concrete

Figures 4.40 and 4.41 depict the effect of anchor inclination on the peak load and its corresponding displacement for anchors embedded in the 20.6 MPa and 34.5 MPa concrete cylinders. Each bar graph represents the average value obtained from four replicate test specimens. It is evident from these figures that irrespective of the other anchoring parameters (i.e. diameter, embedment and base material strength), the change in anchor inclination from 90° to 45° dramatically decreases its normalized peak load and

increases its corresponding displacement. Therefore, in the case of concrete, one could clearly state that whenever possible inclined anchors should be avoided.

(a)

(b)

Figure 4.40: Effect of anchor inclination in 20.6 MPa concrete

(a)

(b)

Figure 4.41: Effect of anchor inclination in 34.5 MPa concrete

4.3.4.4 Anchors in limestone

Figure 4.42 shows the effect of anchor inclination in limestone. Each bar graph represents the mean of four replicate specimens. The results in this case are very similar to those observed in concrete (i.e. the change in inclination from 90° to 45° substantially decreases the normalized peak load and increases the corresponding displacement, irrespective of anchor size and embedment depth). Hence for both solid media (i.e. concrete and limestone), whenever possible, one must avoid incline installations, at least (from this study) close to 45° angle.

The results in the last four subsections clearly demonstrate the negative effect of anchor inclination, or deviation from the vertical, on anchor capacity and displacements. It is, however, not clear if other anchor inclinations, with smaller deviation from the vertical, would have such a major impact on capacity. Clearly, further study is needed to better evaluate the effect of this important parameter on the anchor in those base materials where splitting failure is not the dominant mode.

4.3.5 Effect of anchor edge distance

In this study, edge effect was investigated only in concrete and limestone specimens. All anchors were installed with either 40 mm or 60 mm edge distance. The results are discussed in the following two subsections. Each bar graph in Figures 4.43 to 4.45 represents the mean of at least three replicate specimens. Notice that the results in these figures are presented in terms of normalized peak load and the corresponding displacement, but the normalizing factor is the peak load of the centrally installed

specimens with otherwise similar properties. Hence, the normalized values at a glance show whether edge effect has any significant effect on the anchor properties.

(a)

(b)

Figure 4.42: Effect of anchor inclination in limestone

4.3.5.1 Anchors in concrete

It is evident from Figure 4.43 that the particular edge distances have marginal effect on the behaviour of these anchors. Since neither the anchor peak load nor its corresponding displacement is affected by even the smaller edge distance (40 mm), we can conclude that even a moderately strong concrete will not be affected by edge distance, provided that the edge distance is at least 4 times the anchor diameter.

Figure 4.43 shows data for the anchors installed in the 20.6 MPa concrete. The results in Figure 4.43 (a), which pertain to the smaller diameter anchor, show that edge distance has negligible effect on the normalized anchor peak load, although surprisingly, it reduced anchor slip relative to the centrally installed anchor. The latter is counter-intuitive and needs to be further investigated. One possible reason may be that the top loading ring exerted higher bearing stresses in the vicinity of the anchor and thus reduced anchor slippage. This is a consequence of the testing method rather than the reflective of the actual behaviour of an eccentrically installed anchor. The results for the 9.5 mm anchor in Figure 4.43 (b) indicate that the chosen edge distances had no practical impact on the behaviour of the anchors embedded in the 20.6 MPa concrete.

Figure 4.44 (a) and (b) depict the normalized peak load and the corresponding displacement for the small and large anchor, respectively, installed in 34.5 MPa concrete. It is clear from Figure 4.44 (a), that this parameter does not have any significant effect on the behaviour of the 6.4 mm diameter anchors. Again the larger displacement of the centrally installed anchor may be due to the particular testing configuration. Figure 4.44 (b), on the other hand, shows that the large anchor with the smaller embedment is influenced by the edge distance. An increase in edge distance increases the normalized

peak load. The same trend is observed for the larger anchor embedment, but the percentage increase in normalized peak load, as a function of edge distance, is not as high.

(a)

(b)

Figure 4.43: Effect of edge distance in 20.6 MPa concrete

(a)

(b)

Figure 4.44: Effect of edge distance in 34.5 MPa concrete

It must be stated that the large anchors in the higher strength concrete resist higher loads and therefore are likely to exert greater internal stresses in the base material. Consequently, the edge effect is more likely to be important in shallow embedment depths because any damage to the base material in the vicinity of the anchor will lead to a reduction in resistance of the epoxy-base material interface, causing loss of bond. Hence, it is difficult to separate edge effect from the other anchor installation parameters. In other words, it would not be appropriate to specify a minimum edge distance, irrespective of the anchor size, embedment length and base material properties. In general, however, it is better to avoid eccentrically installed anchors. If an anchor is to be installed eccentrically, then it is better to have a greater embedment than for a centrally installed anchor. Of course, the critical edge distance will strongly depend on the anchor diameter.

4.3.5.2 Anchors in limestone

Figure 4.45 shows the effect of edge distance on the anchors installed in limestone. It is observed that for the 6.4 mm anchors there is an approximately 10% drop in normalized peak load due to edge distance. When anchor embedment is increased to 114 mm, only the anchors with 40 mm edge distance were affected. The larger displacement of the anchors with edge distance of 60 mm and embedment length of 114 mm is rather anomalous and may well be due to the stone properties. Detailed examinations of the data from the three replicate specimens showed displacements of 2.2, 16.8 and 10.8 mm for this edge distance and embedment. These values show a great amount of scatter and therefore do not necessarily reflect the properties of such installations in this substrate in general.

(a)

(b)

Figure 4.45: Effect of edge distance in limestone

For the 9.5 mm diameter anchors, the edge distance has a small effect on the peak load, and it generally causes a reduction in its magnitude. The relative reduction is higher for the smaller embedment. Overall, however, the effect of this parameter, at least for the chosen edge distances, appears to be relatively small.

It must be recognized that in practice it may be difficult to drill holes too close to the edge of a base material because the drilling operation and the corresponding vibration will likely crack the base material. Later, a procedure will be presented which will account for edge distance effect on the anchor strength.

The above test results and observation indicate that anchor edge distance and its associated embedment, together affect the strength and behaviour of chemical anchors. Basically, at the same edge distance, the bond strength for anchors with shallow embedment decreases much more than that of anchors with deeper embedment. The reason is that shallow embedment causes more lateral pressure on the anchor, and correspondingly higher tension in the base material.

4.4 Summary

The results in the preceding few sections have clearly demonstrated the effect of some important parameters on the peak load that an anchor can withstand. It has been shown that of all the parameters considered, the anchor inclination had the greatest effect on the capacity. More importantly, the failure mode is the primary determinant of the anchor capacity. As was demonstrated even anchor inclination may not be important if failure is caused by the splitting of the base material. Hence, whether a parameter is

important or not, depends on the entire anchor system (i.e. anchor size, embedment depth, base material, etc.).

As will be shown in the following chapter, the anchor strength will be determined by the smallest of the loads required to induce each of the failure modes. Since each failure mode is not a function of all the parameters considered in this study, the significance of a parameter can only be gauged by its effect on a given failure mode.

Chapter 5

Numerical Analysis

5.1 Introduction

In this chapter, three dimensional linear elastic finite element analyses are performed to determine the stress distribution in the base material and in the vicinity of the anchor. In each case, the anchor is subjected to direct tension at its free or unembedded end, and the normal and shear stresses at the anchor rod-base material interface as well as in the base material around the anchor are determined. The results of this analysis could be used to determine the cracking of the base material. In the present numerical analysis, the commercial finite element program COSMOS is used.

5.2 Linear elastic numerical analysis

Pullout of chemically bonded anchors was simulated using the computer program COSMOS, a commercial finite element software package. The element used for the linear elastic finite element model of the single anchor system was the three-dimension, 8-node, isoparametric solid element. The same element was used to discretize the steel anchor rod, the epoxy, the solid concrete and the limestone. This element is based on a formulation that includes incompatible displacement shapes and the use of $2 \times 2 \times 2$ integration points to determine the element stiffness matrix.

Based on the experimental results in this research, the steel anchor was modeled as a linear elastic material with Young's modulus $E = 210 \text{ GPa}$ and Poisson's ration $\nu =$

0.3. The 20.6 MPa concrete was modeled using Young's modulus $E = 21$ GPa and Poisson's ratio $\nu = 0.2$, and the 34.5 MPa concrete was modelled using Young's modulus $E = 27.5$ GPa and Poisson's ratio $\nu = 0.2$. Limestone was modeled with Young's modulus $E = 30$ GPa and Poisson's ratio $\nu = 0.25$. Based on the published data from the adhesive manufacturer, a value of 4.2 MPa was used for the epoxy modulus of elasticity and a value of 0.37 for its Poisson's ratio.

Figure 5.1: Finite element model with load and boundary conditions

Taking advantage of symmetry, only a quarter of the anchor sample was numerically discretized. A typical model with, the load and boundary conditions as

shown in Figure 5.1, was used, which includes the discretization of the steel anchor, the epoxy, the solid concrete and the limestone. Figure 5.2 depicts a representative finite element mesh that was used. The wedge shape solid was 150 mm long, 150 mm wide and 150 mm deep. The mesh had 3000 epoxy elements, 1600 steel elements, 3000 concrete or limestone elements in the neighbourhood of the epoxy and 2700 concrete or limestone elements in the remaining part. Tensile loading was applied at the top or free end of the steel anchor. The boundary conditions consists of restraint of translation in the radial direction and of translation in X, Z direction on both planes of symmetry and restraint of translation in the X, Y, Z direction along the support lines as marked in Figure 5.1.

Figure 5.2: Finite element discretization of one quarter of the anchor system

5.3 Result of the numerical analysis

Figure 5.3 shows the distribution of the shear stresses τ_{xy} (τ_{zy}) for the 9.53 mm anchor in 20.6 MPa concrete with a 114 mm vertical embedment and subjected to 20 kN tensile loading. It clearly indicates the conical variation of these stresses and thus explains the conical failure mode that was observed in some of the laboratory tests. The cone is symmetrical around the anchor and the high stresses are limited to a certain region near the surface of the base material. This distribution may explain the relatively shallow depth of the failure cone. It must be emphasized that the failure cone surface is approximately normal to the direction of the principal tensile stresses.

Figure 5.3: Distribution of the shear stresses τ_{xy} (anchor diameter= 9.5 mm, embedment = 114 mm, slope = 90° and tensile load = 20 kN in 20.6 MPa concrete)

The distribution of shear stress τ_{xy} and normal stress σ_{xx} along the epoxy-concrete interface is plotted in Figure 5.4. The stresses are plotted against normalized anchor depth (i.e. distance from the top surface of the block divided by the anchor depth). It can be seen that the maximum normal and shear stresses occur just below the base material surface, and then drop rapidly with depth. The analysis also indicates that the tension zone of the epoxy-concrete interface ranges from the top surface to about 80% of the anchor normalized depth. To obtain a picture of the shear τ_{xy} and normal stress σ_{xx} distribution away from the interface, these stresses are plotted in Figure 5.5 along a radial line located below the surface at the level at which the anchor-epoxy interfacial shear stresses attain their maximum value. From Figure 5.5, it is quite evident that the peak shear stress τ_{xy} occurs in the epoxy and then drops sharply as one moves away from the interfaces. Similarly the radial tensile stresses are maximum at the interface and they drop sharply as one moves away from the interface. The normal stress σ_{xx} increases within the epoxy and its peak value occurs at the epoxy-concrete interface; then it drops down fast and changes to a small amount of compression.

In Figures 5.6 and 5.7 the shear and normal stresses distribution are plotted for the 9.5 mm diameter anchor with 114 mm vertical embedment length in limestone and subjected to 20 kN tensile loading. As expected, there is no significant difference in the stress distribution from that observed previously for the anchor in concrete, but the epoxy and limestone have higher values of maximum shear and normal stresses. This is because the Young's modulus of limestone is larger than that of 20.6 MPa concrete and accordingly resists higher stresses for the same strain.

Figure 5.4: Distribution of shear and normal stress along epoxy-concrete interface (Anchor diameter = 9.5 mm, anchor depth = 114 mm and tensile load = 20 kN in 20.6 MPa concrete)

Figure 5.5: Shear and normal stress of epoxy and concrete elements (Anchor diameter = 9.5 mm, anchor depth = 114 mm and tensile load = 20 kN)

Figure 5.6: Distribution of shear and normal stress along epoxy-limestone interface
(Anchor diameter = 9.5 mm, depth = 114 mm and tensile load = 20 kN in
32.5 MPa limestone)

Figure 5.7: Shear and normal stress in epoxy and limestone just below the stone surface
(Anchor diameter = 9.5 mm, anchor depth = 114 mm and tensile load = 20 kN)

For the anchors in limestone but with 88 mm embedment length, the distribution of stresses in epoxy and limestone are presented in Figures 5.8 and 5.9. These figures show that the maximum shear and normal stresses decrease slightly due to the smaller embedment (88 mm).

It is important to emphasize that the purpose of the present finite element analyses is not to predict all the different failure modes that were discussed in the preceding chapter. It is recognized that the prediction of all the failure modes requires full nonlinear analysis, including the proper modelling of the anchor rod-epoxy and epoxy-concrete interfaces. Furthermore, material nonlinearities in the anchor rod, the base material and the epoxy need to be considered and appropriate failure criteria must be used. Such analyses are beyond the objectives and scope of the current investigation.

The main purpose of the current analysis was to see whether the normal stress σ_{xx} and shear stress τ_{xy} attain their maximum values at nearly the same location. The analysis shows that indeed they do, and this means that the critical location for the initiation of failure can be relatively easily identified. This failure may involve loss of bond at the interface, which would lead to redistribution of stresses. On the other hand, if the tensile strength of the base material is lower than the strength of the interface, then cracking of the base material will occur. The cracks will most likely follow the direction of the principal compressive stresses. Hence, the results of the elastic analysis are useful in predicting cracking of the base material as well as in identifying other highly stressed regions.

Figure 5.8: Distribution of shear and normal stress along epoxy-limestone interface
(Anchor diameter = 9.5 mm, anchor depth = 88 mm and tensile load = 20 kN)

Figure 5.9: Shear and normal stress of epoxy and limestone just below the stone surface
(Anchor diameter = 9.5 mm, anchor depth = 88 mm and tensile load = 20 kN)

Chapter 6

Analysis and Proposed Design Method

6.1 Introduction

In this chapter, a basic analysis of each observed failure mode is presented. Based on these analyses and the test results, a design method for adhesively bonded anchors is proposed. As mentioned in Chapter 3, slip in this study is defined as the measured displacement of the anchor. This value actually includes the elongation of the anchor rod and its slip. Generally, however in the current study, the elongation was relatively small compared to the slip, unless the anchor rod yielded and appreciable slip did not occur. The latter was typically the case when failure was due to fracture of the steel rod, without appreciable pullout.

6.2 Anchor rod failure

6.2.1 Analysis of behaviour

Steel rod failure occurred in specimens set in hollow concrete block, solid concrete and limestone. The anchor displacement was measured using a Linear Voltage Differential Transformer (LVDT) located on the loaded end of the anchor, just above the base material top surface. The displacement therefore included the axial deformation of anchor steel and the slip in the base material. Then, generally for each anchor, the measured displacement is larger than the axial extension of the rod and the experimental

secant stiffness (F/Δ) is greater than the elastic stiffness of the rod (AE/λ), where F , Δ , A , E , and λ are the applied force, anchor elongation, elastic modulus, cross-sectional area and length of the anchor rod, respectively. Detailed anchor properties used in this work are presented in Section 3.4.1, which includes the threaded rods tensile strength, shear strength, yield strength, ultimate strength and their stress-strain relationship.

Experimental results from this work indicate that whenever vertical anchors failed due to steel fracture, they behaved basically the same as in the steel rod tension tests. In other words, when appreciable slip did not occur, the bars simply behaved in a tension coupon test. For the anchors installed at 45-degree inclination, however, the behaviour was not the same. This is because these anchors were subjected to combined tension, shear and bending, thus they failed at relatively lower load than the vertical anchors.

As shown in Figure 6.1 (a), the applied tensile force F always acted perpendicular to the substrate surface on which the anchor was installed. This force F can be resolved into its components normal and parallel to the anchor rod cross-section. The normal and parallel components are denoted as $F_x = F \cos \theta$ and $F_y = F \sin \theta$, where θ is the angle of the inclination of the anchor as shown in Figure 6.1 (b). Note that the angle θ in Figure 6.1 is the complementary angle of inclination and should not be confused with 45° and 90° angles of inclination referred to in the previous chapters.

Since the tangential component F_y induces the shear and bending stresses and the normal component F_x induces axial stress, the maximum tensile and shear stresses acting on the unembedded portion of the anchor rod are given by

(a) Position of inclined anchor versus the applied load

(b) Force components

Figure 6.1: Force components acting on the anchor steel

$$\sigma_{\max} = \frac{F_x}{A} + \frac{F_y L}{S} \quad (6.1)$$

$$\tau_{\max} = \frac{4F_y}{3A} \quad (6.2)$$

where L is the moment arm as shown in Figure 6.1 (b), and A and S are the area and elastic section modulus of the rod cross-section. For a circular section, the modulus S is given by (Gere and Timosheko, 1991)

$$S = \frac{\pi d^3}{32}$$

where, d is the anchor diameter. The reader may also refer to Gere and Timosheko (1991) for the derivation of Equation 6.2.

Since the maximum shear stress occurs at the neutral axis while the maximum normal stress occurs at the periphery of the cross-section of the rod, theoretically one must find the location of the critical point on the cross-section where the combined shear and normal stress will cause yielding. An alternative methods, albeit less accurate, is to combine the average shear stress $\tau_{avg.} = F_y / A$ with the maximum tensile stress in order to determine the yield load. Of course, the average shear stress in this case would be 25% less than the maximum shear stress.

If we calculate the principal stresses, σ_1 and σ_2 , corresponding to $\sigma_{\max}$ and $\tau_{avg.}$, that is

$$\sigma_1, \sigma_2 = \frac{\sigma_{\max}}{2} \pm \sqrt{\left(\frac{\sigma_{\max}}{2}\right)^2 + \tau_{avg.}^2} \quad (6.3)$$

then we can use the von-Mises yield criterion (Popov, 1990) to determine the yield force.

This criterion can be expressed as

$$\left(\frac{\sigma_1}{\sigma_y}\right)^2 - \left(\frac{\sigma_1}{\sigma_y} \frac{\sigma_2}{\sigma_y}\right) + \left(\frac{\sigma_2}{\sigma_y}\right)^2 = 1 \quad (6.4)$$

By expressing $\sigma_{\max}$ and τ_{avg} in terms of the applied load and then calculating σ_1 and σ_2 , we can use Equation 6.4 to calculate the yield load. To calculating the failure load, we need to use the plastic modulus of the section (Z). Since the plastic modulus is equal to the elastic modulus times the so-called shape factor f , we can approximate the applied load that would cause the full plasticization of the section as follows:

$$\sigma_{xx} = \frac{F_x}{A} + \frac{F_y L}{Z} \quad (6.5)$$

$$= \frac{F_x}{A} + \frac{F_y L}{fS} \quad (6.6)$$

where, for a circular section, $f = 1.7$ (Gere and Timosheko, 1991). It is assumed in Equation 6.6 that a yield plateau based on 0.2% strain can be defined. Alternatively,

$$\sigma_{xx} = \frac{F \cos \theta}{A} + \frac{FL \sin \theta}{1.7S} \quad (6.7)$$

where, F is the applied load that the anchor can carry based on normal stresses. This expression ignores strain-hardening.

Equation 6.7 does not include the effect of shear stresses. If we ignore the effect of shear and assume that failure will occur when the maximum normal stress equals the yield stress, then according to Equation 6.7,

$$F_u = \frac{\sigma_y}{\frac{\cos \theta}{A} + \frac{L \sin \theta}{1.7S}} \quad (6.8)$$

where, F_u is the ultimate capacity of the anchor based on maximum normal stress.

Alternatively, failure may occur when the maximum shear stress exceeds the shear capacity of the rod. The maximum shear stress based on the average shear stress distribution is given by

$$\tau_{\max} = \sqrt{\left(\frac{\sigma_{xx}}{2}\right)^2 + \tau_{avg}^2} \leq \frac{\sigma_y}{2} \quad (6.9)$$

$$= \sqrt{\left(\frac{1}{2}\left(\frac{F \cos \theta}{A} + \frac{FL \sin \theta}{1.7S}\right)\right)^2 + \left(\frac{F \sin \theta}{A}\right)^2} \quad (6.10)$$

$$= \frac{F}{2A} \sqrt{\left(\cos \theta + \frac{8L \sin \theta}{1.7d}\right)^2 + (2 \sin \theta)^2} \quad (6.11)$$

$$\text{or } F = 2\tau_{\max} A \frac{1}{\sqrt{\left(\cos \theta + \frac{8L \sin \theta}{1.7d}\right)^2 + (2 \sin \theta)^2}} \quad (6.12)$$

Based on the maximum shear stress criterion, yielding occurs when, $\tau_{\max} = \frac{\sigma_y}{2}$, hence

$$F_u = \frac{F_Y}{\sqrt{\left(\cos \theta + \frac{8L \sin \theta}{1.7d}\right)^2 + (2 \sin \theta)^2}} \quad (6.13)$$

$$\text{or } \frac{F_u}{F_Y} = \frac{1}{\sqrt{\left(\cos \theta + \frac{8L \sin \theta}{1.7d}\right)^2 + (2 \sin \theta)^2}} \quad (6.14)$$

where, $F_Y = \sigma_y A$

The test results indicate that the vertical anchors that did not pullout, failed due to fracturing of the steel. Therefore, their strengths are governed by Equation 6.8. On the other hand, when the anchors yielded but did not rupture, as in some inclined anchors,

their behaviours are likely governed by Equation 6.14. It is obvious that Equation 6.8 reverts to the usual axial strength equation of a rod for vertical anchors because both θ and L would be equal to zero. Thus for vertical anchor, $F_u = \sigma_y A$. This equation ignores strain-hardening, which as stated earlier would increase the ultimate capacity of the anchor, provided that anchor pullout can be prevented. From the design point of view, reliance on strain-hardening is not a good idea because the rod may pull-out before it can reach strain-hardening. Hence, for design purpose one can assume that $F_u = \sigma_y A$, when failure is governed by steel rupture.

In the case of the inclined anchors, as Equation 6.14 shows, the situation is more complex. A number of parameters will affect the ultimate strength of the anchor based on steel yielding. In addition to the anchor inclination θ , the moment arm L will also have a noticeable effect on the anchor capacity.

To check the sensitivity of the anchor strength to the moment arm, the ratio F_u / F_y is plotted against the magnitude of the moment arm in Figure 6.2. It is assumed that $\theta = 45^\circ$. It is evident that this quantity has a major effect on the failure load of the anchor. Note that a 5 mm moment arm can reduce the capacity of the anchor by 50%. Thus the relatively small capacity of the inclined anchors in this work can be explained by the effect of the length of the moment arm. Generally speaking, in the present tests, the moment arm was relatively small because the steel adaptor shown in Figure 6.1 (a) was placed very close to the surface of the base material surface. In practice, it is possible to minimize this moment as much as possible.

Effect of Moment arm on Anchor Failure Load Due to Steel Fracture

Figure 6.2: Effect of the length of moment arm on the anchor failure load due to steel fracture ($\theta = 45^\circ$)

6.2.2 Proposed method of calculating failure load based on steel rod rupture

The steel failure mechanism in tension is usually characterized by tensile yielding and fracture of the anchor rod. The average tensile strength of a single anchor is dependent on the effective stress area of the anchor and the tensile strength of steel. As discussed in the previous section, anchor capacity due to fracture can be expressed by Equations 6.8 and 6.14. In this research work, we adopt the strength factor (α) to account for the presence of combined tension, shear and bending. Hence the anchor ultimate tensile strength is given by the following:

$$F = \alpha f_y A_s \quad (6.15)$$

where F = Tensile capacity of anchor (N)

A_s = Effective area of anchor (mm^2)

f_y = Specified yield strength of anchor (MPa)

α = Strength factor for anchor rod failure

$$\alpha = \frac{1}{\sqrt{\left(\cos \theta + \frac{8L \sin \theta}{1.7d}\right)^2 + (2 \sin \theta)^2}}$$

Notice that as indicated in Section 3.5, based on the offset method, the yield strength of the anchor rod can be taken as approximately 77% and 88% of its ultimate tensile strength for the 6.4 mm and the 9.5 mm anchors, respectively.

Table 6.1 shows the value of the strength factor (α) based on the current test results. We can see that for vertically installed anchors, most values of α are close to 1.0, which means pure tension failure of the anchor steel. Although some values are lower than 1.0, the reason is that these anchors may not be installed at exactly 90-degree, and may be subjected to unintentional bending stresses. For the 6.4 mm anchors at 45-degree inclination, the average value of α is between 0.302 and 0.340 in concrete while the average value in limestone is 0.408. Due to its larger diameter, the 9.5 mm anchor has a higher section modulus that results in lower bending stresses; hence the average value of α for the 9.5 mm anchor at 45-degree inclination is between 0.293 and 0.459 in concrete, while the average value in limestone is 0.332. Table 6.1 also indicates that in hollow concrete block, the average value of α for the 6.4 mm diameter anchors at 45-degree inclination is 0.586. The reason for this higher value is that the hollow concrete block is made of less rigid material compared to concrete and limestone and damages more easily (crush and spall) due to the bending of the rod. This allows the anchor to reduce its

Table 6.1: Value of Strength factor α for anchor steel

Specimen	Diameter (mm)	Embedment (mm)	Nominal Tensile Strength (kN)	Test Result (kN)	Coefficient (α)	Mean of Coefficient	Standard Deviation of Coefficient
CON2-1-N	6.4	88	19.9	17.6	0.883	0.886	0.059
CON2-2-N	6.4	88	19.9	16.4	0.825		
CON2-20-N	6.4	88	19.9	17.3	0.872		
CON2-21-N	6.4	88	19.9	19.2	0.966		
CON2-6-N	6.4	114	19.9	18.9	0.950	0.952	0.081
CON2-7-N	6.4	114	19.9	18.6	0.933		
CON2-24-N	6.4	114	19.9	16.0	0.804		
CON2-25-N	6.4	114	19.9	20.7	1.042		
CON2-26-N	6.4	114	19.9	19.5	0.978		
CON2-27-N	6.4	114	19.9	18.3	0.919		
CON2-28-N	6.4	114	19.9	20.6	1.037		
CON3-1-N	6.4	88	19.9	17.4	0.874	0.881	0.040
CON3-2-N	6.4	88	19.9	18.1	0.907		
CON3-3-N	6.4	88	19.9	16.6	0.836		
CON3-4-N	6.4	88	19.9	18.1	0.910		
CON3-17-N	6.4	88	19.9	17.9	0.900		
CON3-18-N	6.4	88	19.9	17.0	0.854		
CON3-19-N	6.4	88	19.9	17.0	0.856		
CON3-20-N	6.4	88	19.9	17.0	0.855		
CON3-21-N	6.4	88	19.9	17.0	0.855		
CON3-22-N	6.4	88	19.9	19.2	0.967		
CON3-5-N	6.4	114	19.9	18.3	0.920	0.911	0.067
CON3-6-N	6.4	114	19.9	18.3	0.919		
CON3-7-N	6.4	114	19.9	18.6	0.934		
CON3-8-N	6.4	114	19.9	18.9	0.948		
CON3-23-N	6.4	114	19.9	15.2	0.763		
CON3-24-N	6.4	114	19.9	18.7	0.941		
CON3-25-N	6.4	114	19.9	17.1	0.858		
CON3-26-N	6.4	114	19.9	18.2	0.912		
CON3-28-N	6.4	114	19.9	20.0	1.003		
LS-1-N	6.4	88	19.9	20.4	1.023	0.872	0.074
LS-2-N	6.4	88	19.9	16.4	0.825		
LS-3-N	6.4	88	19.9	16.2	0.814		
LS-4-N	6.4	88	19.9	18.1	0.907		
LS-41-N	6.4	88	19.9	16.0	0.802		
LS-42-N	6.4	88	19.9	18.0	0.902		
LS-45-N	6.4	88	19.9	15.7	0.789		
LS-47-N	6.4	88	19.9	17.4	0.875		
LS-49-N	6.4	88	19.9	18.1	0.912		

Table 6.1 (Continued)

Specimen	Diameter (mm)	Embedment (mm)	Nominal Tensile Strength (kN)	Test Result (kN)	Coefficient (α)	Mean of Coefficient	Standard Deviation of Coefficient
LS-21-N	6.4	114	19.9	18.4	0.922	0.965	0.064
LS-22-N	6.4	114	19.9	18.6	0.936		
LS-23-N	6.4	114	19.9	20.6	1.034		
LS-24-N	6.4	114	19.9	17.7	0.887		
LS-50-N	6.4	114	19.9	19.2	0.966		
LS-51-N	6.4	114	19.9	20.8	1.047		
CON2-13-N	9.5	114	51.9	40.8	0.786	0.944	0.098
CON2-16-N	9.5	114	51.9	36.5	0.703		
CON2-39-N	9.5	114	51.9	49.2	0.947		
CON3-13-N	9.5	114	51.9	50.4	0.970	1.014	0.064
CON3-14-N	9.5	114	51.9	53.3	1.027		
CON3-15-N	9.5	114	51.9	56.9	1.097		
CON3-16-N	9.5	114	51.9	54.0	1.041		
CON3-38-N	9.5	114	51.9	48.5	0.933		
LS-15-N	9.5	88	51.9	45.9	0.884	0.930	0.056
LS-16-N	9.5	88	51.9	47.6	0.918		
LS-17-N	9.5	88	51.9	52.5	1.011		
LS-18-N	9.5	88	51.9	47.0	0.905		
LS-35-N	9.5	114	51.9	47.6	0.917	0.961	0.076
LS-36-N	9.5	114	51.9	48.4	0.932		
LS-37-N	9.5	114	51.9	53.1	1.022		
LS-38-N	9.5	114	51.9	47.0	0.906		
LS-63-N	9.5	114	51.9	46.9	0.903		
LS-64-N	9.5	114	51.9	56.4	1.087		
CMU-1-45°	6.4	49	19.9	8.9	0.446	0.446	-
CMUE-1-45°	6.4	77	19.9	11.9	0.597	0.587	0.013
CMUE-2-45°	6.4	77	19.9	11.5	0.578		
CON2-1-45°	6.4	88	19.9	6.0	0.302	0.303	0.038
CON2-2-45°	6.4	88	19.9	6.0	0.300		
CON2-3-45°	6.4	88	19.9	7.0	0.351		
CON2-4-45°	6.4	88	19.9	5.1	0.258		
CON2-5-45°	6.4	114	19.9	7.5	0.376	0.341	0.033
CON2-6-45°	6.4	114	19.9	6.9	0.345		
CON2-7-45°	6.4	114	19.9	5.9	0.297		
CON2-8-45°	6.4	114	19.9	6.9	0.346		

Table 6.1 (Continued)

Specimen	Diameter (mm)	Embedment (mm)	Nominal Tensile Strength (kN)	Test Result (kN)	Coefficient (α)	Mean of Coefficient	Standard Deviation of Coefficient
CON3-1-45°	6.4	88	19.9	6.3	0.319	0.330	0.087
CON3-2-45°	6.4	88	19.9	9.1	0.455		
CON3-3-45°	6.4	88	19.9	5.3	0.264		
CON3-4-45°	6.4	88	19.9	5.6	0.282		
CON3-5-45°	6.4	114	19.9	6.2	0.314	0.336	0.044
CON3-6-45°	6.4	114	19.9	6.7	0.336		
CON3-7-45°	6.4	114	19.9	7.9	0.397		
CON3-8-45°	6.4	114	19.9	5.9	0.296		
LS-3-45°	6.4	88	19.9	8.7	0.437	0.409	0.040
LS-23-45°	6.4	114	19.9	7.6	0.380		
CON2-13-45°	9.5	114	51.9	15.6	0.301	0.293	0.012
CON2-16-45°	9.5	114	51.9	14.8	0.284		
CON3-12-45°	9.5	88	51.9	25.3	0.487	0.487	-
CON3-13-45°	9.5	114	51.9	20.3	0.390	0.459	0.098
CON3-14-45°	9.5	114	51.9	27.5	0.529		
LS-15-45°	9.5	88	51.9	15.3	0.294	0.332	0.035
LS-16-45°	9.5	88	51.9	18.2	0.351		
LS-17-45°	9.5	88	51.9	16.3	0.313		
LS-18-45°	9.5	88	51.9	19.2	0.370		

inclination, and to decrease the overall bending stress on the rod.

By comparing the values of α in Table 6.1 with those in Figure 6.2, we can observe that the effective length of the moment arm may have been only 2.2 mm for the 6.4 mm anchor and 3.1 mm for the 9.5 mm anchor. These values represent a relatively small moment arm, which seem negligible but in fact has a significant effect on the failure load. Observations during the test revealed that the steel fractured near the top surface of the base material and outside the embedded zone. Some anchors fractured at the exact contact point of the rod and the steel adaptor. This type of failure may be due to

stress concentration. Detailed analysis of stress concentration and the effect of moment arm length on the anchor capacity are outside the scope of this research work, but it should be kept in mind when designing chemical anchoring systems.

Finally based on the result in Table 6.1, the following values of strength factor are recommended:

- (1) 1.0 for vertical anchors
- (2) 0.30 for 6.4 mm anchors at 45-degree inclination in concrete and limestone
- (3) 0.35 for 9.5 mm anchors at 45-degree inclination in concrete and limestone
- (4) 0.50 for 6.4 mm anchors at 45-degree inclination in hollow concrete block.

6.3 Pullout (bond) failure

6.3.1 Analysis of behaviour

In this study, pullout (bond) failure occurred at the interface of the adhesive and the base material or the adhesive and the steel rod. Typically, the anchors exhibiting bond failure resisted tensile load up to their bond strength. Beyond that level, the anchor slipped and eventually pulled out. The residual strength or post-peak resistance was due to the mechanical interlock or roughness of the interface. Some anchor specimens experienced crushing or spalling of base material after bond failed. Pullout failure can be caused by many factors, including inadequate anchor embedment length, improper hole preparation, improper anchor installation and inadequate curing time of epoxy. Of course, many of these deficiencies can be rectified by following proper installation procedures. The issue of adequate embedment length will depend on the properties of the base

material. These properties can be satisfactorily determined through laboratory testing. For concrete and limestone, they will depend on the compressive strength of these materials.

In the case of inclined anchors, the situation is more complex because it involves the shear and bending effects as well as the axial force acting on the anchor at its point of entry into the base material. The applied load has the tendency to bend the anchor rod, which tries to separate it from the base material on the tension side of the anchor rod and cause large compression stresses on the base material on the compression side of the bar. Hence, the effective bond length is continuously diminished until it reaches a critical value, below which the anchor pulls out. The determination of this critical value from a purely theoretical point of view is difficult because it will depend on many factors and a full theoretical analysis is beyond the scope of the current study. However, we could use the present experimental data to assess the effect of the anchor inclination on the pullout load.

6.3.2 Proposed method of calculating failure load based on steel rod pullout

In this study, a bond failure always occurred at the interface of the adhesive and the base material. Cook et al. (1998) proposed a simple design model to predict the static ultimate pullout resistance of adhesively bonded anchors exhibiting bond failure. This model was evaluated by assuming a uniform bond stress distribution along the anchor embedment length. Based on this assumption and the observation that bond strength is affected by the compressive strength of base material and the anchor installation angle, (see Sections 4.3.1 and 4.3.4), the average bond stress τ_{avg} will be related to the

compressive strength of base material given by f'_c and to the anchor slope. Hence, designating the pullout resistance of the anchor under tensile loading by P , it is given by,

$$P = k_1 \tau_{avg} \pi d h_{ef} \quad (6.16)$$

where, P = Pullout (bond) resistance (N)

$$\tau_{avg} = \beta \sqrt{f'_c}, \text{ Average bond strength (MPa)}$$

β = Coefficient for bond strength

f'_c = Compressive strength of base material (MPa)

k_1 = Anchor slope factor

$$= \frac{1}{\cos(\theta)}$$

θ = Angle between the anchor and the normal to the base material surface

d = Diameter of anchor (mm)

h_{ef} = Effective anchor embedment (mm)

Using the current experimental data and Equation 6.16, the average bond strength of the current test specimens can be calculated. Table 6.2 shows the mean value of coefficient β for the different base materials. In design, for vertical anchors, it can be conservatively chosen as 2.0 for concrete and limestone, 1.0 for hollow clay and hollow concrete block. When applying Equation 6.16 to the inclined anchors, the component of the applied load acting along the anchor axis was used as P and the effect of the shear and bending acting on the anchor were ignored. Clearly, these effects can not be ignored; hence their effects are reflected by the bond coefficient β .

Table 6.2: Values of coefficient β for pullout resistance

Specimen	Diameter (mm)	Anchor Depth (mm)	Compressive Strength (MPa)	Inclination Factor (k_1)	Test Result (kN)	Coefficient (β)	Mean of Coefficient	Standard Deviation
CON2-3-N	6.4	88	21.7	1.00	16.4	2.001	2.090	0.388
CON2-4-N	6.4	88	21.7	1.00	14.4	1.757		
CON2-5-N	6.4	114	21.7	1.00	17.6	1.666		
CON2-8-N	6.4	114	21.7	1.00	18.7	1.769		
CON2-12-N	9.5	88	21.7	1.00	34.1	2.776		
CON2-14-N	9.5	114	21.7	1.00	35.6	2.243		
CON2-15-N	9.5	114	21.7	1.00	40.7	2.558		
CON2-17-N	6.4	88	21.7	1.00	15.3	1.875		
CON2-18-N	6.4	88	21.7	1.00	14.9	1.822		
CON2-22-N	6.4	88	21.7	1.00	19.9	2.438		
CON3-13-N	9.5	114	37.3	1.00	50.4	2.418	2.576	0.129
CON3-14-N	9.5	114	37.3	1.00	53.3	2.560		
CON3-15-N	9.5	114	37.3	1.00	56.9	2.733		
CON3-16-N	9.5	114	37.3	1.00	54.0	2.593		
LS-59-N	9.5	114	32.5	1.00	45.7	2.350	2.191	0.167
LS-60-N	9.5	114	32.5	1.00	43.7	2.247		
LS-61-N	9.5	114	32.5	1.00	43.0	2.209		
LS-62-N	9.5	114	32.5	1.00	38.1	1.957		
Br-11-N	9.5	28	38.5	1.00	6.0	1.157	1.251	0.133
Br-15-N	9.5	28	38.5	1.00	7.0	1.345		
CMUE-12-N	9.5	55	4.3	1.00	4.0	1.176	1.544	0.520
CMUE-13-N	9.5	55	4.3	1.00	6.5	1.911		
CON2-9-45°	9.5	88	21.7	1.41	11.7	0.672	0.656	0.020
CON2-12-45°	9.5	88	21.7	1.41	11.0	0.633		
CON2-15-45°	9.5	114	21.7	1.41	14.9	0.661		
CON3-11-45°	9.5	88	37.3	1.41	14.9	0.655	0.655	-
LS-2-45°	6.4	88	32.5	1.41	8.2	0.581	0.661	0.186
LS-4-45°	6.4	88	32.5	1.41	9.1	0.643		
LS-22-45°	6.4	114	32.5	1.41	7.6	0.417		
LS-24-45°	6.4	114	32.5	1.41	8.7	0.475		
LS-31-45°	9.5	114	32.5	1.41	25.3	0.919		
LS-32-45°	9.5	114	32.5	1.41	20.8	0.758		
LS-33-45°	9.5	114	32.5	1.41	23.0	0.835		

Since β is calculated using Equation 6.16, where P is assumed equal to the axial component of the applied load, the lower values of β in the case of the inclined anchors clearly show the important effects of the shear force and bending moment on the strength of the anchor. For practical purposes, β for the inclined anchors in concrete and limestone was found to be approximately 0.6, which is significantly lower than the 2.0 values for the vertically installed anchors in the same materials.

Finally, it should be mentioned that the standard deviation/mean value (coefficient of variation) for the β is not very large and the suggested β values are reasonable.

6.4 Combined cone-bond failure

6.4.1 Analysis of behaviour

The combined cone-bond failure is characterized by pullout of an adhesive anchor attached to a shallow cone of base material. As described in Section 4.2.3, anchors resisted the tensile load up to a critical value, then a failure cone formed and the rod began to slip out. A loud cracking sound usually accompanied the peak load. This kind of failure load mainly depended on the tensile strength of the base material and on the anchor embedment length. The size of pullout cone varied with the base material strength, anchor size and embedment depth. For the combined cone-bond failure mode, the question would be whether the shallow cone failure occurred before, after or at the same time as the bond failure. Answer to this question needs more research. Experimental work also revealed that this failure mode only occurred in vertically installed anchors.

6.4.2 Proposed method of calculating the failure load based on cone-bond failure

The mechanism of idealized cone and bond failure is illustrated in Figure 6.3, which depicts the bonded anchor and the failure cone. The depth of the cone is h_c , while the intact bond length $h_b = h_{ef} - h_c$. Observations during test revealed that the test pullout cone usually exhibited a steep angle near the anchor and became shallower farther from the anchor. For the purposes of analysis, the cone angles may be approximated by an average angle ϕ . Furthermore, we assume that the cone and bond failure occur at the same time, thus the tensile capacity of the anchor (T) is the sum of the pullout cone capacity (T_{cone}) and the adhesive bond capacity (T_{bond}) between the adhesive and concrete. We use the notation T rather than P to differentiate the tensile strength of the rod from its capacity in this mode of failure.

Figure 6.3: Idealized combined cone-bond failure

Some existing design codes provisions can be used to predict the cone resistance under static pullout loading. As presented in Chapter 2, the CEB (CEB, 1994) adopted the Concrete Capacity Design (CCD) method proposed by Fuchs, Eligehausen, and Breen (1995) to predict the cone resistance. This method assumes that the cone resistance can be calculated using the product of the following factors: 1) tensile strength of base material f_t ; 2) effective stress area, A_e , which is defined by the horizontally projected area of the inclined cone surface; and 3) size effect given by $\frac{k_2}{\sqrt{h_c}}$, with k_2 being a coefficient (size effect for cone strength).

In this, following the same basic idea, the ultimate cone resistance (T_{cone}) is calculated using Equation 6.18. In Section 6.3.2, the pullout resistance (T_{bond}) of anchors exhibiting bond failure was given. Hence, substituting for T_{cone} and T_{bond} into Equation 6.17, we obtain the tensile capacity (T) of anchors exhibiting combined cone-bond failure as in Equation 6.20.

$$T_{cone} = f_t A_e \frac{k_2}{\sqrt{h_c}} \quad (6.18)$$

$$T_{bond} = k_1 \pi \tau_{avg} d h_b \quad (6.19)$$

$$T = f_t A_e \frac{k_2}{\sqrt{h_c}} + k_1 \pi \tau_{avg} d h_b \quad (6.20)$$

where T = Ultimate tensile cone-bond capacity (N)

T_{cone} = Ultimate cone capacity (N)

T_{bond} = Ultimate bond capacity (N)

f_t = Tensile strength of base material (MPa)

$= 0.1 f'_c$ for hollow brick, hollow concrete block and concrete

$= 0.22 f'_c$ for limestone

$\tau_{avg} = \beta \sqrt{f'_c}$, Average bond stress (MPa)

f'_c = Compressive strength of base material (MPa)

β = Coefficient for average bond strength

A_e = Projected area of failure cone (mm^2)

$$= \pi h_c \tan \varphi (d + h_c \tan \varphi)$$

h_c = Failure cone depth (mm)

k_2 = Size effect for cone strength

φ = Average cone angle (Deg.)

$k_1 = \frac{1}{\cos(\theta)}$, Anchor inclination factor

θ = Anchor inclination angle measured from the normal to the upper surface of base material and the normal direction of base material surface (Deg.)

d = Diameter of anchor (mm)

h_b = Bond failure depth between cone slip and the end of anchor (mm)

$$= h_{ef} - h_c$$

Experimental results also indicated that the tensile strength of the base material could be taken as approximately 10% of its compressive strength for hollow brick, hollow concrete block and concrete, and 22% for limestone.

Table 6.3 shows the test results for cone-bond failure mode. It is observed that that

- (1) Hollow clay brick has an average value of 34 degrees for cone angle and 3.94 for size effect.
- (2) Hollow concrete block has an average value of 67 degrees for cone angle and 5.1 for size effect.
- (3) 20.6 MPa concrete has an average value of 73 degrees for cone angle and 1.42 for size effect
- (4) 34.5 MPa concrete has an average value of 76 degrees for cone angle and 0.22 for size effect
- (5) Limestone has an average value of 46 degrees for cone angle and 3.79 for size effect.

6.5 Cracking and splitting of base material

6.5.1 Analysis of behaviour

This failure was due to cracking, splitting, local fracture, inelastic deformation and crushing of the base material. In the majority of specimens, the development of crack lines was found to follow a similar pattern. Cracking first became visible when 30-40% of the critical load was applied. As loading increased, additional crack lines formed across the width of specimen and the existing crack lines widened. Typically, at peak load, long and large cracks formed along the threaded rod and the applied load dropped significantly. The residual strength of the anchor was due to the mechanical interlock between the hole surface and the anchor. These cracks were caused by a wedging action.

Table 6.3: Average cone angle ϕ and size effect factor k_2 for combined cone-bond failure

Specimen	Anchor Diameter (mm)	Depth (mm)	Tensile Strength (MPa)	Cone depth (mm)	Cone Angle (Deg.)	Cone Resistance (kN)	Coefficient (β)	Bond Resistance (kN)	Test Result (kN)	Size Effect (k_2)	Average Cone Angle	Average Size Effect (k_2)
CON2-9-N	9.5	88	2.2	40	72	17.6	2.08	13.9	33.6	1.12		
CON2-10-N	9.5	88	2.2	25	72	9.1	2.08	18.3	33.3	1.66	73	1.46
CON2-11-N	9.5	88	2.2	10	76	4.3	2.08	22.6	29.4	1.58		
CON3-9-N	9.5	88	3.7	40	78	68.9	2.57	22.6	47.4	0.36		
CON3-10-N	9.5	88	3.7	22	76	21.6	2.57	31.0	34.8	0.18	76	0.24
CON3-11-N	9.5	88	3.7	15	75	11.1	2.57	34.3	36.4	0.18		
CMU-1-N	6.4	35	0.4	30	70	1.8	1.54	0.3	10.8	5.77		
CMU-2-N	6.4	35	0.4	26	73	2.1	1.54	0.6	11.6	5.33		
CMU-3-N	6.4	35	0.4	27	70	1.6	1.54	0.5	8.3	4.97		
CMU-4-N	6.4	35	0.4	26	72	1.8	1.54	0.6	8.3	4.16		
CMU-11-N	9.5	35	0.4	15	68	0.6	1.54	1.9	4.9	4.81	67	5.10
CMU-12-N	9.5	35	0.4	10	60	0.2	1.54	2.4	3.3	4.43		
CMU-13-N	9.5	35	0.4	12	60	0.2	1.54	2.2	3.4	4.74		
CMU-15-N	9.5	35	0.4	25	69	1.3	1.54	1.0	8.5	5.68		
CMUE-3-N	6.4	55	0.4	35	73	3.2	1.54	1.3	18.8	5.50		
CMUE-11-N	9.5	55	0.4	30	66	1.3	1.54	2.4	9.4	5.46		
CMUE-14-N	9.5	55	0.4	22	57	0.4	1.54	3.2	5.4	5.24		
LS-54-N	9.5	88	7.2	36	43	5.4	2.19	19.4	41.4	4.07		
LS-57-N	9.5	88	7.2	12	45	1.7	2.19	28.3	34.6	3.74	46	3.79
LS-58-N	9.5	88	7.2	30	49	6.2	2.19	21.6	45.7	3.86		
Br-1-N	6.4	28	3.9	18	38	0.8	1.23	1.5	4.9	4.15	34	3.96
Br-3-N	6.4	28	3.9	22	30	0.6	1.23	0.9	3.4	4.03		

Once the bond between the anchor and the adhesive was lost, the pullout load was transferred to the base material by bearing stresses, through the adhesive filled nylon or steel screen. The stresses in the base material comprised longitudinal stress (σ_l) and radial stress (σ_r). The radial stress σ_r caused circumferential tensile stress σ_t , which once greater than the tensile strength of base material (f_t), produced radial cracks. The cracks propagated horizontally and vertically within the base material. Eventually, the anchor pulled out.

The radial stress can be regarded as a uniform internal pressure acting against a thick cylinder having an inner diameter equal to the anchor diameter. As indicated by Tepfers (1973), we can find the radial and tangential stresses in the base material using Equations 6.21 and 6.22, respectively. The greatest tangential stress is given by Equation 6.23 and it occurs at the inner interface of the anchor and base material as shown in Figure 6.4.

Figure 6.4: Development of tensile stress in base material

$$\sigma_r = \frac{r^2 \tau \tan \gamma}{(e+r)^2 - r^2} \left[1 - \frac{(e+r)^2}{R^2} \right] \quad (6.21)$$

$$\sigma_t = \frac{r^2 \tau \tan \gamma}{(e+r)^2 - r^2} \left[1 + \frac{(e+r)^2}{R^2} \right] \quad (6.22)$$

$$(\sigma_t)_{\max} = \frac{\tau \tan \gamma [(e+r)^2 + r^2]}{(e+r)^2 - r^2} \quad (6.23)$$

where σ_r = Radial stress in base material

σ_t = Tangential stress in base material

τ = Bond stress along the anchor

γ = Angle between the principal compressive bond stress and the axis of the anchor

= 45 degree, for a plain bar

r = Radius of anchor

e = Nearest edge distance from anchor to base material

Equation 6.23 clearly indicates that $(\sigma_t)_{\max}$ is dependent on 1) bond stress and 2) on edge distance. As the bond stress increases, the wedging force increases and leads to cracking. Also the smaller the edge distance, the higher the tensile stress in the base material. Tepfers (1973) also suggested that the tangential stress $(\sigma_t)_{\max}$ in the base material decreased rapidly at a short edge distance from the anchor and becomes almost uniform. The finite element results in Chapter 5 verify the above suggestion.

The cracking of the base material could be explained in another way. As shown in Figure 6.5, when anchors are installed near a free edge, both the projected bearing area and the lateral pressure acting on the anchor are concurrently reduced, which result in the

crack initiation on the surface near the closest edge. Due to cracking of the base material around the anchor, the bond area between the anchor and the base material becomes gradually smaller. Finally, crack lines develop and widen and the anchor loses its bond area and fails.

(a) Plan – Effective projected failure area

(b) Section – Effective failure surface

Figure 6.5: Effect of free edge

6.5.2 Proposed method of calculating the failure load based on cracking and splitting of base material

As stated earlier, when the tangential stress σ_t in the base material equals or exceeds the tensile strength of the base material, cracks initiate. Tepfers (1973) suggested that the angle between the principal compressive bond stress and the axis of the rod would be 45 degrees if plain rods were used. In this research, even though the bars were threaded, the failure occurred at the epoxy-concrete interface, which may be treated similar to the interface of a plain bar and base material. Hence, the angle γ will be assumed to be 45 degrees. Based on this assumption, we arrive at Equations 6.24 and 6.25, which can be used to find the minimum bond stress required to cause cracking. Before cracking of the base material, if the bond stress distribution is assumed uniform along the anchor embedment length, then the minimum anchor force required to cause cracking in the base material will be given by Equation 6.26. Cracking and splitting in the base material are the worst design case because they give the lowest anchor strength. Based on the solution proposed by Tepfers (1973) and the numerical analysis presented in Chapter 5, we can safely say that the critical edge distance ($e_{critical}$) to prevent cracking in the base material is equal to 3 times the anchor diameter (d). The current results also indicted that the maximum pullout load (N) for anchors exhibiting cracking failure could be evaluated by the product of a minimum cracking force (N_{crack}) and a factor k_3 .

$$(\sigma_t)_{\max} = \frac{\tau \tan \gamma [(e+r)^2 + r^2]}{(e+r)^2 - r^2} \geq f_t \quad (6.24)$$

$$\tau_{crack} \geq \frac{f_t [(e+r)^2 - r^2]}{(e+r)^2 + r^2} \quad (6.25)$$

$$N_{crack} = \tau_{crack} \pi d h_{ef} \quad (6.26)$$

$$e_{critical} = 3d \quad (6.27)$$

$$N = k_1 k_3 \tau_{crack} \pi d h_{ef} \quad (6.28)$$

where, f_t = Tensile strength of base material (MPa)

$e_{critical}$ = Critical edge distance to prevent cracking of base material (mm)

d = Diameter of anchor (mm)

N_{crack} = Minimum force to cause crack lines in the base material (N)

N = Maximum tensile force for cracking of the base material failure (N)

τ_{crack} = Minimum required bond stress to cause cracks in the base material (MPa)

h_{ef} = Effective anchor embedment length (mm)

k_3 = Strength factor for cracking failure

k_1 = Anchor slope factor

r = Radius of anchor

e = Nearest edge distance from anchor to base material

Table 6.4 shows the values of strength factor (k_3) for cracking failure mode. It is observed that vertical anchors have higher values of strength factor than inclination anchors. The reason is that when anchors are installed with a certain inclination, the bottom part of the anchor can be close to the free edge, which can cause higher cracking bond stress (τ_{crack}). As stated earlier, we can see that the smaller the edge distance, the higher the tensile stress in the base material.

Table 6.4: Value of strength factor k_3 for cracking failure

Specimen	Diameter (MM)	Depth (mm)	Edge (mm)	Compressive Strength (MPa)	Inclination Factor (k_1)	Test Result (kN)	Strength Cracking Factor (k_3)	Average of Cracking Factor	Standard Deviation
LS-53-N	9.5	88	40	32.5	1.0	41.5	2.480	2.441	0.055
LS-56-N	9.5	88	60	32.5	1.0	40.7	2.401		
CON2-19-N	6.4	88	40	21.1	1.0	15.8	4.324	5.039	1.073
CON2-23-N	6.4	114	40	21.1	1.0	16.6	3.502		
CON2-29-N	9.5	88	40	21.1	1.0	31.5	5.791		
CON2-30-N	9.5	88	40	21.1	1.0	27.2	4.999		
CON2-31-N	9.5	88	40	21.1	1.0	29.0	5.331		
CON2-32-N	9.5	88	60	21.1	1.0	32.6	5.936		
CON2-33-N	9.5	88	60	21.1	1.0	35.9	6.527		
CON2-34-N	9.5	88	60	21.1	1.0	26.8	4.871		
CON2-35-N	9.5	114	60	21.1	1.0	34.3	4.814		
CON2-36-N	9.5	114	60	21.1	1.0	42.5	5.960		
CON2-37-N	9.5	114	60	21.1	1.0	45.5	6.388		
CON2-38-N	9.5	114	60	21.1	1.0	25.4	3.565		
CON2-40-N	9.5	114	60	21.1	1.0	25.0	3.505		
CON3-29-N	9.5	88	40	37.3	1.0	33.2	3.453	3.372	0.722
CON3-30-N	9.5	88	40	37.3	1.0	25.7	2.674		
CON3-31-N	9.5	88	40	37.3	1.0	19.9	2.073		
CON3-32-N	9.5	88	60	37.3	1.0	33.2	3.417		
CON3-33-N	9.5	88	60	37.3	1.0	25.0	2.567		
CON3-34-N	9.5	88	60	37.3	1.0	42.4	4.362		
CON3-35-N	9.5	114	40	37.3	1.0	53.7	4.315		
CON3-36-N	9.5	114	40	37.3	1.0	43.4	3.488		
CON3-37-N	9.5	114	40	37.3	1.0	38.5	3.091		
CON3-39-N	9.5	114	60	37.3	1.0	48.2	3.831		
CON3-40-N	9.5	114	60	37.3	1.0	48.2	3.826		
Br-2-N	6.4	28	30	38.5	1.0	5.8	2.742	2.037	0.449
Br-4-N	6.4	28	30	38.5	1.0	3.5	1.667		
Br-5-N	6.4	28	30	38.5	1.0	6.3	2.976		
Br-12-N	9.5	28	30	38.5	1.0	6.2	1.990		
Br-13-N	9.5	28	30	38.5	1.0	6.6	2.126		
Br-14-N	9.5	28	30	38.5	1.0	7.0	2.235		
Br-H-1-N	6.4	63	30	38.5	1.0	8.5	1.790		
Br-H-2-N	6.4	63	30	38.5	1.0	7.9	1.655		
Br-H-3-N	6.4	63	30	38.5	1.0	8.2	1.719		
Br-H-4-N	6.4	63	30	38.5	1.0	7.7	1.616		
Br-H-5-N	6.4	63	30	38.5	1.0	9.0	1.889		
Br-H-11-N	9.5	63	30	38.5	1.0	12.7	1.815		
Br-H-12-N	9.5	63	30	38.5	1.0	17.9	2.557		
Br-H-13-N	9.5	63	30	38.5	1.0	8.7	1.243		
Br-H-14-N	9.5	63	30	38.5	1.0	14.3	2.045		
Br-H-15-N	9.5	63	30	38.5	1.0	13.2	1.890		

Table 6.4 (Continued)

Specimen	Diameter (MM)	Depth (mm)	Edge (mm)	Compressive Strength (MPa)	Inclination Factor (k_1)	Test Result (kN)	Strength Cracking Factor (k_3)	Average of Splitting Factor	Standard Deviation
CMU-5-N	6.4	35	95	4.3	1.0	8.1	26.822	27.358	4.714
CMU-14-N	9.5	35	95	4.3	1.0	10.1	22.427		
CMUE-1-N	6.4	55	95	4.3	1.0	11.2	23.534		
CMUE-2-N	6.4	55	95	4.3	1.0	14.3	30.196		
CMUE-4-N	6.4	55	95	4.3	1.0	16.0	33.808		
CON2-10-45°	9.5	88	62	21.1	1.4	15.7	2.020	2.016	0.263
CON2-11-45°	9.5	88	62	21.1	1.4	16.7	2.168		
CON2-10-45°	9.5	114	81	21.1	1.4	18.6	1.860		
CON3-14-45°	9.5	88	62	37.3	1.4	23.7	1.743		
CON3-15-45°	9.5	114	81	37.3	1.4	26.3	1.484	1.597	0.388
CON3-16-45°	9.5	114	81	37.3	1.4	27.7	1.565		
Br-1-45°	6.4	40	30	38.5	1.4	3.8	0.899		
Br-2-45°	6.4	40	30	38.5	1.4	4.0	0.943	0.875	0.103
Br-3-45°	6.4	40	30	38.5	1.4	3.7	0.879		
Br-4-45°	6.4	40	30	38.5	1.4	3.4	0.798		
Br-5-45°	6.4	40	30	38.5	1.4	4.2	0.991		
Br-21-45°	9.5	40	30	38.5	1.4	5.2	0.836		
Br-22-45°	9.5	40	30	38.5	1.4	6.1	0.976		
Br-24-45°	9.5	40	30	38.5	1.4	4.2	0.682		
CMU-2-45°	6.4	49	95	4.3	1.4	7.6	12.992	9.656	2.797
CMU-3-45°	6.4	49	95	4.3	1.4	7.8	13.227		
CMU-4-45°	6.4	49	95	4.3	1.4	7.7	13.129		
CMU-5-45°	6.4	49	95	4.3	1.4	7.7	13.160		
CMU-11-45°	9.5	49	95	4.3	1.4	8.3	9.407		
CMU-13-45°	9.5	49	95	4.3	1.4	5.4	6.142		
CMU-14-45°	9.5	49	95	4.3	1.4	8.1	9.177		
CMU-15-45°	9.5	49	95	4.3	1.4	7.2	8.205		
CMUE-3-45°	6.4	77	95	4.3	1.4	9.8	10.625		
CMUE-4-45°	6.4	77	95	4.3	1.4	10.8	11.661		
CMUE-11-45°	9.5	77	95	4.3	1.4	10.2	7.405		
CMUE-12-45°	9.5	77	95	4.3	1.4	8.8	6.354		
CMUE-13-45°	9.5	77	95	4.3	1.4	10.8	7.852		
CMUE-14-45°	9.5	77	95	4.3	1.4	8.1	5.843		

6.6 Predicted versus measured strength of the tested anchors

The ultimate strengths of all the test specimens, irrespective of their observed modes of failure, are calculated in accordance with the proposed methods discussed in Sections 6.2 to 6.6. The predicted value is assumed to be the smallest of the six failure loads corresponding to the six failure modes. The calculated values are plotted in Figure 6.6 against their values from the tests. It can be observed that the calculated values are in close agreement with the test results. For the 158 test specimens, not including cracking failure mode, the average ratio of the calculated strengths to the test results is 0.961, with a standard deviation of 0.157. For the 75 test specimens exhibiting cracking failure mode, the average ratio of the calculated strengths to test values is 0.997, with a standard deviation of 0.249. Due to the nature of cracking failure mechanism, it has a higher standard deviation. It is recognized that the relatively good comparison in Figure 6.6 is not entirely unexpected because the strength of the anchors with different failure modes contain correction factors that were derived from the present test data. The only true test for ascertaining the validity of these equations is to apply them to other test results in the future.

Figure 6.6: Overall comparison of test results with predicted ultimate loads

6.7 Proposed general design method for adhesive anchors

Based on the analysis and discussion in this chapter, the tensile capacity of adhesively bonded anchors can be calculated using Equation (6.29), which indicates that the capacity of an anchor is the smallest of the following four forces: (1) the tensile strength of anchor steel (F); (2) the pullout resistance (P); (3) the combined cone-bond strength (T) and (4) the maximum cracking force (N).

$$\text{Anchor capacity} = \text{Minimum of } \psi.(F, P, T, N) \quad (6.29)$$

where $F = \alpha f_u A_s$

$$P = k_1 \tau_{avg} \pi d h_{ef}$$

$$T = f_t A_e \frac{k_2}{\sqrt{h_c}} + k_1 \pi \tau_{avg} d h_b$$

$$N = k_1 k_3 \tau_{crack} \pi d h_{ef}$$

A_s = Effective area of anchor (mm^2)

F = Applied tensile force (N)

α = Factor for anchor steel strength

= 1.0, for anchors with vertical embedment

= 0.30, for the 6.4 mm diameter anchors with 45-degree inclined embedment in concrete and limestone

= 0.35, for the 9.5 mm diameter anchors with 45-degree inclined embedment in concrete and limestone

= 0.50, for the 6.4 mm anchors with 45-degree inclination in hollow concrete block

$$\tau_{avg} = \beta \sqrt{f_c} \quad \text{Average bond strength (MPa)}$$

β = Coefficient for bond strength

= 2.0 for concrete and limestone, vertical embedment

= 1.0 for hollow clay brick and hollow concrete block, vertical embedment

= 0.60 for concrete and limestone, 45-degree inclined embedment

f'_c = Compressive strength of base material (MPa)

$$k_1 = \frac{1}{\cos(\theta)}, \text{ Anchor inclination factor}$$

θ = Anchor inclination angle measured from the normal to the base material surface (Deg.)

d = Diameter of anchor (mm)

h_{ef} = Effective anchor embedment (mm)

f_t = Tensile strength of base material (MPa)
 $= 0.1 f'_c$ for hollow brick, hollow concrete block and concrete
 $= 0.22 f'_c$ for limestone

A_e = Projected area of failure cone (mm²)
 $= \pi h_c \tan \varphi (d + h_c \tan \varphi)$

h_c = Failure cone depth (mm)

φ = Average cone angle (Deg.)
 $= 73$ degree for 20.6MPa concrete
 $= 76$ degree for 34.5MPa concrete
 $= 67$ degree for hollow concrete block
 $= 46$ degree for limestone
 $= 34$ degree for hollow brick

k_2 = Size effect for cone strength
 $= 1.42$ for 20.6MPa concrete
 $= 0.22$ for 34.5MPa concrete
 $= 5.10$ for hollow concrete block
 $= 3.80$ for limestone
 $= 4.00$ for hollow brick

h_b = Effective bond length in cone-bond failure mode (mm)
 $= h_{ef} - h_c$

τ_{crack} = Minimum required bond stress to cause crack in base material (MPa)

$$= \frac{f_t [(e+r)^2 - r^2]}{(e+r)^2 + r^2}$$

r = Radius of anchor (mm)

e = Smallest edge distance from anchor to base material edge (mm)

$e_{critical}$ = Critical edge distance to prevent cracking of base material (mm)
 $= 3.d$

k_3 = Strength factor for cracking failure

- = 2.0 for limestone, vertical embedment
- = 5.0 for 20.6MPa concrete, vertical embedment
- = 3.0 for 34.5MPa concrete, vertical embedment
- = 2.0 for hollow brick, vertical embedment length
- = 25.0 for hollow concrete block, vertical embedment
- = 2.0 for 20.6 MPa concrete, 45-degree embedment
- = 1.5 for 24.5 MPa concrete, 45-degree embedment
- = 0.8 for hollow brick, 45-degree embedment
- = 9.0 for hollow concrete block, 45-degree embedment

ψ = Strength reduction factor, dependent on prevailing design safety requirements or codes

$\psi = 0.7$ based on the American design philosophy would be a reasonable value. If

the Canadian limit states design philosophy is applied, then one needs to apply a so-called ϕ factor to each material. For concrete $\phi_c = 0.6$ while for steel, it may be assumed equal to $\phi_s = 0.85$. For the epoxy, one may use a factor similar to concrete, that is $\phi_e = 0.6$.

If ϕ factors are used, then $\psi = 1.0$.

6.8 Comparison of test results with existing design methods

For anchors that are fully embedded in a base material, the Eligehausen (1994) method presented in Section 2.6, the CEB method (1994) presented in Section 2.7.1 and the JCI method (1994) in Section 2.7.2 may be applied. Table 6.5 shows the predictions of these methods and the corresponding experimental results for the relevant anchors in the present testing program. In this table P_u is the pullout load predicted by the Eligehausen method, N_u is the pullout load according to the CEB method and T_1 , T_2 and T_3 are the steel fracture, cone failure loads and pullout strength, respectively base on the JCI method. Hence, for the JCI method the design failure load would be the smallest of the three values given by T_1 , T_2 and T_3 . This means that in Table 6.5, where the average experimental peak load (i.e. anchor strength) is expressed in forms of the predicted strength values; one must select the largest value from this table as the predicted strength. The selected value is shown in bold form. If the largest values are less than 1.0, then method gives unconservative prediction.

We observe that the Eligehausen and CEB methods consistently overestimate the capacity of the current test specimens. The difference between the observed and predicted results is quite large and it would not be safe to use these equations. The JCI method results, albeit unconservative in some cases, are much better and they compare reasonably in the current experimental work. Other failure modes were observed that are not recognized by any of the current design methods. Therefore, a better and more comprehensive design method is needed. The proposed design method in the current study is an attempt in that direction.

Table 6.6: Comparisons of existing design methods of adhesive anchor with experimental results

Test Specimen	Anchor Diameter (mm)	Anchor Depth (mm)	Base Material Compressive Strength (MPa)	Average Peak Load (kN)	Average Peak Load P_u	Average Peak Load N_u	Average Peak Load T_1	Average Peak Load T_2	Average Peak Load T_3
CON2-1,2,3,4-N	6.4	88	21.1	16.2	0.484	0.287	0.811	0.575	0.910
CON2-5,6,7,8-N	6.4	114	21.1	18.5	0.334	0.225	0.926	0.403	0.808
CON2-9,10,11,12-N	9.5	88	21.1	32.6	0.976	0.578	0.628	1.123	1.222
CON2-13,14,15,16-N	9.5	114	21.1	38.4	0.696	0.467	0.740	0.818	1.120
CON3-1,2,3,4-N	6.4	88	37.3	17.6	0.395	0.234	0.880	0.470	0.743
CON3-5,6,7,8-N	6.4	114	37.3	18.5	0.252	0.169	0.928	0.304	0.609
CON3-9,10,11,12-N	9.5	88	37.3	40.4	0.910	0.539	0.778	1.046	1.139
CON3-13,14,15,16-N	9.5	114	37.3	54.8	0.746	0.501	1.055	0.877	1.201
LS-1,2,3,4-N	6.4	88	32.5	17.8	0.428	0.254	0.890	0.509	0.805
LS-15,16,17,18-N	9.5	114	32.5	48.3	0.704	0.473	0.930	0.828	1.134
LS-21,22,23,24-N	6.4	88	32.5	18.8	0.454	0.269	0.943	0.539	0.852
LS-35,36,37,38-N	9.5	114	32.5	49.0	0.715	0.480	0.945	0.841	1.152
Mean					0.591	0.373	0.871	0.694	0.974
Standard Deviation					0.231	0.145	0.114	0.262	0.209

Chapter 7

Summary, Conclusions and Recommendations

7.1 Summary

In this study 234 anchors were tested in tension to determine the effect of various parameters on the behaviour, strength and failure mode of adhesively bonded anchors. Two different anchor diameters and embedment lengths were used. The anchors were installed in concrete cylinders with either 20.6 MPa or 34.5 MPa compressive strength, in limestone blocks, in hollow clay bricks and in hollow concrete masonry blocks. In the case of hollow clay brick and masonry blocks, nylon or stainless steel screens were used to provide containment of the epoxy during installation. In addition, some of the anchors were installed at an inclination of 45 degree and were loaded with vertical tension force acting on the anchor free end. The anchor systems used, including the epoxy, are proprietary products.

The tests were performed by applying the load statically and monitoring the anchor displacement using a LVDT. The results showed that the anchor capacity could vary greatly depending on its mode of failure. The major modes of failure include the fracture of the steel anchor rod, pullout due to loss of bond, cone-bond failure (i.e. combined loss of bond and fracture of the base material, and splitting of the base material). The splitting and fracturing of the base material and its location with respect to the anchor rod depend on the base material and the geometry of the anchor system. The

inclination of the anchor led to a drastic reduction in its capacity. Anchors installed in the high strength concrete and limestone yielded the highest strength, provided they had adequate embedment. The provision of the screens increased the capacity of anchors in hollow clay brick and concrete masonry, but it also caused the brittle splitting failure, particularly in the clay brick masonry, where the brick webs fractured due to high tension.

The observed failure modes can be explained by theoretical analyses, but one cannot use theory alone to predict the capacity of the anchors in all cases. To be able to predict the anchor capacity in each case, a method of design is proposed. This method, which uses some design coefficients derived from the present test results, needs to be verified by either additional tests or by available data for other anchors in the literature.

7.2 Conclusions

The current experimental and theoretical study supports the following conclusions:

- (1) Six different basic modes of failure are possible for the adhesively bonded anchors in concrete and masonry. They are anchor steel fracture, pullout (bond) failure, combined cone-bond failure, cracking and splitting of base material, breakout cone failure and brick web failure.
- (2) The capacity of adhesively bonded anchors generally increases with embedment length; consequently, it is advantageous to use longer embedment length.

- (3) The larger anchor diameter contributes to a higher pullout strength. However, this effect was not observed for the 9.5 mm diameter anchors installed in hollow clay bricks and hollow concrete blocks. In the latter case, the use of the nylon screen adhesively affected the bond between the adhesive and the base material.
- (4) Anchor inclination has a significant effect on the behaviour and strength of adhesively bonded anchors. Anchor slip increases with deviation from the vertical load, while its failure load decreases. The reason is that the 45-degree inclined anchors are subjected to a combination of tension, shear and bending, which create higher normal and shear stresses within the anchor rod, the base material and their interface.
- (5) The concrete compressive strength affects the pullout strength and failure mode of adhesively bonded anchors. Using higher concrete strength increases the bond strength, reduces the required embedment length and prevents failure by splitting.
- (6) The edge distance significantly affects the strength of adhesively bonded anchors and most test specimens with anchors close to the edge experienced cracking and splitting of the base material. However, once the edge distance exceeds the critical value, edge distance has no influence on the performance of the anchor. The magnitude of the critical edge distance depends on embedment depth, anchor diameter and the tensile strength of the base material.

- (7) Performance of adhesively bonded anchors is critically dependent on the quality of the bond between the adhesive and the base material on the one hand, and between the adhesive and anchor steel on the other hand. The pullout bond stress varies with base material, anchor diameter, embedment length and characteristic strength of the adhesive.
- (8) Failure by steel fracture will be achieved if pullout and cracking of the base material can be prevented. For this failure mode, for vertically installed anchors, their strength can be calculated as being equal to the nominal tensile strength of the steel rod. For the inclined anchors, even when steel fracture happens to be the failure mode, the anchor capacity still depends on the base material.
- (9) Adhesively bonded anchors with sufficient embedment length do not fail in the form of cone radiating outward from the base of the anchor. Instead, a partial cone forms and the cone size varies with the tensile strength of the base material. The pullout loading of combined cone-bond failure can be predicted by a method proposed in the current study. It involves assuming the tensile strength of the base material and a uniform bond stress distribution along the intact portion of the embedment length. Results from the proposed method agree well with experimental data obtained in this study.
- (10) Cracking and splitting of the base material are caused by a wedging action. Cracks initiate when the radial stress at the epoxy-concrete interface causes sufficient circumferential tensile stresses exceed the tensile strength of the

base material. A rational method based on the test results and analysis can be used to predict the cracking load.

- (11) The numerical analysis suggests that the concrete failure mode initiates at the concrete-adhesive interface and propagates vertically and laterally within the base material.
- (12) The proposed design method, which takes into account all the critical failure modes, can be used to predict the current test anchors capacity.
- (13) In designing an anchor system, one must avoid installing anchors in the proximity of free edges and must make sure that they have adequate embedment length. The required embedment length will depend on the average bond strength of the adhesive-base material interface, which is strongly affected by the compressive strength of the base material. The higher the compressive strength of the material, the smaller the required embedment length. Of course, the lower limit of the embedment depth will be determined by the shear strength of the adhesive or the bond strength of the adhesive-anchor interface. Generally, however, for concrete and masonry, it is the strength of the base material which governs embedment length.
- (14) The effect of nylon and stainless steel screens on the ultimate capacity of the anchor system is not clear. It is not appropriate to assume that these screens can compensate for inadequate embedment in the base material.

7.3 Recommendations for future work

Based on the results of the present research, the following recommendations are made for further investigation.

- (1) To determine the critical edge distance, additional tests are needed. The effect of critical edge distance should be studied in combinations with embedment length.
- (2) Since in the present study anchor inclination led to a dramatic reduction in anchor strength in both concrete and limestone, it is recommended to study other anchor inclinations to fully quantify the effect of this parameter on an anchor strength and deformation.
- (3) In the case of inclined anchors, it might be advantageous to use a steel washer epoxied to the surface of the base material. The washer may relieve the high stresses in the base material caused by the bending of the steel rod. The problem of blunting the stresses at the points of entry of the anchor into the base material requires further examination.
- (4) The observed failure modes were affected by a number of parameters, but the main factors which determine any failure mode are the stress combinations and level, and the strength of the base material and the anchor rod. The stress distribution in the base material and the magnitude of the stresses depend on the stress-strain relationship of the epoxy and the base material and on their interface properties. The effect of these parameters can be more accurately assessed by the performance of detailed nonlinear finite element analyses. Therefore, it is strongly recommended that full three-

dimensional non-linear finite element analyses be performed to assess the redistribution of stresses due to changes in material and interface stiffness with applied loads and to determine the distribution of stress at ultimate and its effect on the anchor failure mode. Such analyses can give great insight into the behaviour of adhesive anchors.

- (5) Since most anchor systems are proprietary, it is recommended that anchor systems which use different resin/anchor rod combinations be tested for developing a complete data base and more robust design guidelines.
- (6) It is not evident whether epoxy is the most appropriate adhesive for anchors installed in hollow clay brick. A less rigid adhesive with adequate strength may be more appropriate to avoid splitting failure. This problem needs further investigation.
- (7) The metal and nylon screens used for adhesive anchors in hollow brick and concrete may not be the ideal system because they tend to cause splitting failure at very low loads. It is desirable to investigate alternative techniques of anchoring in hollow media in order to achieve higher strength and lower displacement.

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Appendix A

Property of epoxy

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LABORATORY REPORT

125 Swanson Street Ottawa, IL 61350 (815) 433-1495 Fax (815) 433-1795

**Advanced
Materials
Center, Inc.**

To: ITW Ramset / Red Head
Mr. Chris LaVine

Date: November 14, 2002
Project : 02 P 1301
PO# CLLAB1002

Purpose:

Evaluate cartridge mixed New G-5 Epoxy adhesive for conformance to ASTM C 881-99, Type IV (Grade 3) specifications for properties.

Sample Identification:

A. New G-5 Epoxy adhesive, 22 oz dual cartridge
B. E 102 Injection Tool

Source:

ITW Ramset / Red Head
ITW Ramset / Red Head

Conclusions:

In our opinion, the New G-5 Epoxy adhesive conforms or exceeds ASTM C881-99, Type IV, Grade 3 requirements except for tensile strength.

Results:

<u>Test Method:</u>	<u>Spec. Value:</u>	<u>New G-5 Value:</u>
Viscosity, ASTM D2393		111 Pa.s (initial mix) 23°C 55.2 Pa.s (11 minutes) 33°C
Consistency	1/4 inch - max	0.0 inches (no sag)
Gel Time (60g) (Peak Exotherm °C) 30 min		15.0 minutes (103.75 °C)
(80g) (Peak Exotherm °C) 30 min		14.5 minutes (131.25 °C)
Bond Strength, ASTM C39, C882	1000 psi 1500 psi	>3330 psi (2 day) >3330 psi (14 day)
Absorption, ASTM D 570, 24 hr	<1% max	0.036%
Heat Deflection (TMA- 7 day)	50 °C (120 ° F)	62 °C (143.6 ° F)
Shrinkage, ASTM D 2566(max)	0.005	0.003
Tensile Strength, ASTM D 638	7000 psi (7day) 1% Elong.(7day)	4,355 psi (7day) 7.56 % elongation (7day)
Compression, ASTM D 695	10,000 psi (7day)	10,344 psi (7 day)
Compressive Modulus	200,000 (7 day)	665,251 (7 day)

Appendix B

Results for all anchor specimens

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
1	Br-1-N	6.4	28	30	4.9	0.4	0.4	4.5	Combined Cone-Bond	4.8	1.30	0.5	0.17
2	Br-2-N				5.8	0.4	5.8	0.4	Splitting and Cracking				
3	Br-3-N				3.4	0.4	3.0	5.9	Combined Cone-Bond				
4	Br-4-N				3.5	0.639	8.0	0.5	Splitting and Cracking				
5	Br-5-N				6.3	0.8	6.3	0.8	Splitting and Cracking				
6	Br-11-N	9.5	28	30	6.0	0.1	1.0	5.6	Pullout (Bond)	6.6	0.44	0.4	0.24
7	Br-12-N				6.2	0.4	5.3	0.5	Splitting and Cracking				
8	Br-13-N				6.6	0.2	1.0	2.8	Splitting and Cracking				
9	Br-14-N				7.0	0.8	7.0	0.8	Splitting and Cracking				
10	Br-15-N				7.0	0.4	0.7	4.8	Pullout (Bond)				
11	Br-1-45°	6.4	40	30	3.8	4.6	5.4	4.6	Splitting and Cracking	3.8	0.31	2.4	1.46
12	Br-2-45°				4.0	1.0	0.0	0.0	Splitting and Cracking				
13	Br-3-45°				3.7	1.8	3.7	1.8	Splitting and Cracking				
14	Br-4-45°				3.4	3.2	2.7	3.4	Splitting and Cracking				
15	Br-5-45°				4.2	1.7	4.2	1.8	Splitting and Cracking				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
16	Br-21-45°	9.5	40	30	5.2	2.2	0.0	17.2	Splitting and Cracking	4.9	0.76	3.1	1.15
17	Br-22-45°				6.1	4.5	2.6	10.8	Splitting and Cracking				
18	Br-23-45°				4.3	1.8	3.4	4.9	Brick Web				
19	Br-24-45°				4.2	3.9	1.0	8.9	Splitting and Cracking				
20	Br-25-45°				4.6	3.0	4.6	3.0	Brick Web				
21	Br-H-1-N	6.4	63	30	8.5	1.3	0.2	3.0	Splitting and Cracking	8.2	0.52	1.1	0.84
22	Br-H-2-N				7.9	2.4	0.1	4.3	Splitting and Cracking				
23	Br-H-3-N				8.2	0.3	5.0	0.7	Splitting and Cracking				
24	Br-H-4-N				7.7	1.1	5.6	3.1	Splitting and Cracking				
25	Br-H-5-N				9.0	0.5	0.0	1.9	Splitting and Cracking				
26	Br-H-11-N	9.5	63	30	12.7	0.4	2.1	2.5	Splitting and Cracking	13.4	3.30	0.5	0.15
27	Br-H-12-N				17.9	0.7	3.1	3.8	Splitting and Cracking				
28	Br-H-13-N				8.7	0.6	0.2	3.3	Splitting and Cracking				
29	Br-H-14-N				14.3	0.6	0.7	2.5	Splitting and Cracking				
30	Br-H-15-N				13.2	0.4	8.5	1.3	Splitting and Cracking				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
31	CMU-1-N	6.4	35	—	10.8	1.4	5.5	5.2	Combined Cone-Bond	9.4	1.67	0.8	0.35
32	CMU-2-N				11.6	0.7	4.5	5.8	Combined Cone-Bond				
33	CMU-3-N				8.3	0.8	0.0	3.2	Combined Cone-Bond				
34	CMU-4-N				8.3	0.4	0.0	2.7	Combined Cone-Bond				
35	CMU-5-N				8.1	0.9	0.0	1.5	Splitting and Cracking				
36	CMU-11-N	9.5	35	—	4.9	0.5	2.2	5.7	Combined Cone-Bond	6.0	3.11	1.4	2.17
37	CMU-12-N				3.3	0.3	1.5	5.6	Combined Cone-Bond				
38	CMU-13-N				3.4	0.2	0.3	1.0	Combined Cone-Bond				
39	CMU-14-N				10.1	5.2	8.4	5.9	Splitting and Cracking				
40	CMU-15-N				8.5	0.5	0.0	2.4	Combined Cone-Bond				
41	CMU-1-45°	6.4	49	—	8.9	8.8	6.2	10.3	Steel Fracture	7.9	0.52	8.3	2.13
42	CMU-2-45°				7.6	11.4	7.6	11.4	Splitting and Cracking				
43	CMU-3-45°				7.8	8.2	7.8	8.2	Splitting and Cracking				
44	CMU-4-45°				7.7	5.5	7.7	5.5	Splitting and Cracking				
45	CMU-5-45°				7.7	7.8	7.7	7.8	Splitting and Cracking				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
46	CMU-11-45°	9.5	49	-	8.3	6.2	8.0	7.8	Splitting and Cracking	7.9	1.79	6.4	5.43
47	CMU-12-45°				10.4	15.5	5.9	20.5	Pullout (Bond)				
48	CMU-13-45°				5.4	2.5	1.3	3.8	Splitting and Cracking				
49	CMU-14-45°				8.1	5.7	8.1	5.7	Splitting and Cracking				
50	CMU-15-45°				7.2	2.0	7.2	2.0	Splitting and Cracking				
51	CMUE-1-N	6.4	55	-	11.2	0.5	11.2	0.5	Splitting and Cracking	15.1	3.19	0.8	0.47
52	CMUE-2-N				14.3	0.9	14.3	0.9	Splitting and Cracking				
53	CMUE-3-N				18.8	1.4	8.8	4.1	Combined Cone-Bond				
54	CMUE-4-N				16.0	0.3	16.0	0.3	Splitting and Cracking				
55	CMUE-11-N	9.5	55	-	9.4	0.2	1.8	7.1	Combined Cone-Bond	6.1	2.49	6.1	7.11
56	CMUE-12-N				4.0	15.2	1.7	18.6	Pullout (Bond)				
57	CMUE-13-N				6.5	8.1	5.1	19.5	Pullout (Bond)				
58	CMUE-14-N				4.4	0.7	2.1	12.6	Combined Cone-Bond				
59	CMUE-1-45°	6.4	77	-	11.9	11.9	11.9	11.9	Steel Fracture	11.0	0.91	13.0	1.77
60	CMUE-2-45°				11.5	12.7	11.5	12.7	Steel Fracture				
61	CMUE-3-45°				9.8	15.6	9.8	15.6	Splitting and Cracking				
62	CMUE-4-45°				10.8	12.0	10.8	12.0	Splitting and Cracking				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
63	CMUE-11-45°	9.5	77	-	10.2	8.6	10.2	8.6	Splitting and Cracking	9.5	1.28	6.4	2.46
64	CMUE-12-45°				8.8	6.6	8.8	6.6	Splitting and Cracking				
65	CMUE-13-45°				10.8	7.3	10.8	7.3	Splitting and Cracking				
66	CMUE-14-45°				8.1	2.9	8.1	2.9	Splitting and Cracking				
67	CON2-1-N	6.4	88	-	17.6	1.9	13.7	2.9	Steel Fracture	16.2	1.33	3.5	2.35
68	CON2-2-N				16.4	6.9	13.4	10.4	Steel Fracture				
69	CON2-3-N				16.4	3.1	0.0	17.3	Pullout (Bond)				
70	CON2-4-N				14.4	2.0	0.0	14.1	Pullout (Bond)				
71	CON2-5-N	6.4	114	-	17.6	4.4	6.4	16.3	Pullout (Bond)	18.5	0.59	4.4	1.16
72	CON2-6-N				19.0	2.9	18.8	3.4	Steel Fracture				
73	CON2-7-N				18.6	5.7	7.0	14.9	Steel Fracture				
74	CON2-8-N				18.7	4.7	7.6	14.4	Pullout (Bond)				
75	CON2-9-N	9.5	88	-	33.6	1.8	0.0	14.5	Combined Cone-Bond	32.6	2.14	2.3	0.55
76	CON2-10-N				33.3	2.2	9.6	17.7	Combined Cone-Bond				
77	CON2-11-N				29.4	2.1	3.7	17.1	Combined Cone-Bond				
78	CON2-12-N				34.1	3.1	11.1	16.3	Pullout (Bond)				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
79	CON2-13-N	9.5	114	-	40.8	1.1	40.6	1.2	Steel Fracture	38.4	2.71	2.0	1.25
80	CON2-14-N				35.6	3.8	15.0	17.1	Pullout (Bond)				
81	CON2-15-N				40.7	2.1	4.8	17.6	Pullout (Bond)				
82	CON2-16-N				36.5	1.1	36.1	1.2	Steel Fracture				
83	CON2-1-45°	6.4	88	-	6.0	16.8	4.6	17.4	Steel Fracture	6.0	0.75	17.7	1.13
84	CON2-2-45°				6.0	16.8	5.8	16.9	Steel Fracture				
85	CON2-3-45°				7.0	19.1	5.7	19.4	Steel Fracture				
86	CON2-4-45°				5.1	18.1	3.9	18.1	Steel Fracture				
87	CON2-5-45°	6.4	114	-	7.5	9.5	6.1	10.9	Steel Fracture	6.8	0.65	10.9	1.46
88	CON2-6-45°				6.9	9.7	4.9	11.5	Steel Fracture				
89	CON2-7-45°				5.9	12.2	3.9	13.2	Steel Fracture				
90	CON2-8-45°				6.9	12.1	5.8	12.9	Steel Fracture				
91	CON2-9-45°	9.5	88	-	11.7	1.7	9.0	2.1	Pullout (Bond)	13.8	2.85	10.8	6.63
92	CON2-10-45°				15.7	17.0	15.7	17.0	Splitting and Cracking				
93	CON2-11-45°				16.7	14.2	16.7	14.2	Splitting and Cracking				
94	CON2-12-45°				11.0	10.3	4.6	16.0	Pullout (Bond)				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
95	CON2-13-45°	9.5	114	-	15.6	13.5	9.6	15.3	Steel Fracture	16.0	1.82	15.0	3.38
96	CON2-14-45°				18.6	19.2	16.1	19.8	Splitting and Cracking				
97	CON2-15-45°				14.9	11.3	11.0	17.7	Pullout (Bond)				
98	CON2-16-45°				14.8	16.0	14.8	16.0	Steel Fracture				
99	CON2-17-N	6.4	88	40	15.3	3.3	8.9	13.8	Pullout (Bond)	15.4	0.48	3.2	1.21
100	CON2-18-N				14.9	4.3	9.5	9.8	Pullout (Bond)				
101	CON2-19-N				15.8	1.9	4.0	10.3	Splitting and Cracking				
102	CON2-20-N	6.4	88	60	17.3	3.9	14.1	4.0	Steel Fracture	18.8	1.33	3.3	0.51
103	CON2-21-N				19.2	3.3	16.2	3.3	Steel Fracture				
104	CON2-22-N				19.9	2.8	13.9	11.9	Pullout (Bond)				
105	CON2-23-N	6.4	114	40	16.6	1.8	4.9	7.9	Splitting and Cracking	17.8	2.57	1.9	0.33
106	CON2-24-N				16.0	1.6	11.5	1.7	Steel Fracture				
107	CON2-25-N				20.7	2.3	20.7	2.4	Steel Fracture				
108	CON2-26-N	6.4	114	60	19.5	2.0	16.4	2.1	Steel Fracture	19.5	1.17	3.0	1.19
109	CON2-27-N				18.3	2.9	15.1	3.0	Steel Fracture				
110	CON2-28-N				20.6	4.3	20.4	4.6	Steel Fracture				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
111	CON2-29-N	9.5	88	40	31.5	1.3	30.0	1.4	Splitting and Cracking	29.2	2.16	1.9	0.78
112	CON2-30-N				27.2	1.6	3.7	5.3	Splitting and Cracking				
113	CON2-31-N				29.0	2.8	26.9	3.0	Breakout Cone				
114	CON2-32-N	9.5	88	60	32.6	2.3	1.4	8.8	Splitting and Cracking	31.8	4.62	2.5	1.21
115	CON2-33-N				35.9	3.9	13.3	13.9	Splitting and Cracking				
116	CON2-34-N				26.8	1.5	3.6	13.3	Splitting and Cracking				
117	CON2-35-N	9.5	114	40	34.3	2.6	13.7	7.6	Splitting and Cracking	40.8	5.80	2.5	1.01
118	CON2-36-N				42.5	3.4	39.5	3.6	Splitting and Cracking				
119	CON2-37-N				45.5	1.4	3.7	21.0	Splitting and Cracking				
120	CON2-38-N	9.5	114	60	50.4	2.4	18.6	21.6	Splitting and Cracking	41.5	14.34	2.0	0.42
121	CON2-39-N				49.2	1.7	39.4	4.3	Steel Fracture				
122	CON2-40-N				25.0	1.8	10.6	6.8	Splitting and Cracking				
123	CON3-1-N	6.4	88	-	17.4	2.1	13.6	2.9	Steel Fracture	17.6	0.69	4.2	2.54
124	CON3-2-N				18.1	6.9	14.7	9.0	Steel Fracture				
125	CON3-3-N				16.6	5.9	16.5	6.1	Steel Fracture				
126	CON3-4-N				18.1	2.1	12.7	2.9	Steel Fracture				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm))	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
127	CON3-5-N	6.4	114	-	18.3	5.9	18.3	6.1	Steel Fracture	18.5	0.27	4.5	1.10
128	CON3-6-N				18.3	3.3	15.8	5.3	Steel Fracture				
129	CON3-7-N				18.6	4.0	15.1	5.6	Steel Fracture				
130	CON3-8-N				18.9	4.8	15.3	6.0	Steel Fracture				
131	CON3-9-N	9.5	88	-	47.4	3.1	10.1	15.8	Combined Cone-Bond	40.7	6.07	3.1	1.86
132	CON3-10-N				34.8	3.7	9.7	17.0	Combined Cone-Bond				
133	CON3-11-N				36.4	5.2	18.5	17.6	Combined Cone-Bond				
134	CON3-12-N				44.3	0.7	41.7	1.0	Splitting and Cracking				
135	CON3-13-N	9.5	114	-	50.4	3.2	0.0	17.3	Steel Fracture	54.8	2.69	3.4	0.64
136	CON3-14-N				53.3	3.5	12.1	17.8	Steel Fracture				
137	CON3-15-N				56.9	2.7	17.1	14.3	Steel Fracture				
138	CON3-16-N				54.0	4.2	24.7	16.5	Steel Fracture				
139	CON3-1-45°	6.4	88	-	6.3	11.3	5.3	13.1	Steel Fracture	6.6	1.72	12.2	1.12
140	CON3-2-45°				9.1	11.7	8.2	12.1	Steel Fracture				
141	CON3-3-45°				5.3	13.8	4.7	14.2	Steel Fracture				
142	CON3-4-45°				5.6	12.0	4.7	14.0	Steel Fracture				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
143	CON3-5-45°	6.4	114	-	6.2	14.1	4.8	15.9	Steel Fracture	6.7	0.88	10.4	2.83
144	CON3-6-45°				6.7	9.7	5.6	11.1	Steel Fracture				
145	CON3-7-45°				7.9	10.3	6.6	11.8	Steel Fracture				
146	CON3-8-45°				5.9	7.3	3.8	9.1	Steel Fracture				
147	CON3-9-45°	9.5	88	-	18.8	16.3	8.7	22.0	Pullout (Bond)	20.0	5.93	15.4	1.73
148	CON3-10-45°				23.7	16.7	22.2	16.9	Splitting and Cracking				
149	CON3-11-45°				12.1	15.8	4.1	22.8	Pullout (Bond)				
150	CON3-12-45°				25.3	12.9	20.0	13.9	Steel Fracture				
151	CON3-13-45°	9.5	114	-	20.3	17.1	17.5	18.3	Steel Fracture	23.1	5.42	15.7	1.92
152	CON3-14-45°				27.5	13.1	15.5	16.2	Steel Fracture				
153	CON3-15-45°				16.8	17.2	10.7	19.9	Splitting and Cracking				
154	CON3-16-45°				27.7	15.6	26.4	15.8	Splitting and Cracking				
155	CON3-17-N	6.4	88	40	17.9	2.6	13.7	4.4	Steel Fracture	17.3	0.51	2.3	0.87
156	CON3-18-N				17.0	3.1	14.2	3.4	Steel Fracture				
157	CON3-19-N				17.0	1.4	14.7	2.1	Steel Fracture				
158	CON3-20-N	6.4	88	60	17.0	3.1	14.5	4.7	Steel Fracture	17.8	1.29	2.4	0.62
159	CON3-21-N				17.0	1.9	14.4	2.0	Steel Fracture				
160	CON3-22-N				19.2	2.1	16.1	2.2	Steel Fracture				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
161	CON3-23-N	6.4	114	40	15.2	1.3	10.5	3.3	Steel Fracture	17.0	1.77	1.4	0.51
162	CON3-24-N				18.7	1.0	15.0	1.0	Steel Fracture				
163	CON3-25-N				17.1	2.0	13.5	2.0	Steel Fracture				
164	CON3-26-N	6.4	114	60	18.2	0.9	15.0	0.9	Steel Fracture	18.4	1.39	1.7	0.98
165	CON3-27-N				17.2	2.8	5.5	11.1	Pullout (Bond)				
166	CON3-28-N				20.0	1.4	19.2	1.5	Steel Fracture				
167	CON3-29-N	9.5	88	40	33.2	1.3	9.5	7.0	Breakout Cone	26.3	6.65	1.7	0.99
168	CON3-30-N				25.7	2.9	9.6	13.6	Splitting and Cracking				
169	CON3-31-N				19.9	1.0	1.9	5.5	Splitting and Cracking				
170	CON3-32-N	9.5	88	60	33.2	0.9	27.8	1.3	Splitting and Cracking	33.5	8.73	1.0	0.15
171	CON3-33-N				25.0	1.2	2.4	10.1	Splitting and Cracking				
172	CON3-34-N				42.4	1.0	6.7	14.6	Splitting and Cracking				
173	CON3-35-N	9.5	114	40	53.7	2.8	27.4	3.9	Breakout Cone	45.2	7.77	2.0	0.70
174	CON3-36-N				43.4	1.5	38.2	1.9	Splitting and Cracking				
175	CON3-37-N				38.5	1.7	30.3	2.3	Splitting and Cracking				
176	CON3-38-N	9.5	114	60	48.5	1.9	36.7	2.1	Steel Fracture	48.3	0.14	2.5	0.59
177	CON3-39-N				48.2	2.6	12.3	16.3	Splitting and Cracking				
178	CON3-40-N				48.2	3.0	17.8	21.8	Splitting and Cracking				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
179	LS-1-N	6.4	88	-	20.4	1.3	11.3	12.2	Steel Fracture	17.8	1.92	1.6	0.18
180	LS-2-N				16.4	1.8	9.3	9.4	Steel Fracture				
181	LS-3-N				16.2	1.6	8.3	9.0	Steel Fracture				
182	LS-4-N				18.1	1.6	4.9	7.3	Steel Fracture				
183	LS-15-N	9.5	88	-	45.9	1.6	13.5	9.8	Steel Fracture	48.3	2.93	2.3	1.32
184	LS-16-N				47.6	2.0	22.8	10.7	Steel Fracture				
185	LS-17-N				52.5	4.2	16.8	9.5	Steel Fracture				
186	LS-18-N				47.0	1.3	16.0	8.9	Steel Fracture				
187	LS-21-N	6.4	114	-	18.4	1.0	1.6	10.0	Steel Fracture	18.8	1.25	2.2	2.01
188	LS-22-N				18.6	0.7	6.4	9.1	Steel Fracture				
189	LS-23-N				20.6	1.9	4.4	6.8	Steel Fracture				
190	LS-24-N				17.7	5.1	14.1	7.0	Steel Fracture				
191	LS-35-N	9.5	114	-	47.6	2.7	9.2	13.5	Steel Fracture	49.0	2.74	2.1	0.48
192	LS-36-N				48.4	2.2	11.2	13.5	Steel Fracture				
193	LS-37-N				53.1	1.8	11.0	11.8	Steel Fracture				
194	LS-38-N				47.0	1.7	11.2	12.2	Steel Fracture				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
195	LS-1-45°	6.4	88	-	10.9	14.2	7.4	15.1	Steel Fracture	9.2	1.16	15.2	1.86
196	LS-2-45°				8.2	17.1	2.6	20.7	Pullout (Bond)				
197	LS-3-45°				8.7	13.1	7.6	14.7	Steel Fracture				
198	LS-4-45°				9.1	16.4	4.7	20.5	Pullout (Bond)				
199	LS-15-45°	9.5	88	-	15.3	13.7	15.3	13.7	Steel Fracture	17.2	1.80	16.5	2.07
200	LS-16-45°				18.2	17.0	16.3	18.9	Steel Fracture				
201	LS-17-45°				16.3	16.5	13.2	16.8	Steel Fracture				
202	LS-18-45°				19.2	18.7	16.3	19.7	Steel Fracture				
203	LS-21-45°	6.4	114	-	13.0	20.0	12.1	20.1	Steel Fracture	9.2	2.57	14.4	5.60
204	LS-22-45°				7.6	13.6	3.8	22.8	Pullout (Bond)				
205	LS-23-45°				7.6	17.1	7.1	18.7	Steel Fracture				
206	LS-24-45°				8.7	7.0	1.1	10.5	Pullout (Bond)				
207	LS-31-45°	9.5	114	-	25.3	25.3	21.2	29.0	Pullout (Bond)	23.9	2.56	17.0	5.78
208	LS-32-45°				20.8	16.8	20.8	16.8	Pullout (Bond)				
209	LS-33-45°				23.0	13.6	8.2	18.4	Pullout (Bond)				
210	LS-34-45°				26.6	12.5	22.9	13.1	Steel Fracture				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
211	LS-41-N	6.4	88	40	16.0	1.7	12.4	3.3	Steel Fracture	16.0	1.97	2.3	0.63
212	LS-42-N				18.0	2.2	15.2	2.5	Steel Fracture				
213	LS-43-N				14.0	3.0	4.4	14.4	Pullout (Bond)				
214	LS-44-N	6.4	88	60	16.3	2.0	4.8	12.1	Pullout (Bond)	16.4	0.74	2.3	0.55
215	LS-45-N				15.7	2.9	15.7	2.9	Steel Fracture				
216	LS-46-N				17.2	2.0	5.0	14.4	Pullout (Bond)				
217	LS-47-N	6.4	114	40	17.4	1.0	17.4	1.0	Steel Fracture	17.5	0.63	1.3	0.23
218	LS-48-N				16.9	1.5	2.9	10.7	Pullout (Bond)				
219	LS-49-N				18.1	1.3	15.6	2.6	Steel Fracture				
220	LS-50-N	6.4	114	60	19.2	2.3	16.1	3.1	Steel Fracture	18.7	2.38	9.9	7.29
221	LS-51-N				20.8	16.8	20.8	16.8	Steel Fracture				
222	LS-52-N				16.2	10.8	11.2	22.8	Pullout (Bond)				
223	LS-53-N	9.5	88	40	41.5	3.6	6.0	22.1	Splitting and Cracking	42.7	2.04	1.7	1.64
224	LS-54-N				41.4	0.6	13.7	21.6	Combined Cone-Bond				
225	LS-55-N				45.0	1.0	44.5	1.0	Breakout Cone				
226	LS-56-N	9.5	88	60	40.7	1.6	37.5	2.0	Splitting and Cracking	40.4	5.40	1.2	0.37
227	LS-57-N				34.9	0.9	15.0	21.8	Combined Cone-Bond				
228	LS-58-N				45.7	1.1	33.0	20.2	Combined Cone-Bond				

No.	Sample Number	Rod Diameter (mm)	Effective Depth (mm)	Edge Distance (mm)	Peak Load (kN)	Slip At Peak Load (mm)	Ultimate Load (kN)	Ultimate Slip (mm)	Failure Mode	Average Peak Load (kN)	Standard Deviation of Peak Load	Average Slip at Peak Load (mm)	Standard Deviation of Slip at Peak Load
229	LS-59-N	9.5	114	40	45.7	1.9	14.7	22.5	Pullout (Bond)	44.1	1.42	1.9	0.03
230	LS-60-N				43.7	1.9	20.7	21.2	Pullout (Bond)				
231	LS-61-N				43.0	2.0	19.4	22.3	Pullout (Bond)				
232	LS-62-N	9.5	114	60	38.1	1.5	27.8	14.0	Pullout (Bond)	47.1	9.19	1.7	0.29
233	LS-63-N				46.9	2.0	40.8	4.3	Steel Fracture				
234	LS-64-N				56.4	1.5	47.1	1.6	Steel Fracture				

Appendix C

**Load – displacement curves of anchor specimen in
hollow clay brick**

Figure C.1: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 28 mm, slope = 90° in hollow brick

Figure C.2: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 28 mm, slope = 90° in hollow brick

Figure C.3: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 40 mm, slope = 45° in hollow brick

Figure C.4: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 28 mm, slope = 45° in hollow brick

Figure C.5: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 63 mm, slope = 90° in hollow brick

Figure C.6: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 63 mm, slope = 90° in hollow brick

Appendix D

**Load – displacement curves of anchor specimen in
hollow concrete block**

Figure D.1: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 35 mm and slope = 90° in hollow side concrete block

Figure D.2: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 35 mm and slope = 90° in hollow side concrete block

Figure D.3: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 49 mm and slope = 45° in hollow side concrete block

Figure D.4: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 49 mm and slope = 45° in hollow side concrete block

Figure D.5: Load–displacement relationship of anchor diameter = 6.4 mm, depth = 55 mm and slope = 90° in hollow edge concrete block

Figure D.6: Load–displacement relationship of anchor diameter = 9.5 mm, depth = 55 mm and slope = 90° in hollow edge concrete block

Figure D.7: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 77 mm and slope = 45° in hollow edge concrete block

Figure D.8: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 77 mm and Slope = 45° in hollow edge concrete block

Appendix E

**Load – displacement curves of anchor specimen in
20.6 MPa concrete**

Figure E.1: Load-Displacement Relationship of anchor diameter = 6.4 mm, depth = 88 mm and slope = 90° in 20.6 MPa concrete

Figure E.2: Load-displacement relationship of anchor diameter= 6.4 mm, depth = 114 mm and slope = 90° in 20.6 MPa Concrete

Figure E.3: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 88 mm and slope = 90° in 20.6 MPa Concrete

Figure E.4: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm and slope = 90° in 20.6 MPa concrete

Figure E.5: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm and slope = 45° in 20.6 MPa concrete

Figure E.6: Load-displacement Relationship of anchor diameter = 6.4 mm, depth = 114 mm and slope = 45° in 20.6 MPa concrete

Figure E.7: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 88 mm and slope = 45° in 20.6 MPa concrete

Figure E.8: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm and slope = 45° in 20.6 MPa concrete

Figure E.9: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm, edge = 40 mm and slope = 90° in 20.6 MPa concrete

Figure E.10: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm, edge = 60 mm and slope = 90° in 20.6 MPa concrete

Figure E.11: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 114 mm, edge = 40 mm and slope = 90° in 20.6 MPa concrete

Figure E.12: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 114 mm, edge = 60 mm and slope = 90° in 20.6 MPa concrete

Figure E.13: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 88 mm, edge = 40 mm and slope = 90° in 20.6 MPa concrete

Figure E.14: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 88 mm, edge = 60 mm and slope = 90° in 20.6 MPa concrete

Figure E.15: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm, edge = 40 mm and slope = 90° in 20.6 MPa concrete

Figure E.16: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm, edge = 60 mm and slope = 90° in 20.6 MPa concrete

Appendix F

Load – displacement curves of anchor specimen in

34.5 MPa concrete

Figure F.1: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm and slope = 90° in 34.5 MPa concrete

Figure F.2: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 114 mm and slope = 90° in 34.5 MPa concrete

Figure F.3: Load-displacement Relationship of anchor diameter = 9.5 mm, depth = 88 mm and slope = 90° in 34.5 MPa concrete

Figure F.4: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm and slope = 90° in 34.5 MPa concrete

Figure F.5: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm and slope = 45° in 34.5 MPa concrete

Figure F.6: Load-displacement Relationship of anchor diameter = 6.4 mm, depth = 114 mm and slope = 45° in 34.5 MPa concrete

Figure F.7: Load-displacement Relationship of anchor diameter = 9.5 mm, depth = 88 mm and slope = 45° in 34.5 MPa concrete

Figure F.8: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm and slope = 45° in 34.5 MPa concrete

Figure F.9: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm, edge = 40 mm and slope = 90° in 34.5 MPa concrete

Figure F.10: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm, edge = 60 mm and slope = 90° in 34.5 MPa concrete

Figure F.11: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 114 mm, edge = 40 mm and slope = 90° in 34.5 MPa concrete

Figure F.12: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 114 mm, edge = 60 mm and slope = 90° in 34.5 MPa concrete

Figure F.13: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 88 mm, edge = 40 mm and slope = 90° in 34.5 MPa concrete

Figure F.14: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 88 mm, edge = 60 mm and slope = 90° in 34.5 MPa concrete

Figure F.15: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm, edge = 40 mm and slope = 90° in 34.5 MPa concrete

Figure F.16: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm, edge = 60 mm and slope = 90° in 34.5 MPa concrete

Appendix G

Load - displacement curves of anchor specimen in limestone

Figure G.1: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm and slope = 90° in limestone

Figure G.2: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 114 mm and slope = 90° in limestone

Figure G.3: Load–displacement relationship of anchor diameter = 9.5 mm, depth = 88 mm and slope = 90° in limestone

Figure G.4: Load–displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm and slope = 90° in limestone

Figure G.5: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm and slope = 45° in limestone

Figure G.6: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 114 mm and slope = 45° in limestone

Figure G.9: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm, edge = 40 mm and slope = 90° in limestone

Figure G.10: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 88 mm, edge = 60 mm and slope = 90° in limestone

Figure G.11: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 114 mm, edge = 40 mm and slope = 90° in limestone

Figure G.12: Load-displacement relationship of anchor diameter = 6.4 mm, depth = 114 mm, edge = 60 mm and slope = 90° in limestone

Figure G.13: Load–displacement relationship of anchor diameter = 9.5 mm, depth = 88 mm, edge = 40 mm and slope = 90° in limestone

Figure G.14: Load–displacement relationship of anchor diameter = 9.5 mm, depth = 88 mm, edge = 60 mm and slope = 90° in limestone

Figure G.15: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm, edge = 40 mm and slope = 90° in limestone

Figure G.16: Load-displacement relationship of anchor diameter = 9.5 mm, depth = 114 mm, edge = 60 mm and slope = 90° in limestone