TENSION CAPACITY DETERMINATION OF A NORMAL SLEEVE ANCHOR

MUHAMMAD IKHWAN BIN SABLI

CIVIL ENGINEERING
UNIVERSITI TEKNOLOGI PETRONAS
JUNE 2010

Tension Capacity Determination of a Normal Sleeve Anchor

By

Muhammad Ikhwan Bin Sabli

Dissertation submitted in partial fulfillment of
the requirements for the
Bachelor of Engineering (Hons)
(Civil Engineering)

JUNE 2010

Universiti Teknologi Petronas
Bandar Seri Iskandar
31750 Tronoh
Perak Darul Ridzuan

CERTIFICATION OF APPROVAL

Tension Capacity Determination of a Normal Sleeve Anchor

by

Muhammad Ikhwan Bin Sabli

A project dissertation submitted to the
Civil Engineering Programme
Universiti Teknologi PETRONAS
in partial fulfillment of the requirement for the
BACHELOR OF ENGINEERING (Hons)
(CIVIL ENGINEERING)

Approved by,

____
(Mr. Kalaikumar A/L Vallyutham)

UNIVERSITI TEKNOLOGI PETRONAS
TRONOH, PERAK
June 2010

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

MUHAMMAD IKHWAN BIN SABLI

ABSTRACT

There are many applications nowadays using the anchorage system and every application requires suitable anchor type or capacity that can sustain the design load. The normal sleeve anchor that can be found in the normal hardware shop is usually produced based on its dimension only. The specification of the anchor's capacity is not provided. This situation can cause danger if the application load is greater than the anchor's capacity can handle. On the other hand, if the normal sleeve anchor has the required capacity same as the custom-made anchor and expensive that can handle wide range of application, it will result in saving money and time in order to get the specially design anchor. In this paper, the tension capacity of a normal sleeve anchor with selected anchor diameters are obtained by pull-out test. The pull-out test is carried out using the Universal Tensile Machine (UTM) 1000kN in Universiti Teknologi Petronas's laboratory. The normal sleeve anchor is installed in the concrete specimen with average compressive strength of $30N/mm^2$. Only adhesive failure which is the friction failure between the anchor and the concrete is considered. The result has revealed that the tension capacity of a normal sleeve anchor can be obtained using pull-out test and similar to the theoretical values. Greater diameter of anchor provided greater contact surface between the anchor and the concrete, thus greater tensile strength.

ACKNOWLEDGEMENT

First and foremost, I would like to praise to God the Almighty for His guidance. Though difficulties occurred, His guidance gave me the chance to still complete this challenging final year project successfully. Here, I would like to use this special opportunity to express my heartfelt gratitude to everyone that has contributed to the success of this project.

My deepest appreciation goes to my supervisors, Mr. Kalaikumar Vallyutham for his advice and guidance plus the moral supports throughout the completion of the project. I really acknowledge all the precious words and assistance from him.

I also would like to extend my deepest appreciation to those who had involved directly and indirectly toward the completion of the project. My sincere gratitude goes to the Head of Civil Engineering Department, Assoc. Prof. Dr. Shamsul Rahman Bin Mohamed Kutty, and all technicians in concrete laboratory for helping me in completing the test for the project.

Last but not least, I like to express my gratitude to everybody who has been helping and encouraging me directly or indirectly from the beginning to the end of my commitment on the final year project.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1
INTRODUCTION

1.1 Background of Study

Anchor embedment is a common technology that is widely used in civil engineering. Every application, ranging from hanging acoustical ceilings and installing window and door frames to performing seismic upgrades, requires the use of an anchoring product.

There are many types of anchor that regularly have been used in concrete and masonry construction. The anchor types depend on how it is fasten in the concrete. Under the cast-in place installation method, the anchor types are threaded sleeves, channel bars, headed anchors and others. In post-installed installation method, for drill installation types of anchor are sleeve expansion anchors, undercut anchors, bonded anchors, bonded-expansion anchors, bonded-undercut anchors and screw anchors. While for direct installation type of anchor mostly is the power-actuated fasteners [1].

The normal wall plug that will be experimented in this project as shown in figure 1.1 is the drill-in type of anchor and also known as the sleeve anchor. This normal sleeve anchor consist of threaded rod with nut, washer, spacer and expansion sleeve, thus, it falls under the category of torque-controlled expansion anchors and can be classified as sleeve-type anchor [1].

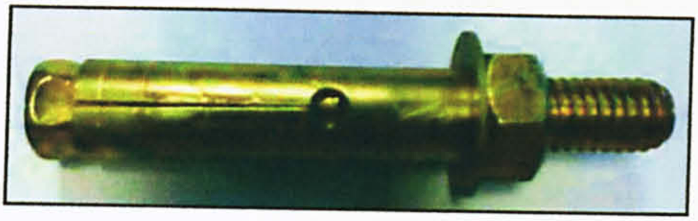


Figure 1.1: Normal sleeve anchor

There are three (3) load-transfer mechanisms as shown in figure 1.2 on how the fasteners transfer applied tension load to the base material. The mechanisms can be usually identified as mechanical interlock, friction interlock and bond interlock [1]. For expansion types of anchor, the load-transfer mechanism applied is due to friction interlock.

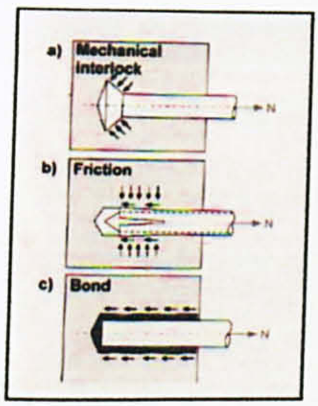


Figure 1.2: Load transfer mechanism [1]

1.2 Problem Statement

The normal wall plug usually comes with two different types of material; the plastic threaded type and the sleeve expansion type.

Currently, the normal wall plug especially the sleeve anchor type that available in the hardware shop did not come with its technical data specifications. It is produced for user based on its dimension only while the information on its specification such as maximum load capacity is not provided. Different size in dimension of the anchor may provide different handling capacity of loads.

Normally, for higher load applications will require the user to spend a lot of money to get the specific anchor system that can sustain the loads of the application. In contrast to that, the user maybe need to spend less money if the capacity of a normal sleeve anchor is known and its loads capacity meet the requirement for the higher loads application.

Moreover, since there is no test or quality check done on the normal sleeve anchor to determine its load capacity after it is being produced, the quality of the anchor of each batch produced might be different with each other. The inappropriate use of the normal sleeve anchor can cause danger to the intended use of application as if the load applied is greater than the anchor capacity can handle.

Thus, the project is carried out to determine the capacity of a normal sleeve anchor which to come out with the experimental setup and to carry out the experimental work in determining the capacity of the anchor.

1.3 Objective

The objective of the project is to determine the capacity of a normal sleeve anchor in terms of its tensile strength. To be more specific, the purpose is to determine the maximum tension loading for different sizes of normal sleeve anchor in concrete. However, there are many parameters to be considered in this project. The parameters may provide more results for the project and it will be explain in Section 1.4.

1.4 Scope of Study

The scope of this project is limited to the experiment that is conducted in UTP Laboratory. The only available testing machine in the lab to determine the tensile strength that meets the requirement of the project is the Universal Tensile Machine (UTM).

The sizes of 8mm, 10mm, 12mm and 16mm anchor diameter are tested in this experiment. Each anchor is plugged into the same size of concrete sample to get the tensile capacity between different sizes of anchor.

The type of loading failure that is considered into account of the project result is only the adhesive failure mode. This includes the friction failed at the contact surface between the anchor and the concrete surface.

The dimension of 140mm x140mm x 550mm concrete is casted to meet the standard requirement for anchor installation purpose and to suit the requirement size at the tensile machine for testing.

CHAPTER 2
LITERATURE REVIEW

There are many publications available that related to the project. Only relevant publications have been selected to be reviewed and summarized. The literature review has been separated into two parts; the first part consider the previous experiments or method done to determine the tensile capacity of anchor; and the second part describes the failure mechanisms of anchor in hardened concrete.

2.1 Tensile Capacity of Anchor

2.1.1 Direct Tension Pull-out Bond Test

An experimental study known as Direct Tension Pullout Bond Test (DTP-BT) has carried out by Tastani to measure the lower bound bond properties of steel and Glass Fiber Reinforced Polymer (GFRP) bars fixed in normal strength concrete [2]. As shown in Figure 2.1, Tastani (2002) says that the "frictional concept" was apply to explain the stress transfer between steel and concrete; whereby the bond stress is the term used for shear stress that occurs along the lateral surface of the bar, is basically a function of the normal confining pressure take up by the concrete surrounding the bar surface [2]. The aims of the alternative bond test are to measure the low bound of bond strength that suitable for both steel and GFRP reinforcement that may be subjected from the concrete cover (while the effects of bar curvature are not present). In conclusion, the reason of this DTP-BT test is proposed because based on the test conducted, the most unpleasant conditions for bond is when the concrete cover is under a direct tension stress field, also known as yield bond conditions that differ insignificantly to those resulted by other established bond test [2].

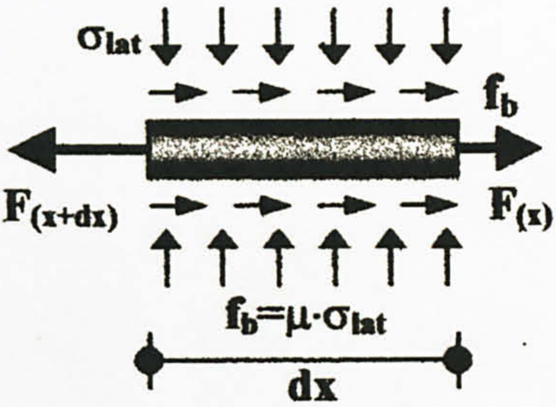


Figure 2.1: Frictional model for bond: $f_b=\mu.\sigma_{lat}$ [2]

2.1.2 Effects of Various Types of Cracks to the Anchorage Capacity

An actual model of tests was conducted by Jang, Suh and Lee to come out with the model showing the effect of the various types of cracks [3]. The outcome of the test was then applied in the evaluating the anchorage capacity of equipment for seismic qualification. The test was done according to the *Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements* in the ASTM E488 [8]. The crack width, crack depth, the distance between crack and anchor, and crack pattern was selected as the test variables in the experiment. Result of load-displacement curve from concrete breakout failure test as shown in figure 2.2a-c; for load-displacement curve for non-cracked test and cracked test, it is shown that the curve after the maximum concrete breakout strength of the non-cracked test results produced a slightly inclination compared to the cracked test results that are more steep declining inclination [3].

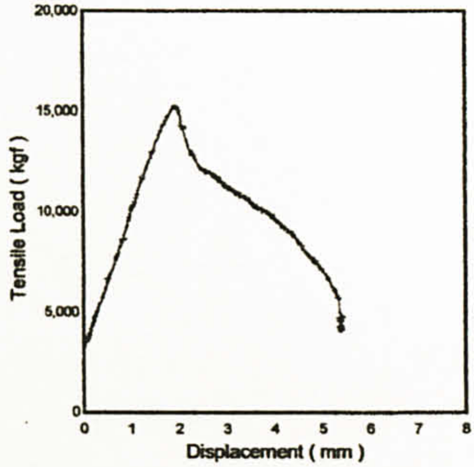


Figure 2.2a: Load-displacement curve of concrete breakout failure [3]

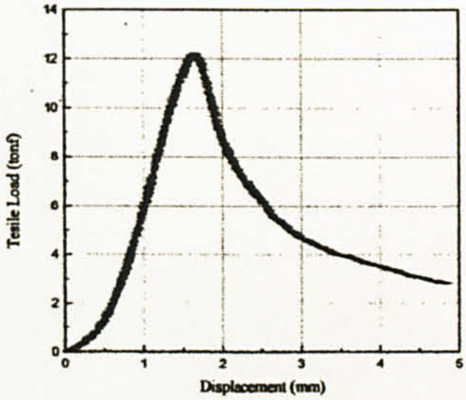


Figure 2.2b: Load-displacement curve for non-cracked test [3]

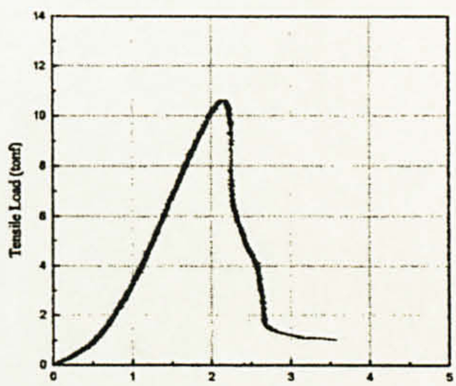


Figure 2.2c: Load-displacement curve for crack test [3]

At the end of the tests, it is concluded that crack width that was taken as design criteria which was necessary to modify with the test results, was less important compare to that the distance from anchor to crack and the crack depth [3].

2.1.3 Numerical Modeling of the Dynamic Pull-out Failure Loads

Since experimental pull-out tests is difficult to be conducted and uneconomic; Walter, Baillet and Brunet have come out with the idea of modeling that aimed to predict numerically the mechanical response of the anchors embedded in concrete using the finite elements program, PLAST2, that is based on a dynamic explicit method [4]. The PLAST2 program includes a pre-processor, a solution program and an efficient post-processor that can be interactively controlled and simulates in several levels of real time graphics [4]. Numerical results of the program was computed in the Table 2.1a and Table 2.1b for different types of anchorage which are named A1 that have a diameter and a length smaller than named A2 in order to cover the range of various set of anchor in the industry. It was concluded that the dynamic and quasi-static load carrying capacity of anchors rely more significantly on the material and the contact friction behavior compared to the tensile strength and toughness of the concrete [4].

Table2.1a: Pull-out failure loads F of the anchor A1 for both types of concrete with different values of the friction coefficient, $\mu_{c/c}$ [4]

$\mu_{c/c}$	0.1	0.2	0.3	0.35	0.4	experi-mental
F (N) "weak" concrete	10331	10800	14900	13600	11110	17483
F (N) "stiff" concrete	17960	21500	21640	22500	23350	24800

Table 2.1b: Pull-out failure loads F of the anchor A2 for both types of concrete with different values of the friction coefficient, $\mu_{c/c}$ [4]

$\mu_{c/c}$	0.1	0.2	0.3	0.35	0.4	experi-mental
F (N) "weak" concrete	24523	30790	36260	41110	37400	49450
F (N) "stiff" concrete	43470	52410	61670	67420	82630	70000

2.1.4 Artificial Neural Networks (ANNs)

Many design parameters should be considered in determining the tensile strength of an anchor. Conventional product-specific and condition-specific testing are mostly done for determining the tensile capacity of such type of anchors due to its complexity in developing rational models [5]. Sherief and Ashraf have attempted to use the Artificial Neural Networks (ANNs) method to predict the tensile strength of single adhesive anchors [5]. As shown in the Figure 2.3a, the trained ANN results that the tensile capacity of adhesive anchors was linearly proportional to the embedment depth. The tensile capacity of adhesive anchors was reliant on the effect of the concrete compressive strength [5]. Figure 2.3b show that the ANN model was capable of predicting the tensile capacity of adhesive anchors.

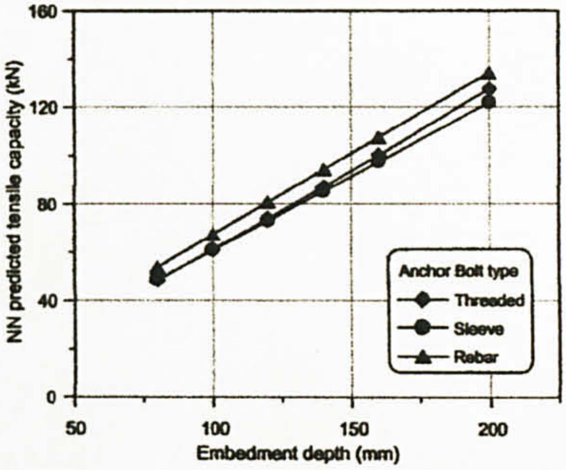


Figure 2.3a: Effect of the anchor bolt type and the embedment length on the predicted ultimate tensile capacity [5]

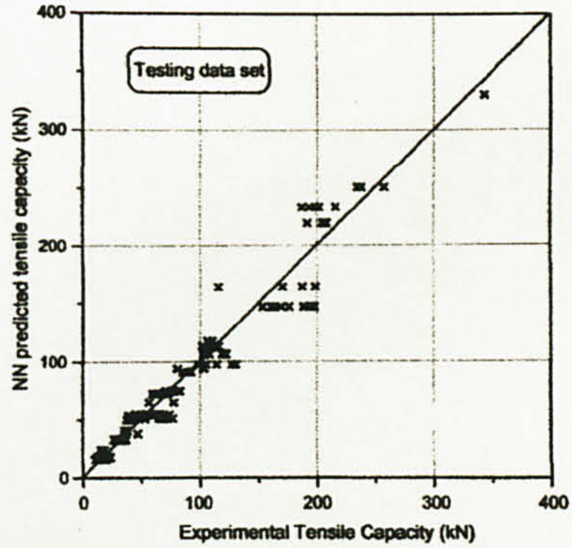


Figure 2.3b: Comparison of experimental and predicted ultimate tensile capacities for the ANN testing data set [5]

2.1.5 Friction Coefficient of Steel and Concrete

An experiment has been carried out by Rabbat and Russell to determine the coefficient of static friction between rolled steel plate and cast-in-place concrete or grout [6]. In this experiment, the boundary between concrete and steel plate was tested with wet and dry conditions. The results of the bond strength and the coefficient of friction are shown in Table 2.2.

Table 2.2: Bond Strength and Coefficient of Friction [6]

Specimen (1)	Bond strength, in pounds per square inch (2)	Shear Stress, in Pounds per Square Inch		Coefficient of Friction			
		Peak (3)	Effective (4)	Peak (5)	Average of each set (6)	Effective (7)	Average of each set (8)
CWA-1	52.8	62.0	62.0	0.62	0.64	0.62	0.64
CWA-2	48.6	63.5	63.5	0.63		0.63	
CWA-3	51.4	68.0	67.8	0.68		0.68	
CWB-1	55.0	42.0	40.7	0.70	0.68	0.68	0.65
CWB-2	25.0	40.4	39.6	0.67		0.66	
CWB-3	89.0	40.5	36.1	0.68		0.60	
CWC-1	61.2	13.8	12.8	0.69	0.70	0.64	0.67
CWC-2	83.6	14.2	14.0	0.71		0.70	
CWC-3	78.4	13.9	13.5	0.69		0.67	
CDB-1	67.3	40.5	33.7	0.68	0.69	0.56	0.57
CDB-2	53.0	46.2	35.0	0.77		0.58	
CDB-3	58.2	36.9	34.0	0.62		0.57	
GWB-1	8.0	41.2	41.2	0.69	0.68	0.69	0.68
GWB-2	—	40.9	40.9	0.68		0.68	
GWB-3	—	40.0	40.0	0.67		0.67	

Note: 1 psi = 6.895 kPa.

It was concluded that the bond strength for concrete specimens varied between 0.17MPa and 0.61MPa. The average effective coefficients of static friction were 0.57 and 0.69 for concrete specimens with dry interface and a normal stress of 0.41MPa [6].

2.2 Failure Mechanisms of Anchor in Hardened Concrete

Tensile failure modes has been explained by Matthew in his paper to design the capacity of the grouted anchor, the tensile strength and nominal strength for a single anchor can be determine based on the Equation (1a) to Equation (8b) as shown in the Appendix-A [7]. Table 2.3 shows the values for coefficient to calculate the characteristic bond strength in Equation (4) in Appendix-A. The testing method is based on performance in ASTM E488 tests. Figure 2.4 below show the typical tension failure modes for grouted anchors.

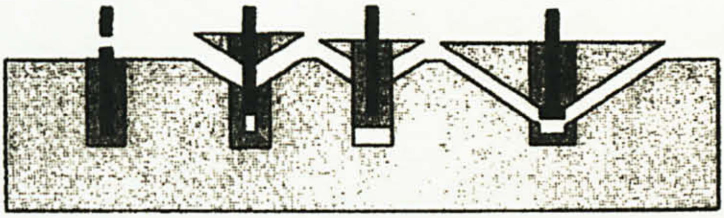


Figure 2.4: Typical tension failure modes for grouted anchors [7]

CHAPTER 3
METHODOLOGY

3.1 Project Flow Chart

The project was divided into seven phases. This was to ensure the project flow is smooth and accomplished in the frame time. Figure 3.1 below was the activities flow chart for this project.

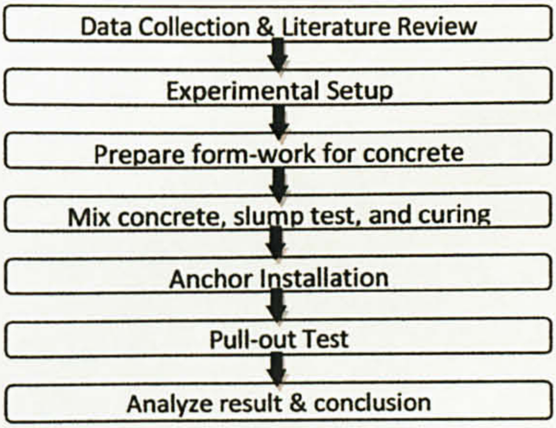


Figure 3.1: Flow chart of project

3.1.1 Data Collection and Literature Review

In the first phase, information related to the project were gathered such as journals, articles, websites, reference books, and thesis to get an overview and better understanding on the project scope.

3.1.2 Experimental Setup

Experimental setup for the project was prepared in terms of its test objective, theories, test apparatus, and test procedures. The potential hazards were also included in this experimental setup so that preventive actions are prepared.

3.1.3 Preparation of Form-work for Concrete

Before concrete was casted, form-work was prepared according to the intended size of concrete required for the testing purpose. To meet installation standard of the anchor and the required dimension that can suit the tensile machine, 140mm x 140mm x 550mm dimension of concrete form-work was prepared using ply wood and timber.

3.1.4 Concrete Sample Preparation

Preparation of concrete sample involved designing concrete mix proportion, mixing, casting and curing. All the procedures of preparing the concrete were based on the Mix Design provided by the UTP concrete lab as per attach in the Appendix-B. Each batch of mix consist of two (2) concrete specimens with dimension 140mm X 140mm X 550mm each as shown in figure 3.2 and six (6) test cubes with dimension of 150mm X 150mm X 150mm each.

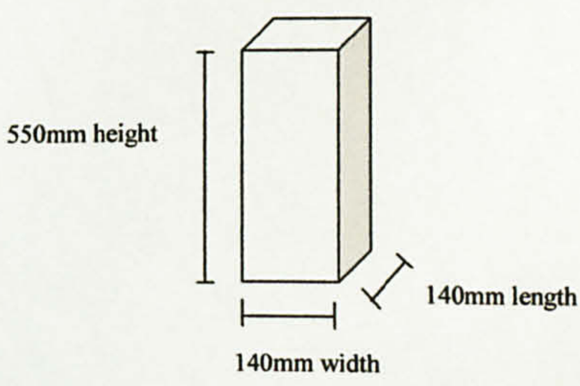


Figure 3.2: Typical dimension of concrete specimen

Three (3) cubes were tested after seven (7) days and the remaining three (3) after 28 days of curing respectively. The result of the concrete compressive test was computed as shown in Appendix-C.

3.1.5 Anchor Installation

The concrete was drilled using automatic driller machine. The drill bit size used was same as the anchor diameters which were 8mm, 10mm, 12mm and 16mm. The position of the anchor in concrete was as shown in figure 3.3 where the anchor was installed at the center of the concrete with surface area of 140mm x 140mm.

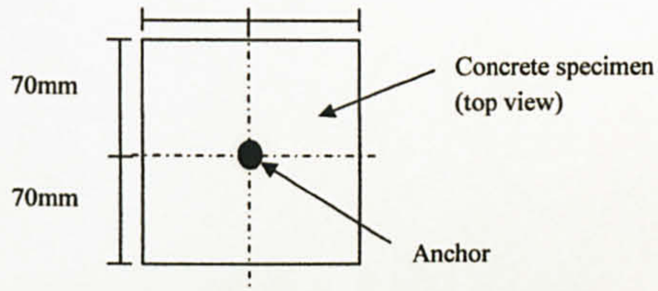


Figure 3.3: Position of the anchor in the concrete specimen

The installation procedure follows the method as shown in figure 3.4 where after the concrete was drilled, the concrete dust was removed using vacuum. Then the anchor was plugged into the drilled hole and torque was applied to the anchor nut to tighten the anchor.

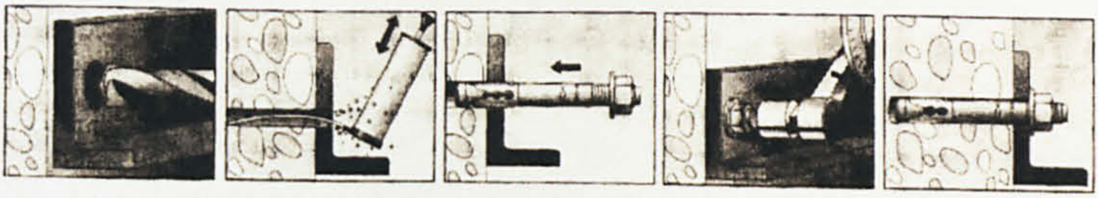


Figure 3.4: Procedure of installing anchor [1]

3.1.6 Pull-out Test

Pull-out test was carried out using the Universal Tensile Machine (UTM) as shown in figure 3.5, that available in the laboratory that suit the standard requirement for testing to determine the tensile capacity of the normal sleeve expansion anchor. The head adapter was custom made to pull-out the anchor from the concrete as shown in figure 3.6. The result were analyzed and compared with the theoretical values.

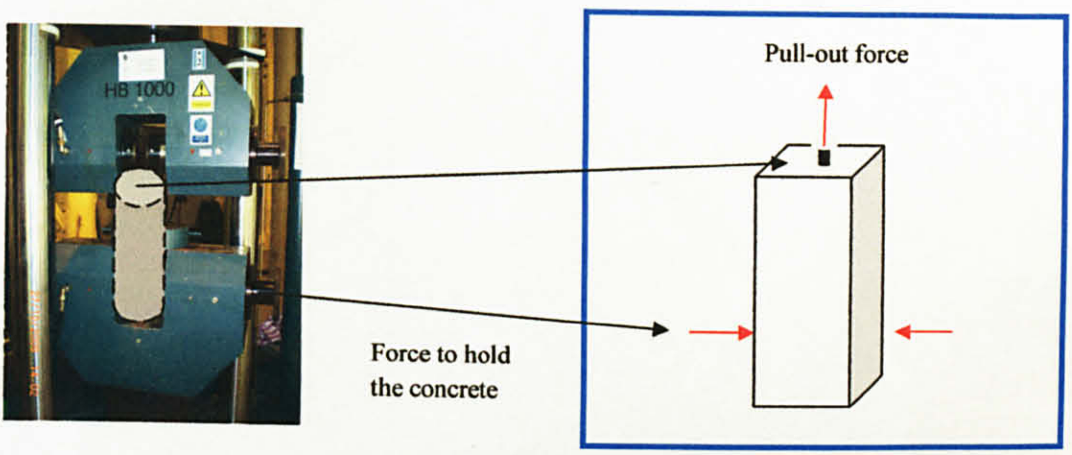


Figure 3.5: Experimental setup using UTM

Figure 3.6: Custom made head adapter to pull-out anchor from concrete

The proposed testing method was accordance to the standards provided in ASTME E488 - 96(2003) *Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements* [8] in Appendix-D; and ASTM C900 – 01 *Standard Test Method for Pullout Strength of Hardened Concrete* [9] which shown in the Appendix-E.

3.1.7 Final Report

The pull-out test result were analyzed and concluded at the end of the project. Complete report about the project including test results, discussion and conclusion was documented and presented at the end of the project schedule.

CHAPTER 4
RESULT AND DISCUSSION

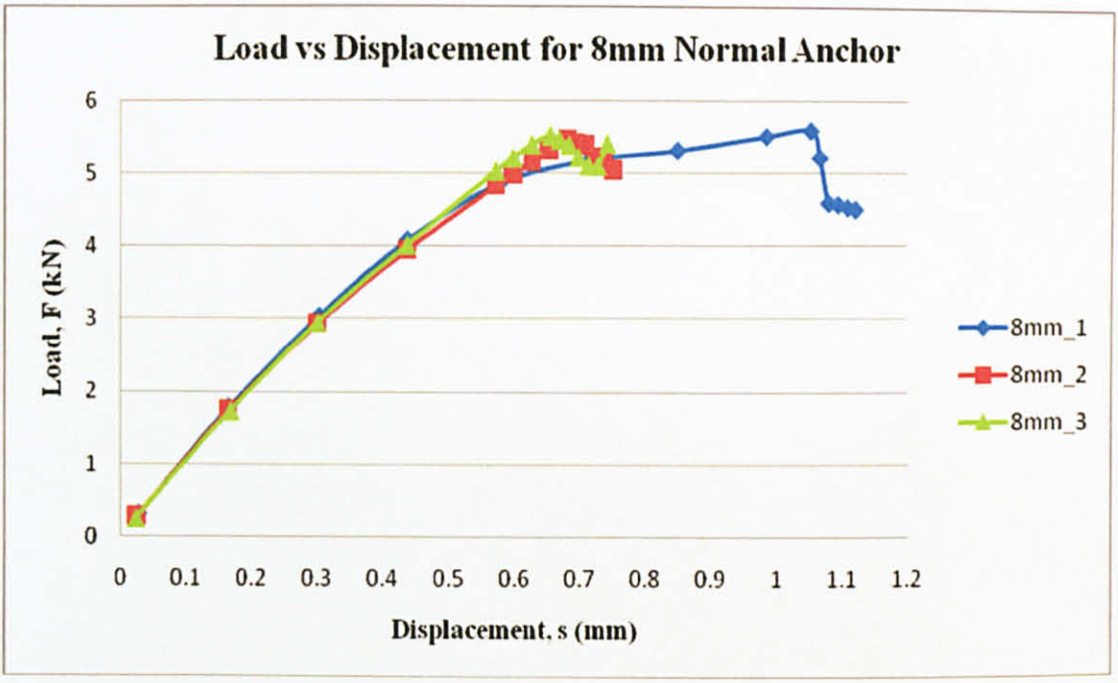
4.1 Pull-out Test Result

The type of failure that was considered in this test was the adhesive failure mode which the friction failure between the anchor and the concrete. The pull-out fail value was taken once the anchor starts slipping from the concrete as shown in figure 4.1.

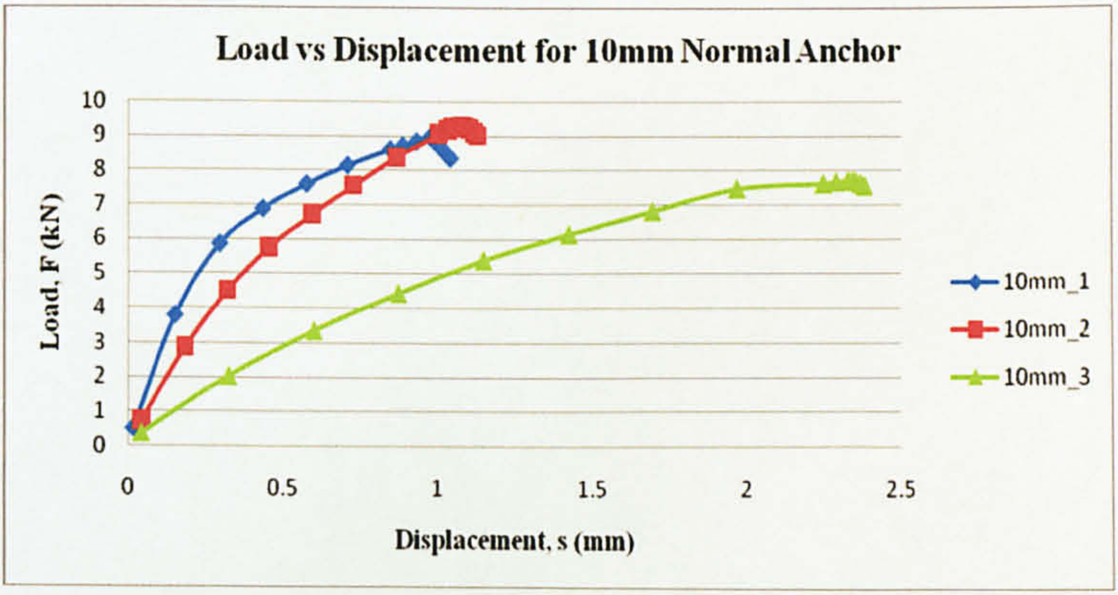
(i) (ii)

Figure 4.1: Condition of anchor (i) before pull-out (ii) after pull-out

Three (3) tests was carried out for each diameter of anchor and pull-out value for each anchor was represented in the graph as shown in figure 4.2a and figure 4.2b.

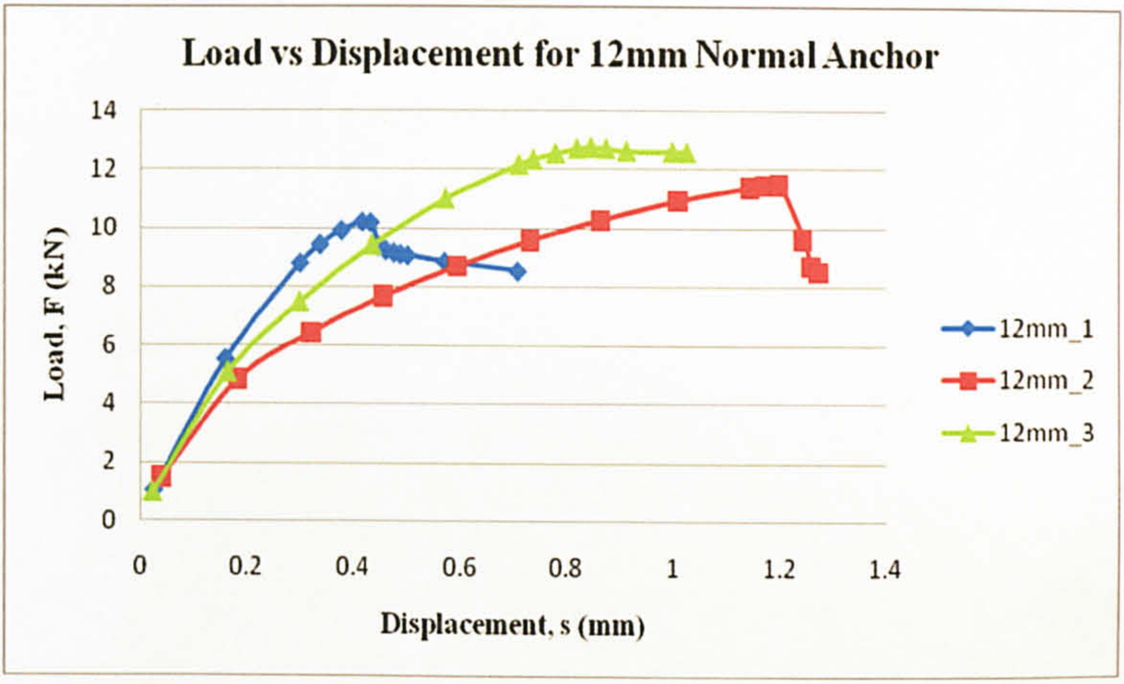


(i)

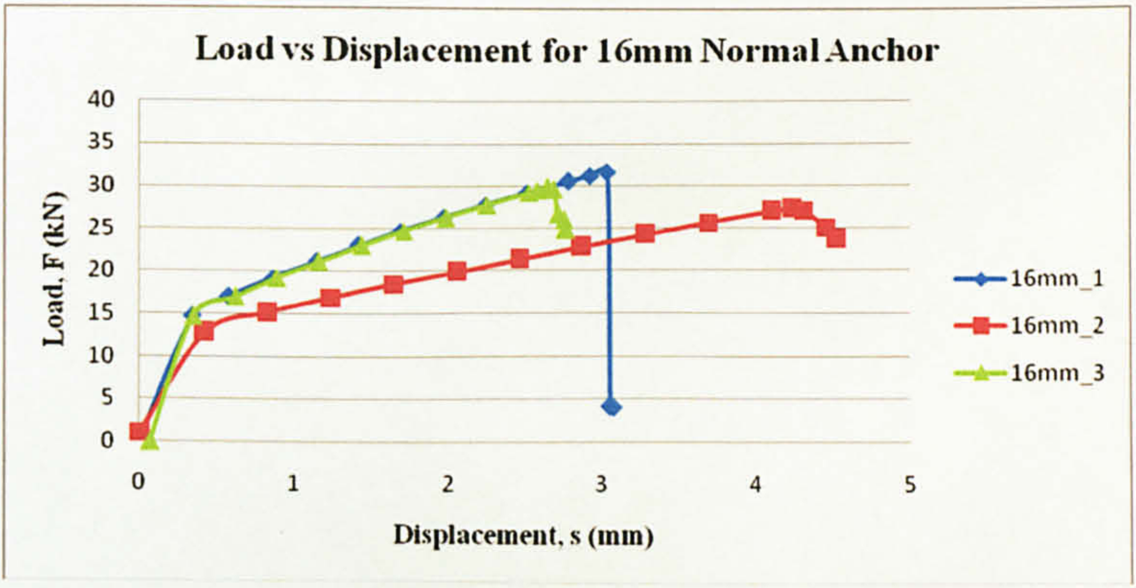


(ii)

Figure 4.2a: Load-displacement curve for (i) 8mm (ii) 10mm anchor



(iii)



(iv)

Figure 4.2b: Load-displacement curve for (iii) 12mm (iv) 16mm anchor

The highest peak of each graph show the ultimate strength of the anchor in concrete which the maximum tension loading the anchor can withstand when subjected to pull-out load. The pull-out test result and the average value was obtained as shown in Table 4.1:

Table 4.1: Pull-out test result

Normal Anchor Diameter (mm)	**Pull-out failure load (kN)**	**Average (kN)**
8mm	5.6	5.5
	5.5	
	5.5	
10mm	9.0	8.7
	9.3	
	7.7	
12mm	10.2	11.5
	11.5	
	12.7	
16mm	31.7	29.6
	27.2	
	29.9	

4.2 Theoretical Result

The computed theoretical results are presented in Table 4.2:

Table 4.2: Theoretical result

Anchor	$D_{e\,(mm)}$	L (mm)	L_e (mm)	f_c (N/mm^2)	Friction Coefficient, μ	Max. Pull-out Load (kN)
1.	8	50	17	30	0.35	4.5
2.	10	68	27	30	0.35	8.9
3.	12	75	27	30	0.35	10.7
4.	16	100	38	30	0.35	20.1

Where D_e is the effective depth

L is the anchor length

L_e is the effective length

f_c is the concrete compressive strength

4.3 Calculation of Theoretical Value

The calculation for the theoretical values was based on the BS 8110-1:1997 Section 3.12.8.2 *Anchorage Bond Stress* [10]:

$$\mathbf{f_b = F_s / \pi D_e L_e} \qquad \textbf{(1)}$$

Where f_b is the bond stress

F_s is the force in the bar or maximum pull-out loading

D_e is the effective bar size (anchor diameter)

L_e is the anchorage effective length

While according to the 'frictional concept' applied by Tastani previously [2], the bond stress, f_b:

$$\mathbf{f_b = \mu \cdot \sigma_{lat}} \qquad \mathbf{(2)}$$

Where μ is the friction coefficient

σ_{lat} is the lateral compressive stress

The friction coefficient between elements, μ, was referred to guidance from BS EN 12812:2004, *Falsework- Performance Requirements and General Design, Informative Annex A* [11] where the friction between steel and concrete element provided maximum is 0.4 and minimum is 0.3.

By taking the average value 0.35 of the friction coefficient into equation (2), the bond stress produced for each concrete with compressive strength 30 N/mm:

$f_b = 0.35\ (30\ \text{N/mm})$

$f_b = 10.5\ \text{N/mm}$

By arranging the equation (1) into:

$$\mathbf{F_s = f_b\ (\pi D_e L_e)} \qquad \mathbf{(3)}$$

Then value of f_b = 10.5 N/mm is applied into the equation (3) for each anchor diameter and its effective length:

a) 8mm diameter / 17mm effective length

$F_s = (10.5\ \text{N/mm})\ (\pi \times 8\text{mm} \times 17\text{mm})$

$F_s = 4486\ \text{N} \approx 4.5\ \text{kN}$

b) 10mm diameter / 27mm effective length

$F_s = (10.5\ \text{N/mm})\ (\pi \times 10\text{mm} \times 27\text{mm})$

$F_s = 8906\ \text{N} \approx 8.9\text{kN}$

c) 12mm diameter / 27mm effective length

F_s = (10.5 N/mm) (π x 12mm x 27mm)

F_s = 10, 688 N ≈ 10.7 kN

d) 16mm diameter / 38mm effective length

F_s = (10.5 N/mm) (π x 16mm x 38mm)

F_s = 20, 056 N ≈ 20.1 kN

4.4 Comparison between Theoretical Result and Experimental Result

As shown above, the maximum pull-out fail load for 8mm anchor theoretically calculated was 4.5 kN while the experimental result for 8mm anchor produced 5.5 kN which 18% higher than the theoretical value. For 10mm anchor, the theoretical value showed 8.9 kN for the maximum pull-out fail load which about 2% higher than the experimental result that produced 8.7 kN. The experimental result for 12mm anchor was 11.5 kN while the theoretical value calculated was 10.7 kN which about 7% higher than the experimental result. For 16mm anchor, the experimental result produced 29.6 kN of maximum pull-out fail load which higher than the theoretical value that was 20.1 kN, and the percentage difference was about 32%. The comparison between the theoretical value and the experimental result was summarized as shown in Table 4.3.

Table 4.3: Percentage difference between theoretical and experimental result

Maximum Pull-out Load (kN)		
Theoretical Value	**Experimental Result**	**Percentage Difference (%)**
4.5	5.5	18
8.9	8.7	2
10.7	11.5	7
20.1	29.6	32

4.5 Discussion

From the graph shown in figure 4.2, there were slightly different between the readings of each anchor test where some of the anchor reached it pull-out failure at longer time than the other. This was maybe due to the some of the anchor was installed in different batch of concrete mix thus might provides different bonding force between the anchor and the concrete surface.

The working load for each size of the tested anchor was determined by dividing the strength of the anchor with the safety coefficient. The working load for each size of anchor was shown in Table 4.4. The value of 3.0 for safety coefficient was considered for the tested anchor because the selected value was in the range value of standard safety coefficient for less tried material under average conditions of environment, load and stress [12].

Table 4.4: Application load for tested anchor

Normal anchor diameter (mm)	Anchor tensile strength (kN)	Safety coefficient	Application load (kN)
8	5.5	3.0	1.8
10	8.7	3.0	2.9
12	11.5	3.0	3.8
16	29.6	3.0	9.9

The pull-out test result was then compared with the testing result provided by the supplier from other similar type of anchor produce in the market as shown in Table 4.5. From the both pull-out test result and available test result provided by supplier in the market, it was found that the tensile strength of the normal sleeve expansion anchor was not much different with the tensile strength of branded anchor.

The slight different in the tensile strength value was due to the safety coefficient applied by the supplier was higher than the value used by the author. The supplier of the available anchor applied average safety coefficient of 4.0 to 5.0. Higher safety coefficient provides high safety precaution for intended application and provide more confident for the user.

Table 4.5: Testing result from similar type of anchor [13]

Anchor size (inch)	**1/4"**	**5/16"**	**3/8"**	**1/2"**
Drill bit diameter (mm)	8	10	12	16
Anchor length (mm)	50	68	75	100
Max tightening torque (Nm)	8-10	20-25	40-50	90-100
Application load (kN)	2.0	3.3	4.5	7.1
Applied safety coefficient	4.5	4.5	4.5	4.5
Anchor tensile strength (kN)	9	14.85	20.25	31.95

Other parameters that might affect the different reading of the anchor tensile strength were the effective diameter and length of the anchor. Effective length of the anchor was the length of the anchor surface directly that makes a contact with the concrete surface as the anchor tighten into the concrete. The effective length depends on the torque applied to the anchor nut during the tightening process. The effective diameter and length of the anchor are proportionally linear to the anchor's tensile strength (maximum pull-out loading), where greater diameter and effective length provide greater surface area of anchor bonded with concrete surface, thus as the effective diameter and length increase, the tensile strength increase.

Moreover, the friction coefficient that been taken into account when calculating the theoretical value also might be the parameter that leads to slightly different reading with the pull-out test result. The friction coefficient depends on the condition of the material surface where rougher surface gives higher friction coefficient. Higher concrete grade normally required greater interlock between the aggregates. The different type of aggregates in the concrete provides rough surface that created the interlock between the concrete and the anchor sleeve. The friction coefficient was also linearly proportional with the anchor tensile strength where as higher friction between the anchor and the concrete surface, higher pull-out load was required.

Another parameter that might effects the experimental result was the embedded anchor's length. Besides the effective length of the anchor that create the interlock between the anchor and the concrete aggregate, the embedded length of anchor may also provides friction between the anchor and the concrete surface. Greater contact of the anchor' surface with the concrete provides higher friction thus could result in higher pull-out capacity.

CHAPTER 5
CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The tension capacity of normal sleeve anchor by pull-out test using the Universal Testing Machine (UTM) 1000kN was the first time carried out in this paper. The capacity of the anchor was determined in term of its tensile strength, which the friction failure between the anchor and the concrete surface. From the result, it is concluded that the larger diameter and effective length of anchor provide greater tensile capacity. The material used for producing the anchor might affect the tensile capacity. Due to that a consideration should be given by providing suitable safety coefficient for the working load. As the tensile strength of the normal sleeve anchor can be determine and known, provided with safety precaution measured in designing the application load, the normal sleeve anchor can now be widely used in the engineering or construction industry.

5.2 Recommendation

Since the normal sleeve anchor was proven to be quite reliable in term of its tension strength compared to the some branded and expensive anchor found in the market, other test are recommended to be done in determining the shear strength, both shear and tensile strength of the anchor in concrete to provide more specification of the anchor product. Other type of failure mode such as steel failure mode also can be considered to know the strength of the anchor's material before the anchor can be promoted to be widely applied in the engineering world.

5.3 Lesson Learned

Throughout the project, there were many problems occurs that has delayed the progress of the project schedule as shown in Appendix-F. Some of the problems were:

i. The available concrete mould in the UTP laboratory was not meeting the required dimension for testing, thus form-work made of ply wood was used to create the required mould.

ii. Availability of smaller concrete mixer has somehow delayed the progress which caused more mixes was done to cast the mould.

iii. Malfunctioning of the Universal Testing Machine occurred at scheduled time for testing. It took almost three (3) months before the rectification work was done and ready to be used again.

From the problems stated above, the lesson learned was that it is important to start the project as soon as possible as if there is any problem occurs, more time are available to tackle the problem. In addition to that, backup plan for each activities and quick action in finding alternative solutions for any problem also important to minimize the effect of the problem to the overall project's progress.

CHAPTER 6
ECONOMIC BENEFITS

6.1 Project or Research Cost

The overall project in determining the tension capacity of a normal sleeve anchor costed about RM217.50. The overall expenditure included the cost of the normal sleeve anchors as the testing subject, the cost of cement as one of the ingredient for concrete specimens, the cost of nails for form-work, the drilling bits to drill the concrete for installation of the anchor; and the charge for cut and weld the head adapter that fits to the pull-out testing machine. While, some others that not included in the overall cost like sand, aggregates, ply wood, timbers, the drill machine and the testing machine because all are provided in the laboratory and some are from recycled items.

6.2 Economic Value

On the business element or others that relevant in term of economic values, from this project, the tension capacity of a normal sleeve anchor was determined to be compared with the capacity of the branded and expensive anchor. The normal sleeve anchor can be found in the normal hardware shop is very cheap compared to the custom made anchor and other trusted brand anchor that has the same application of the tested anchor in this project which can be more than 30% cheaper. Even with cheaper price, the tension capacity of the normal sleeve anchor still has the required capability for its normal application that may use other types of anchorage system.

On the safety aspect, the important of knowing the capacity of the normal sleeve anchor is a must to ensure the safety of the users. Every application should use the suitable anchor capacity to support the intended load. It can cause danger and there is a possibility of damages to the intended application if the load applied is greater than the anchor capacity can handle. As if damages occur due to unsuitable application of the normal sleeve anchor, thus the cost of repair work might be unbearable as the impact of the failure might involve the environmental damages and hazard to human life.

REFERENCES

1. Rolf Eligehausen, Rainer Mallée, and John F. Silva (2006). *Anchorage in Concrete Construction*, Ernst & Sohn A Wiley Company. p1-75

2. S.P. Tastani (2002), *Experimental Evaluation Of The Direct Tension-Pullout Bond Test,* Bond in Concrete- from research to standards, Budapest

3. Jang Jung-Bum, Suh Yong-Pyo, Lee Jong-Rim (2003), *An Evaluation Of Tensile Capacity Of Cast-In-Place Anchor With Crack By Actual Model Test,* SMiRT 17, Prague, Czech Republic

4. Walter H., Baillet L., Brunet M. (1997), *Numerical Modeling Of The Dynamic Pull-out Failure Loads Of Anchors In Concrete Structures,* SMiRT 14, Lyon, France

5. Sherief S.S. Sakla, Ashraf F. Ashour (2005), *Prediction Of Tensile Capacity Of Single Adhesive Anchors Using Neural Networks,* Computers and Structures 83

6. B.G. Rabbat, M. ASCE and H.G. Russell (1985), *Friction Coefficient Of Steel On Concrete Or Grout,* Journal of Structural Engineering, Vol. 111

7. Matthew Miltenberger (August 2001), *Capacity Design Of Grouted Anchors,* SMiRT 16, Washington DC

8. ASTME E488 - 96(2003) *Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements,* Annual Book of ASTM Standards

9. ASTM C900 – 01 *Standard Test Method for Pullout Strength of Hardened Concrete,* Annual Book of ASTM Standards

10. BS 8110-1:1997, Section 3.12.8.2 *Anchorage Bond Stress*

11. BS EN 12812:2004, *Falsework- Performance Requirements and General Design, Informative Annex A*

12. http://www.roymech.co.uk/Useful_Tables/ARM/Safety_Factors.html [Accessed 7 April 2010]

13. http://www.syw-ind.com/sleeve.htm [Accessed 28 April 2010]

APPENDICES

APPENDIX A –
EQUATIONS FOR TENSION CAPACITY [7]

Fastener Steel Strength

$$N_s = f_y A_e \tag{1a}$$

$$N_s = f_{ut} A_e \tag{1b}$$

Where:
f_y = the reinforcing steel yield strength, MPa
f_{ut}= the steel fracture strength (not exceed $1.9f_y$ or 862 MPa)
A_e= the effective cross-sectional area of the steel fastener, mm^2

Adhesive Bond Strength

$$N_a = \tau'_a \pi d h_{ef} \tag{2a}$$

$$N_{n,a} = \psi_{Ng} \psi_{Ne} \psi_{Ncr} N_a \tag{2b}$$

Where:
N_a= basic adhesive bond strength to steel, N
$N_{n,a}$= nominal adhesive bond strength to steel, N

Plug Bond Strength

$$N_o = \tau'_o \pi d_o h_{ef} \tag{3a}$$

$$N_{n,o} = \psi_{Ng} \psi_{Ne} \psi_{Ncr} N_o \tag{3b}$$

Where:
N_o= basic plug bond strength to steel, N
$N_{n,o}$=nominal plug bond strength to steel, N

Characteristic Bond Strength

$$\tau' = \tau(1 - kC.O.V.) \tag{4}$$

Table 2.3: Coefficients for calculating the characteristic bond strength

Number of tests	k
3	5.311
4	3.957
5	3.400
6	3.092
10	2.568
15	2.329
20	2.208

Concrete Breakout Strength

$$N_c = 12.5\sqrt{f_c'}\,h_{ef}^{1.5} \quad \textbf{(when } h_{ef} < 280 \textbf{ mm)} \tag{5a}$$

$$N_c = 4.75\sqrt{f_c'}\,h_{ef}^{1.67} \quad \textbf{(when } 280 \le h_{ef} \le 635 \textbf{ mm)} \tag{5b}$$

$$N_{n,c} = \psi_{Ng}\psi_{Ne}\psi_{Ncr}N_c \tag{5c}$$

Where:
f'_c= specified concrete compressive strength, MPa
h_{ef}= effective embedment depth, mm
N_c= basic breakout strength, N
$N_{n,c}$= nominal concrete breakout strength, N

Anchor Head Bearing Strength

$$N_b = 11 A_b f_c' \tag{6a}$$

$$N_{n,b} = \psi_{Ncr} N_b \tag{6b}$$

Where:
A_b= anchor head bearing area, mm^2
N_b= basic anchor head bearing strength, N
$N_{n,b}$= nominal anchor head bearing strength, N

NOTE: When using plates or washers to increase A_b the diameter of the bearing area is increased by no more than twice the thickness of the plate or washer.

Side-face Blowout Strength

$$N_{sb} = 13.3c\sqrt{A_b}\sqrt{f_c'} \quad \text{(when } c < 0.4\, h_{ef}\text{)} \tag{7a}$$

$$N_{sbg} = \left(1 + \frac{s_o}{6c}\right) N_{sb} \quad \text{(when } c < 0.4\, h_{ef}\text{)} \tag{7b}$$

Where:
A_b= anchor head bearing area, mm^2
f'_c= specified concrete compressive strength, MPa
N_{sb}= side-face blowout strength of a single anchor, N
N_{sbg}= side-face blowout strength of a group, N
S_o= spacing between outer most fasteners along an edge in the group, mm

Design Strength Equations for Fasteners Loaded in Tension

$$\phi N_n = \min(\phi_s N_s; \phi N_{n,a}; \phi N_{n,o}; \phi N_{n,c}) \quad \text{(unheaded fastener)} \tag{8a}$$

$$\phi N_n = \min(\phi_s N_s; \phi N_{n,o}; \phi N_{n,c}; \phi N_{n,b}; \phi N_{sb}) \quad \text{(headed fastener)} \tag{8b}$$

APPENDIX B –
MIX DESIGN PROCESS

Part two The mix design process

5 Flow chart of procedures

The manner in which this method links the various factors involved in the process of designing a mix is shown as a flow chart in Figure 2. Also a suitable mix design form for recording the values derived is shown in Table 1*. It will be seen from the flow chart that initial information is divided into two categories:

a) Specified variables, the values of which are usually nominated in specifications, and
b) Additional information, which is normally available to the producer of the concrete.

This initial information is used in conjunction with reference data, which appear in the form of figures or tables in this publication, to evaluate a number of 'derived values' which are also subdivided into two categories:

a) The mix parameters, several of which form an intermediate step to the derivation of the second category, and
b) The final unit proportions, which are defined in terms of weights of materials required to produce 1 cubic metre of compacted concrete, expressed to the nearest 5 kg.

In order to clarify the sequence of operation, and for ease of reference, the flow process is divided into five stages. Each of these stages deals with a particular aspect of the design and ends with an important mix parameter or final unit proportions.

Stage 1 deals with strength leading to the free-water/cement ratio
Stage 2 deals with workability leading to the free-water content
Stage 3 combines the results of Stages 1 and 2 to give the cement content
Stage 4 deals with the determination of the total aggregate content
Stage 5 deals with the selection of the fine and coarse aggregate contents.

The mix design form shown in Table 1 is sub-divided into the same five stages and the separate item numbers correspond with the relevant boxes of the flow chart in Figure 2.

5.1 Selection of target water/cement ratio (Stage 1)

If previous information concerning the variability of strength tests comprises less than 40 results the standard deviation to be adopted should be that obtained from line A in Figure 3. If previous information is available consisting of 40 or more results, the standard deviation of such results may be used provided that this value is not less than the appropriate value obtained from line B. The margin can then be derived from calculation C1:

$$M = k \times s \qquad \ldots$$

where M = the margin
k = a value appropriate to the 'percentage defectives permitted below the characteristic strength (see Paragraph 4.4)
s = the standard deviation.

Calculation C2 determines the target mean strength (expresse to two significant figures):

$$f_m = f_c + M \qquad \ldots ($$

where f_m = the target mean strength
f_c = the specified characteristic strength
M = the margin.

Next, a value is obtained from Table 2 for the strength of a mix made with a free-water/cement ratio of 0·5 according to th specified age, the type of cement and the aggregate to be used. This strength value is then plotted on Figure 4 and a curve is drawn from this point and parallel to the printed curves until it intercepts a horizontal line passing through the ordinate representing the target mean strength. The corresponding value for the free-water/cement ratio can then be read from the abscissa. This should be compared with any maximum free-water/cement ratio that may be specified and the lower of these two values used.

5.2 Selection of free-water content (Stage 2)

Stage 2 consists simply of determining the free-water content from Table 3 depending upon the type and maximum size of the aggregate to give a concrete of the specified slump or V-B time.

5.3 Determination of cement content (Stage 3)

The cement content is determined from calculation C3:

$$\text{Cement content} = \frac{\text{free-water content}}{\text{free-water/cement ratio}} \qquad \ldots \text{C3}$$

The resulting value should be checked against any maximum or minimum value that may be specified. If the calculated cement content from C3 is below a specified minimum, this minimum value must be adopted. As a result, either the free-water/cement ratio of the mix may be less than that determined in Stage 1 or the free-water content may be greater than that determined in Stage 2. This will result in a concrete that has a mean strength somewhat higher than the target mean strength, or a workability somewhat higher than that initially chosen, depending on the choice made.

*The form is also printed at the end of this publication for ease of removal and subsequent use.

Conc ete mix design form

Stage	Item		Reference or calculation	Values
1	1.1	Characteristic strength	Specified	30 N/mm² at 28 days; Proportion defective 5 per cent
	1.2	Standard deviation	Fig 3	N/mm² or no data 8 N/mm²
	1.3	Margin	C1	(k = 1.64) × 8. = 13.12 N/mm²
	1.4	Target mean strength	C2	30 + 13.12 = 43.12 N/mm²
	1.5	Cement type	Specified	OPC/SRPC/RHPC
	1.6	Aggregate type: coarse Aggregate type: fine		crushed uncrushed
	1.7	Free-water/cement ratio	Table 2, Fig 4	0.54 — Use the lower value
	1.8	*Maximum free-water/cement ratio*	*Specified*	0.6
2	2.1	Slump or V-B	Specified	Slump 60-180 mm or V-B 0-3 s
	2.2	Maximum aggregate size	Specified	20 mm
	2.3	Free-water content	Table 3	205 kg/m³
3	3.1	Cement content	C3 $\frac{w}{c}$	205 ÷ 0.54 = 379.63 kg/m³
	3.2	*Maximum cement content*	*Specified*	— kg/m³
	3.3	*Minimum cement content*	*Specified*	290 kg/m³ — Use if greater than Item 3.1 and calculate Item 3.4
	3.4	Modified free-water/cement ratio		
4	4.1	Relative density of aggregate (SSD)		2.7 known/assumed
	4.2	Concrete density	Fig 5	2410 kg/m³
	4.3	Total aggregate content	C4	2410 − 379.63 − 205 = 1825.37 kg/m³
5	5.1	Grading of fine aggregate	BS 882	Zone 3
	5.2	Proportion of fine aggregate	Fig 6	31% 39% / 35% per cent
	5.3	Fine aggregate content	C5	0.35 × 1825.37 = 638.88 kg/m³
	5.4	Coarse aggregate content	C5	1825.37 − 638.88 = 1186.49 kg/m³

Quantities	Cement (kg)	Water (kg or l)	Fine aggregate (kg)	Coarse aggregate (kg)
per m³ (to nearest 5 kg)	380	205	640	1190
per trial mix of 0.042 m³ (2 samples + 6 test cubes)	15.96	8.61	26.88	49.98

Items in italics are optional limiting values that may be specified.

1 N/mm² = 1 MN/m² = 1 MPa.

OPC = ordinary Portland cement; SRPC = sulphate-resisting Portland cement; RHPC = rapid-hardening Portland cement.

Relative density = specific gravity.

SSD = based on a saturated surface-dry basis

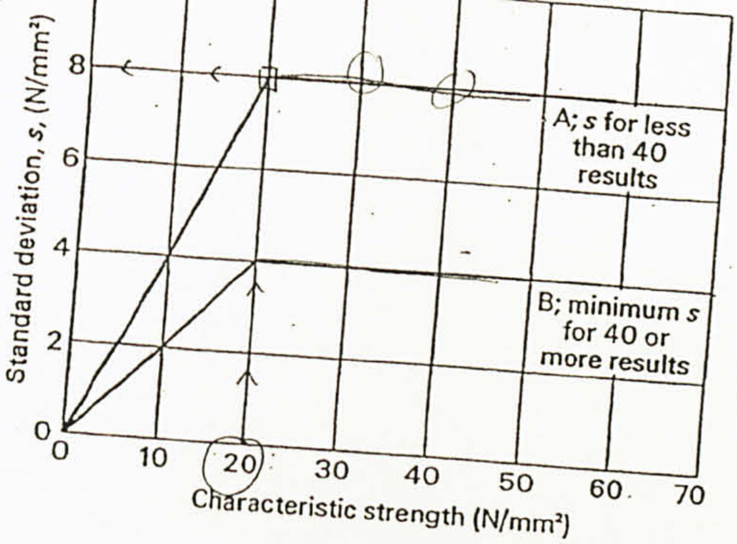


Figure 3 Relationship between standard deviation and characteristic strength

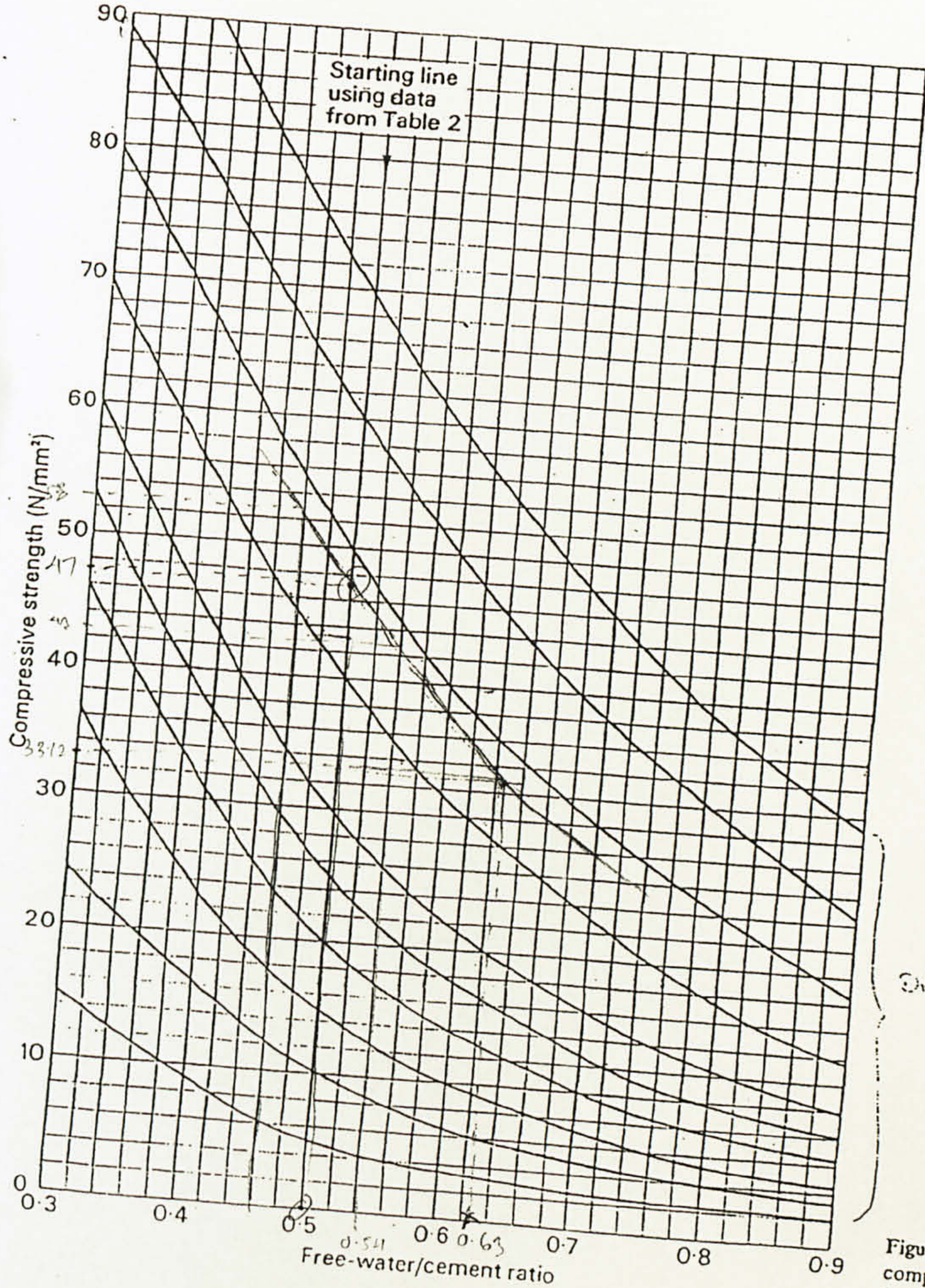


Figure 4 Relationship between compressive strength and free-water/cement ratio

(i) concrete mix
(ii) cement
(iii) aggregates

... compressive strengths (N/mm²) of concrete mixes made with a free-water/cement ratio of $\boxed{0{\cdot}5}$

Type of cement	Type of coarse aggregate	Compressive strengths (N/mm²) Age (days) 3	7	28	91
Ordinary Portland (OPC) or sulphate-resisting Portland (SRPC)	Uncrushed	18	27	40	48
	Crushed	23	33	$\boxed{47}$	55
Rapid-hardening Portland (RHPC)	Uncrushed	25	34	46	53
	Crushed	30	40	53	60

1 N/mm² = 1 MN/m² = 1 MPa (see footnote on page 8).

Table 3 Approximate free-water contents (kg/m³) required to give various levels of workability

Slump (mm)		0–10	10–30	30–60	$\boxed{60–180}$
V-B (s)		>12	6–12	3–6	$\boxed{0–3}$
Maximum size of aggregate (mm)	Type of aggregate				
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
$\boxed{20}$	Uncrushed	135	160	180	$\boxed{195}$
	Crushed	170	190	210	$\boxed{225}$
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205

Note: When coarse and fine aggregates of different types are used, the free-water content is estimated by the expression

$$\tfrac{2}{3} W_f + \tfrac{1}{3} W_c$$

where W_f = free-water content appropriate to type of fine aggregate
and W_c = free-water content appropriate to type of coarse aggregate.

$$Fw = \frac{2}{3}\times 195 + \frac{1}{3}\times 225$$

$$= 210$$

On the other hand, if the design method indicates a cement content that is higher than a specified maximum then it is probable that the specification cannot be met simultaneously on strength and workability requirements with the selected materials. Consideration should then be given to changing the type of cement, the type and maximum size of aggregate or the level of workability of the concrete.

5.4 Determination of total aggregate content (Stage 4)

Stage 4 requires an estimate of the density of the fully compacted concrete which is obtained from Figure 5 depending upon the free-water content and the relative density* of the combined aggregate. If no information is available regarding the relative density of the aggregate an approximation can be made by assuming a value of 2·6 for uncrushed aggregate and 2·7 for crushed aggregate. From this estimated density of the concrete the total aggregate content is determined from calculation C4:

$$\text{Total aggregate content (saturated and surface-dry)} = D - W_c - W_{FW} \qquad \ldots \text{C4}$$

where D = the wet density of concrete (kg/m³)
W_c = the cement content (kg/m³)
W_{FW} = the free-water content (kg/m³).

5.5 Selection of fine and coarse aggregate contents (Stage 5)

Stage 5 involves deciding how much of the total aggregate should consist of material smaller than 5 mm, i.e. the sand or fine aggregate content. Figure 6 shows recommended ranges for the proportion of fine aggregate depending on the maximum size of aggregate, the workability level, the grading zone of the fine aggregate and the free-water/cement ratio. The best proportion of fines to use in a given mix will depend on the shape of the particular aggregate, the actual grading relative to the zone limits as defined in BS 882 and the use to which the concrete is to be put. However, adoption of a proportion within the bands recommended in Figure 6 will generally give a satisfactory concrete in the first trial mix which can then be adjusted as required for the exact conditions prevailing.

The final calculation, C5, to determine the fine and coarse aggregate contents, consists of multiplying the value obtained from Figure 6 by the total aggregate content derived in Stage 4:

$$\left.\begin{array}{l} \text{Fine aggregate content} = \\ \quad \text{total aggregate content} \times \text{proportion of fines} \\ \text{Coarse aggregate content} = \\ \quad \text{total aggregate content} - \text{fine aggregate content} \end{array}\right\} \quad \ldots \text{C5}$$

The coarse aggregate content itself can be subdivided if single sized 10, 20 and 40 mm materials are to be combined. Again, the best proportions will depend on aggregate shape and concrete usage but the following ratios are suggested as a general guide:

1:2 for combination of 10 and 20 mm material
1:1·5:3 for combination of 10, 20 and 40 mm material.

*The internationally known term 'relative density' used in this publication is synonymous with 'specific gravity' and is the ratio of the mass of a given volume of substance to the mass of an equal volume of water.

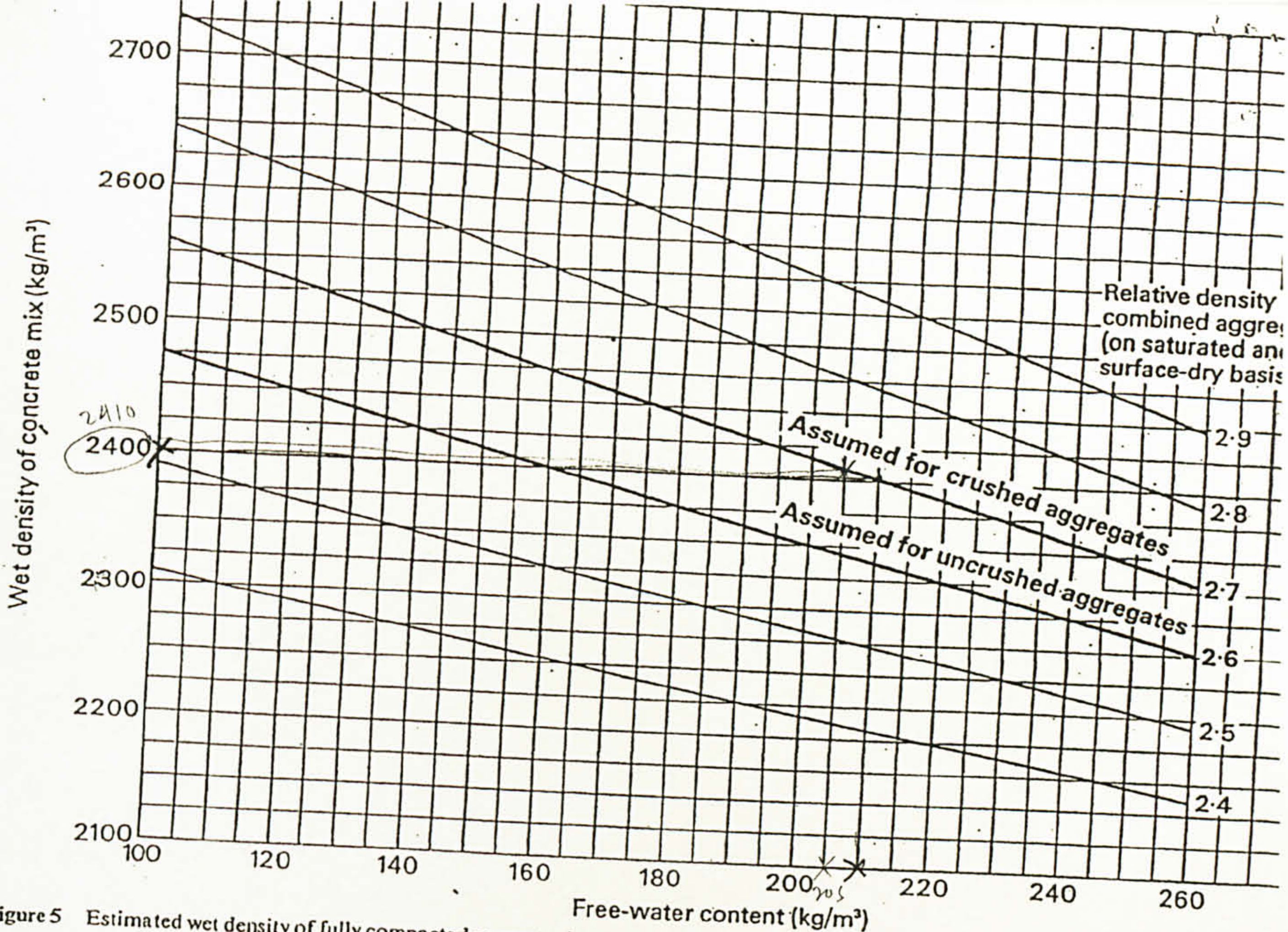


Figure 5 Estimated wet density of fully compacted concrete

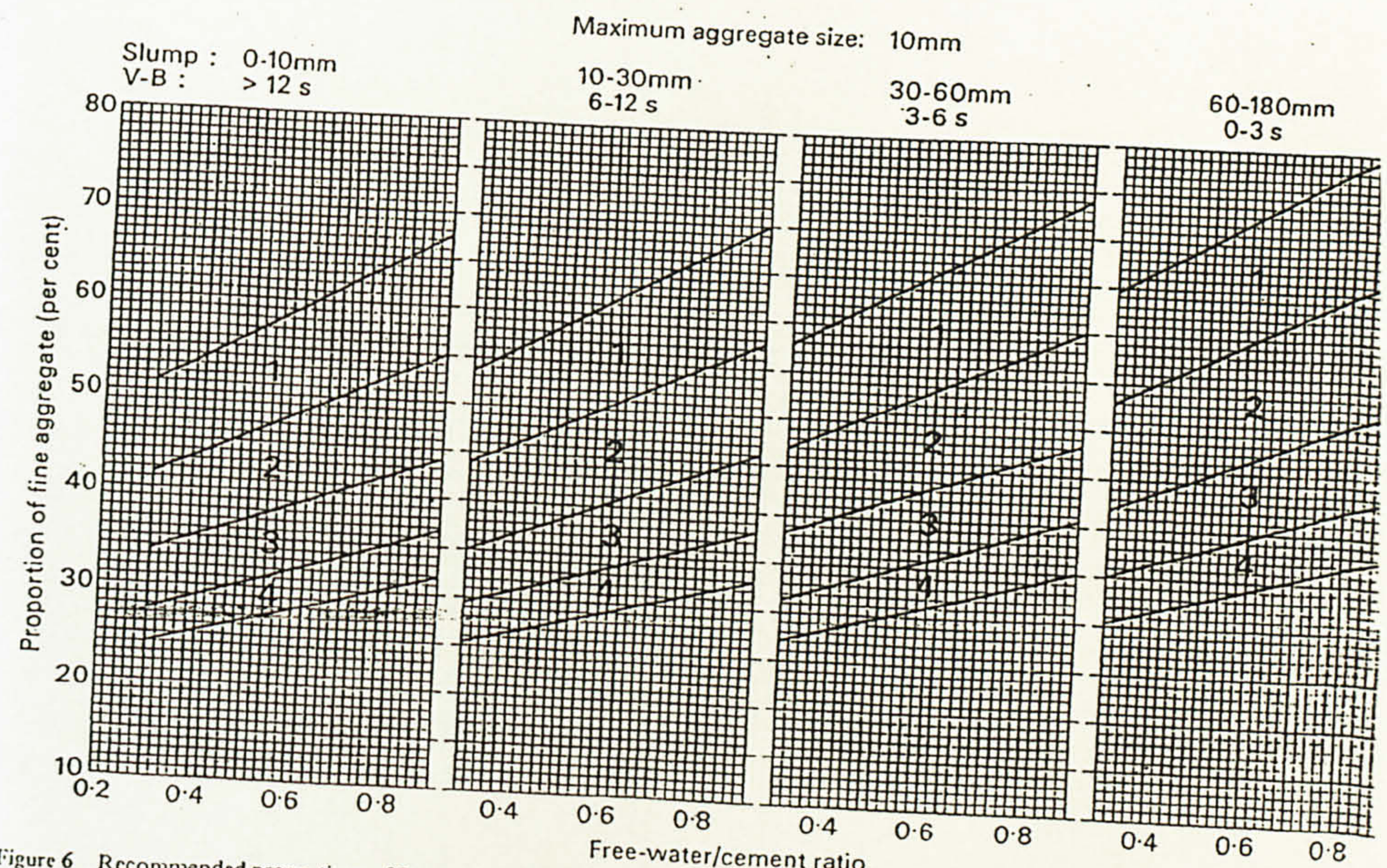


Figure 6 Recommended proportions of fine aggregate for BS 882 grading zones 1, 2, 3 and 4

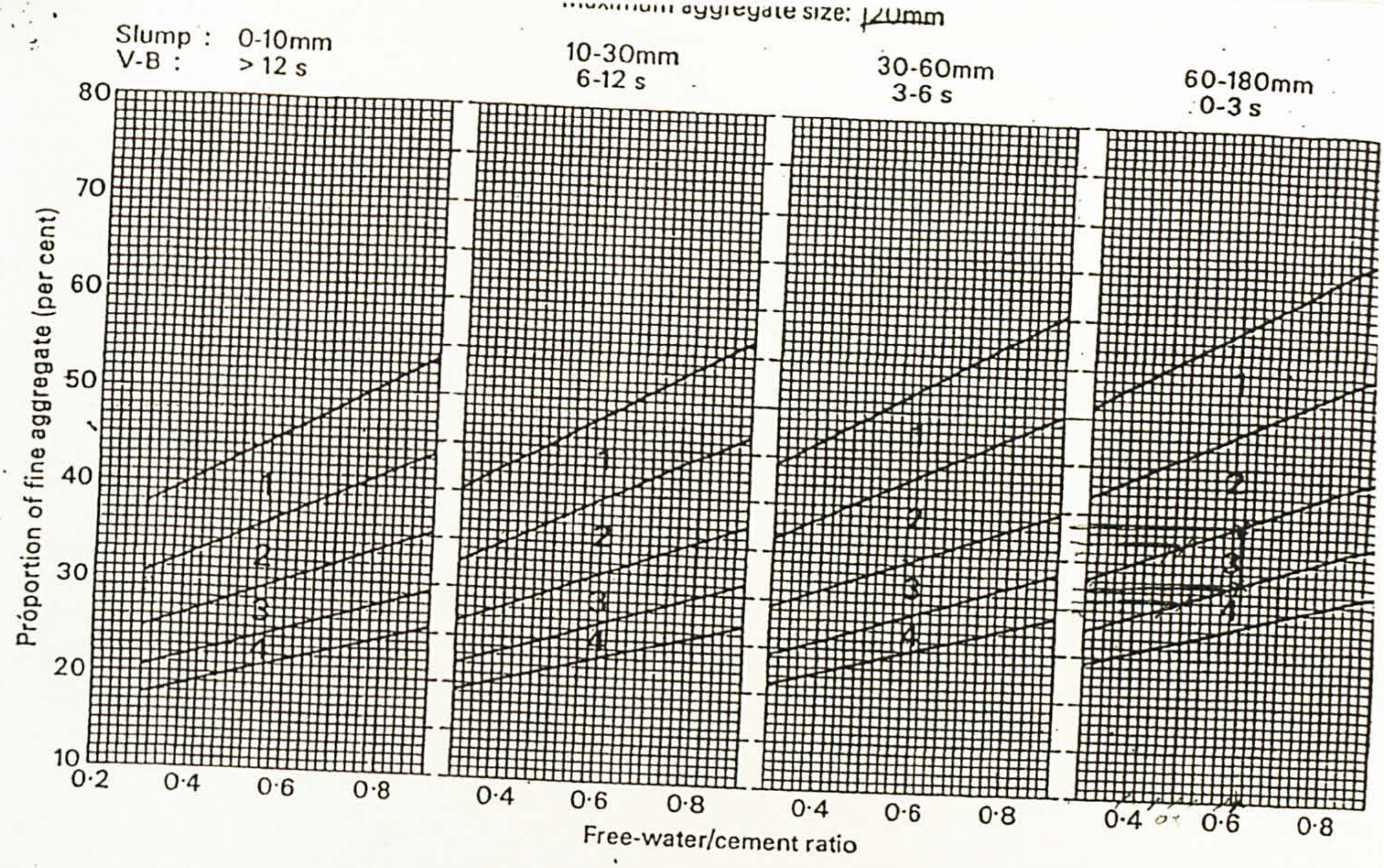


Figure 6 (*continued*)

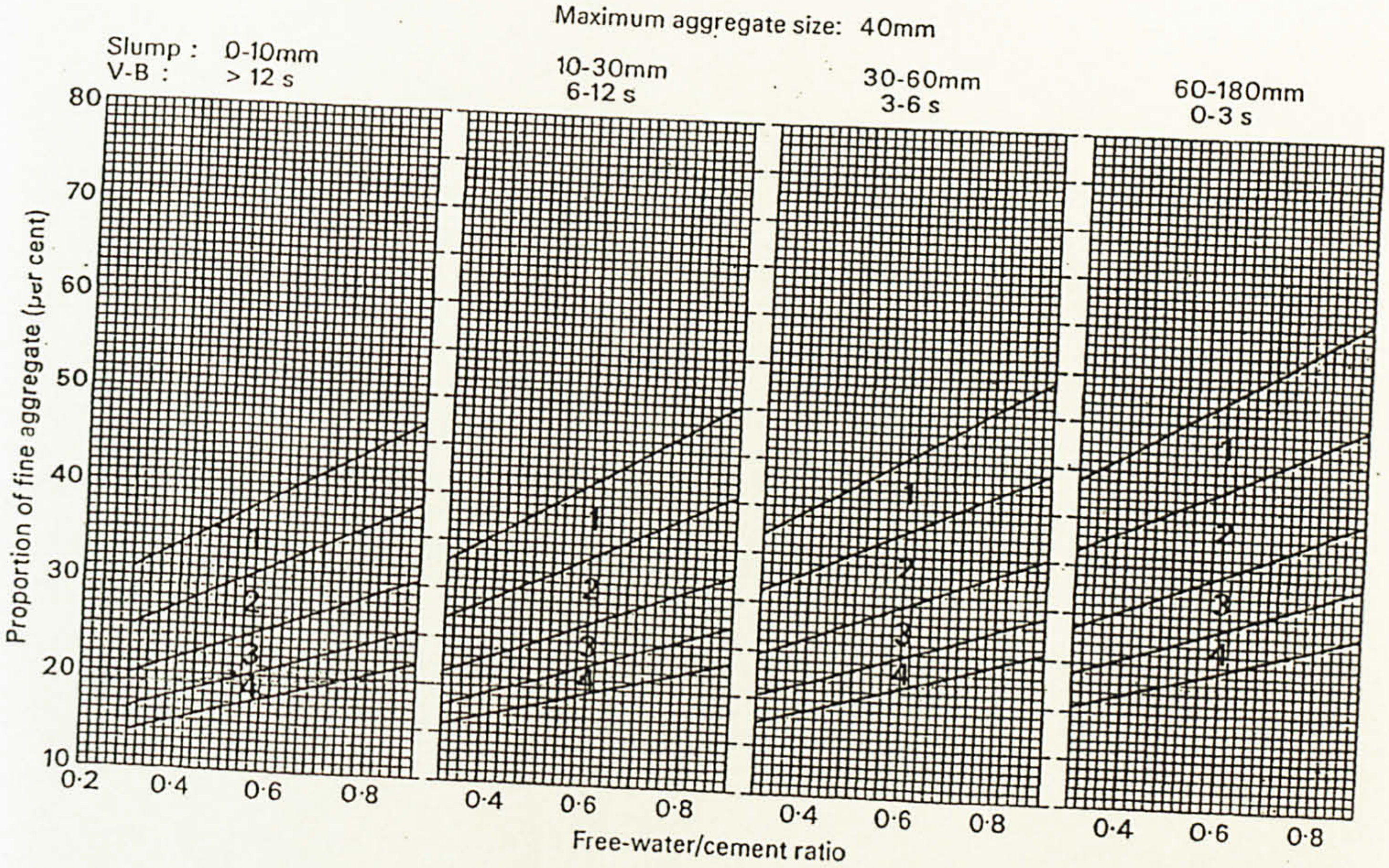


Figure 6 (*continued*)

MIXING AND SAMPLING FRESH CONCRETE

1. OBJECTIVE

Mixing and sampling fresh concrete in the laboratory (as recommended by **BS 1881: Part 125:1986**)

2. APPARATUS

A non-porous timber or metal platform, a pair of shovels, a steel hand scoop, measuring cylinder and a small concrete mixer (if machine mix)

3. PROCEDURE

a. Weight the quantities of cement, sand and course aggregate to make 1:2:4 concrete mix at water ratio of 0.6

b. Hand Mixing

i. Mix cement and sand first until uniform on the non-porous platform
ii. Pour course aggregate and mix thoroughly until uniform
iii. Form a hole in the middle and add water in the hole. Mix thoroughly for 3 minutes or until the mixture appears uniform in color.

c. Machine Mixing

i. Wet the concrete mixer.
ii. Pour aggregate and mix for 25 second.
iii. Add half of water and mix for 1 minute and leave for 8 minutes.
iv. Add cement and mix for 1 minute.
v. Add remaining water available and mix for 1 minute.
vi. Stop the machine and do hand mixing to ensure homogeneity.
vii. Pour out the concrete onto the non porous surface.

4. PRECAUTIONS

a. The room temperature should be approximately 25-27 C

b. Make sure that fine and aggregate are dry. If they are wet find the content of the aggregates to determine the quantity of water required.

UNIVERSITY TECHNOLOGY PETRONAS
CIVIL ENGINEERING PROGRAMME
BANDAR SERI ISKANDAR
31750 TRONOH
PERAK DARUL RIDZUAN

SLUMP TEST - TEST FOR WORKABILITY

1. OBJECTIVE

To measure the workability of a sample from a batch of fresh concrete of a given mix (as recommended by **BS 1881: Part 102:1983**)

2. THEORY

The measurement of the workability of fresh concrete is important in assessing the practicality of compacting the mix and also in maintaining consistency throughout the job.

3. APPARATUS

Truncated conical mould 100mm in diameter at the top, 200mm at bottom and 300mm high, with a steel tamping rod (16mm diameter & 600mm long), rounded at one end, a scoop, a steel ruler and a steel trowel.

4. PROCEDURE

a. Clean the inside mould and place it on a hard, flat and nonabsorbent surface.

b. Take a representative sample (about15kg) from a fresh concrete mix.

c. Fill the mould in four layers of concrete of approximately equal depth (each layer is about 75mm). Each layer is rodded 25 times with the rounded end of a steel rod. Make sure each rodding passes through the height of each layer.

d. After the top layer has been rodded, the surface of the concrete is struck off with a trowel to level up with the top of the mould.

e. Clean away any spillage of the concrete around the base of the mould.

f. Carefully and slowly lift the mould vertically from the concrete. Invert the mould and place it next to the moulded concrete. The concrete will slump.

g. Place the rod across the top of the mould.

h. The slump is the difference between the height of the slumped concrete and the mould. Using the steel ruler, measure the slump from the top of the concrete to the underside of the rod.

i. Record the slump to the nearest 5mm.

5. PRECAUTIONS

a. The test should be done 6 minutes after water is added to dry concrete mix(as recommended by BS 1881-Part 102,1983)
b. During filling the mould must be firmly pressed against its base
c. The rodding should be applied uniformly through the entire area of the concrete.
d. The bottom layer should be rodded throughout its depth.
e. Vibrations from nearby machine might increase the subsidence.
f. If the specimen collapses off laterally, repeat the test with another sample of the same batch of concrete.
g. If, in repeat test, the specimen should again collapse or shear, record the slump.

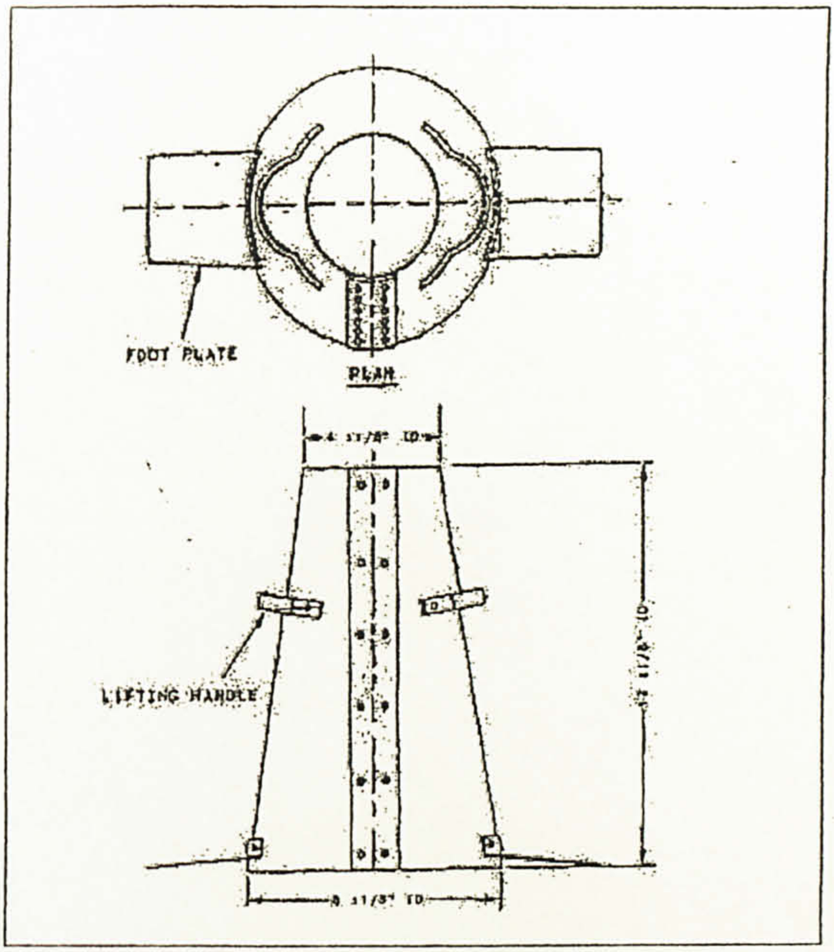


FIGURE 1: A Slump Cone

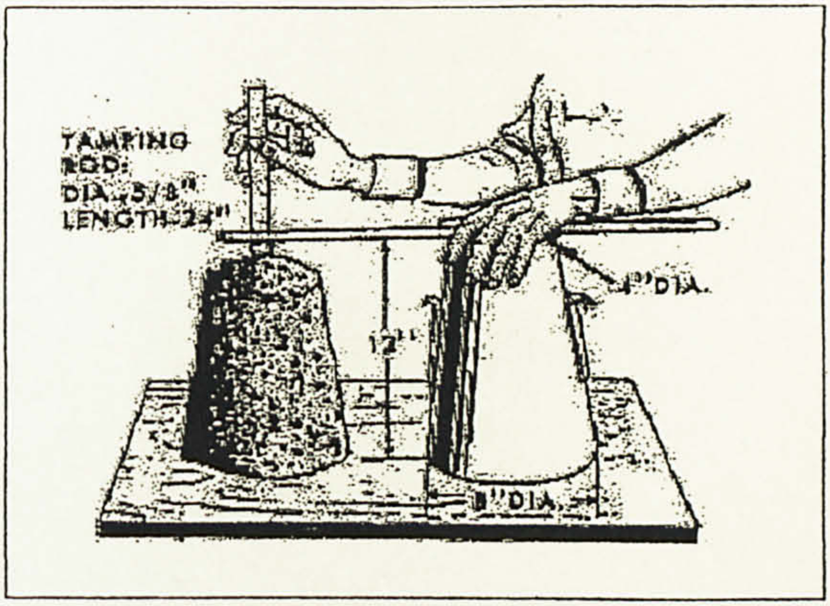
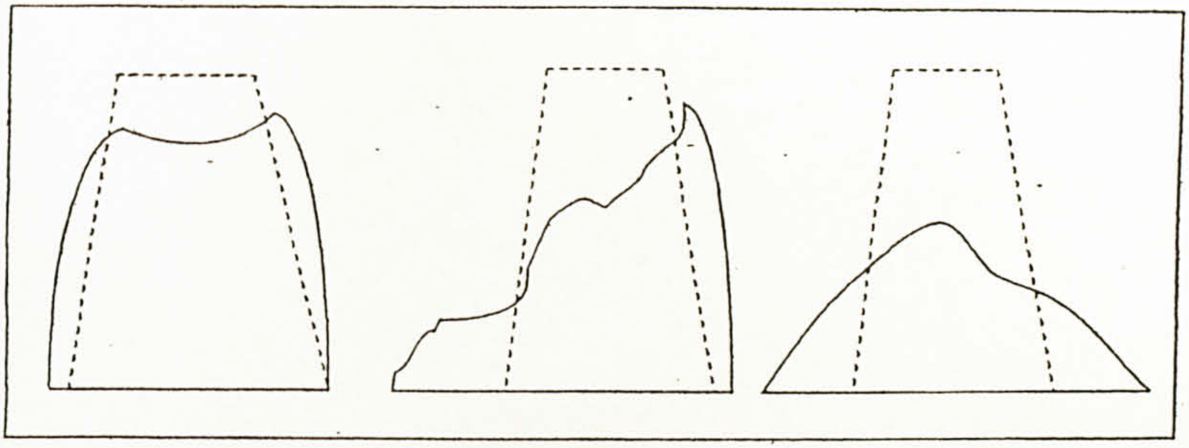


FIGURE 2: Measure of Slump



(a) True Slump (b) Shear Slump (c) Collapse Slump

Figure 3(a), 5(b) and 5(c): Types of slump

TABLE 1: Slump Test Apparatus & Their Remarks

INDEX	APPARATUS	REMARKS
1	Slump Cone	Heavy Gauge sheet steel, 4" top diameter, 8" bottom diameter, 12" height
2	Inspection Scale	Machine steel, 0-10 cm slump measurement, 1 cm increment
3	Base Plate	Steel sheet, carrying handle, 600mm x 600mm x 5mm
4	Scoop	Cast Aluminum
5	Trowel	Pointed Type
6	Brush	Steel wire
7	Tamping Rod	Machine steel, galvanized 16mm diameter, 600 mm length

UNIVERSITI TEKNOLOGI PETRONAS

UNIVERSITY TECHNOLOGY PETRONAS
CIVIL ENGINEERING PROGRAMME
BANDAR SERI ISKANDAR
31750 TRONOH
PERAK DARUL RIDZUAN

MAKING AND CURING CUBES AND TEST BEAMS

1. OBJECTIVE

To cast and cure test cubes and test beams of 150mm standard sizes of a given mix (as recommended by **BS 1881: Part 111:1983**)

2. APPARATUS

150mm x 150mm x 150mm internal size of steel mould for the test cubes, 150mm x 750mm steel moulds for the test beams, a 300mm long steel tamping bar with a rimming face 25mm square and a steel trowel.

3. PROCEDURE

a. Brush the inner faces of moulds with oil and tighten the screws.

b. Fill the mould with concrete sample in layers of 50mm deep approximately.

c. Tamp each layer with the square face steel tamping bar 25times for test cube and 175 times for test beam. Make sure each tamping passes through of each layer.

d. After the top layer has been tamped, the surface of the concrete is struck off level with the top off the mould with a trowel.

e. Using a nail mark the top surface of the concrete test cube to indicate number and date of casting.

f. Cover the moulds with polythene sheet or damp cloth to prevent evaporation and keep in the curing room for 24hours.

g. After 24hours the concrete specimen should be removed from the moulds and stored in the curing tank until they are to be tested at a temperature of 20+ 5 °C.

h. Preferred ages for test are 7days and 28days

i. At least 2 specimens are made for each mix.

4. PRECAUTIONS

a. The fresh concrete samples should be tested for workability before casting.
b. Test specimen should be made as soon as possible after concrete is mixed.
c. The specimen in the mould should not be moved within the first few hours after casting as this may lead to segregation and excessive bleeding of the concrete.
d. If there is no curing room, place the specimen in the mould in the laboratory which will be free from vibration.

UNIVERSITI TEKNOLOGI PETRONAS	UNIVERSITY TECHNOLOGY PETRONAS CIVIL ENGINEERING PROGRAMME BANDAR SERI ISKANDAR 31750 TRONOH PERAK DARUL RIDZUAN

COMPRESSIVE STRENGTH TEST CUBES – TEST FOR STRENGTH

1. OBJECTIVE

To determine the compressive strength (Crushing strength) of concrete according to **BS 1881: Part 116: 1983**

2. THEORY

One of the most important properties of concrete is its strength in compression. The strength in compression has a definite relationship with all other properties of concrete. The other properties are improved with the improvement in compressive strength.

The compressive strength is taken as the maximum compressive load it can be carry per unit area. Compressive strength tests for concrete with maximum size of aggregate up to 40mm are usually conducted on 150mm cubes.

3. APPARATUS

Compression Testing Machine (it complies with the requirement of **BS 1610**)

4. PROCEDURE

a. Remove the specimen from curing tank and wipe surface water and grit off the specimen.

b. Weight each specimen to the nearest kg.

c. Clean the top and lower platens of the testing machine. Carefully center the cube on the lower platen and ensure that the load will be applied to two opposite cast faces of the cube.

d. Without shock, apply and increase the load continuously at a nominal rate within the range 0.2N/mm^{2}s to 0.4 N/mm^2 until no greater load can be sustained. Record the maximum load applied to the cube.

e. Note the type of failure and appearance of cracks.

f. Calculate the compressive strength of each cube by dividing the maximum load by the cross sectional area. Express the results to the nearest 0.5 N/mm^2

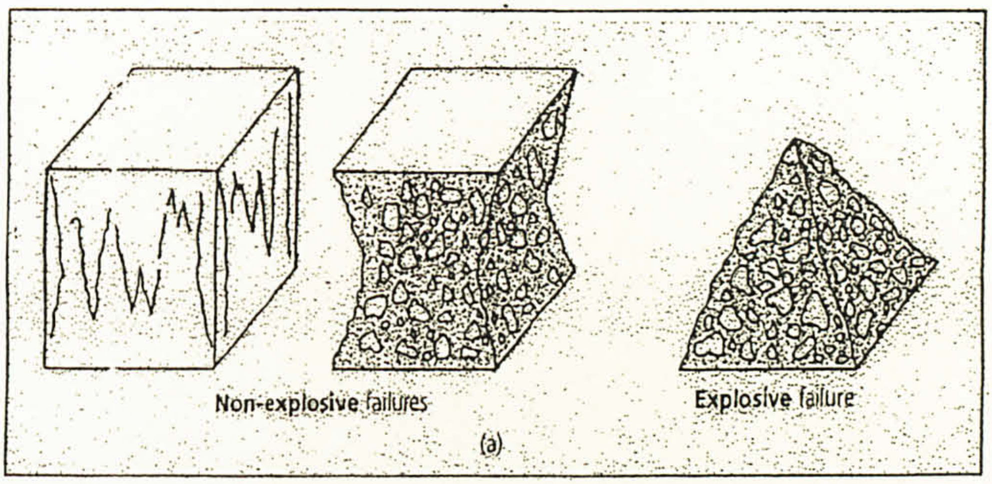


FIGURE 5: The outcome of cube test – normal case

APPENDIX C –
CONCRETE COMPRESSIVE TEST RESULT

CONCRETE COMPRESSIVE STRESS

Grade (Batch Mix)	Slump test (types/measures)	Date Cast	Age (Days)	Dimension (mm)	Weight (kg)	Weight/ Vol (kg/m^3)	Fail Load (kN)	Strength (N/mm^2)	Average Strength (N/mm^2)
C30 (1)	collapse/-	10/2/2010	7	150	8.0	2370.37	382.6	17.01	17.08
C30 (1)	collapse/-	10/2/2010	7	150	8.2	2429.63	385.6	17.14	
C30 (1)	collapse/-	10/2/2010	7	150	8.0	2370.37	384.2	17.08	
C30 (2)	true/25mm	12/2/2010	7	150	8.3	2459.26	802.3	35.66	35.75
C30 (2)	true/25mm	12/2/2010	7	150	8.3	2459.26	805.1	35.78	
C30 (2)	true/25mm	12/2/2010	7	150	8.3	2459.26	805.7	35.81	
C30 (3)	true/34mm	19/2/2010	7	150	7.9	2340.74	587.6	26.12	25.88
C30 (3)	true/34mm	19/2/2010	7	150	8.0	2370.37	580.2	25.79	
C30 (3)	true/34mm	19/2/2010	7	150	8.0	2370.37	578.7	25.72	
C30 (4)	true/40mm	24/2/2010	7	150	8.0	2370.37	507.1	22.54	24.50
C30 (4)	true/40mm	24/2/2010	7	150	8.2	2429.63	577.5	25.67	
C30 (4)	true/40mm	24/2/2010	7	150	8.2	2429.63	568.9	25.28	
C30 (5)	collapse/-	26/2/2010	7	150	8.1	2400.00	548.3	24.37	23.73
C30 (5)	collapse/-	26/2/2010	7	150	8.0	2370.37	511.5	22.73	
C30 (5)	collapse/-	26/2/2010	7	150	8.1	2400.00	541.9	24.08	
C30 (6)	true/20mm	2/3/2010	7	150	8.3	2459.26	719.3	31.97	31.98
C30 (6)	true/20mm	2/3/2010	7	150	8.1	2400.00	719.0	31.95	
C30 (6)	true/20mm	2/3/2010	7	150	8.3	2459.26	720.4	32.02	
C30 (7)	true/25mm	4/3/2010	7	150	8.3	2459.26	681.5	30.29	29.67
C30 (7)	true/25mm	4/3/2010	7	150	8.2	2429.63	656.9	29.2	
C30 (7)	true/25mm	4/3/2010	7	150	8.2	2429.63	664.3	29.52	
C30 (8)	true/35mm	8/3/2010	7	150	8.4	2488.89	663.2	29.48	25.55
C30 (8)	true/35mm	8/3/2010	7	150	8.3	2459.26	513.3	22.82	
C30 (8)	true/35mm	8/3/2010	7	150	8.3	2459.26	548.0	24.35	

Grade (Batch Mix)	Slump test (types/measures)	Date Cast	Age (Days)	Dimension (mm)	Weight (kg)	Weight/ Vol (kg/m^3)	Fail Load (kN)	Strength (N/mm^2)	Average Strength (N/mm^2)
C30 (1)	collapse/-	10/2/2010	28	150	7.9	2340.74	511.6	22.74	23.02
C30 (1)	collapse/-	10/2/2010	28	150	7.8	2311.11	523.4	23.26	
C30 (1)	collapse/-	10/2/2010	28	150	8.0	2370.37	518.7	23.05	
C30 (2)	true/25mm	12/2/2010	28	150	8.2	2429.63	822.2	36.54	37.42
C30 (2)	true/25mm	12/2/2010	28	150	7.9	2340.74	763.4	33.93	
C30 (2)	true/25mm	12/2/2010	28	150	8.3	2459.26	940.3	41.79	
C30 (3)	true/34mm	19/2/2010	28	150	7.9	2340.74	726.6	32.29	34.50
C30 (3)	true/34mm	19/2/2010	28	150	7.8	2311.11	842.7	37.45	
C30 (3)	true/34mm	19/2/2010	28	150	7.4	2192.59	759.6	33.76	
C30 (4)	true/40mm	24/2/2010	28	150	8.1	2400.00	690.2	30.68	31.87
C30 (4)	true/40mm	24/2/2010	28	150	8.1	2400.00	795.7	35.36	
C30 (4)	true/40mm	24/2/2010	28	150	8.2	2429.63	665.3	29.57	
C30 (5)	collapse/-	26/2/2010	28	150	8.0	2370.37	598.1	26.58	26.42
C30 (5)	collapse/-	26/2/2010	28	150	8.1	2400.00	606.8	26.97	
C30 (5)	collapse/-	26/2/2010	28	150	8.1	2400.00	578.2	25.70	
C30 (6)	true/20mm	2/3/2010	28	150	8.3	2459.26	1069.0	47.53	48.60
C30 (6)	true/20mm	2/3/2010	28	150	8.3	2459.26	1154.0	51.30	
C30 (6)	true/20mm	2/3/2010	28	150	8.2	2429.63	1057.0	46.97	
C30 (7)	true/25mm	4/3/2010	28	150	8.3	2459.26	737.0	32.76	35.03
C30 (7)	true/25mm	4/3/2010	28	150	8.2	2429.63	746.6	33.18	
C30 (7)	true/25mm	4/3/2010	28	150	8.3	2459.26	880.7	39.14	
C30 (8)	true/35mm	8/3/2010	28	150	8.2	2429.63	818.4	36.38	31.19
C30 (8)	true/35mm	8/3/2010	28	150	8.4	2488.89	612.2	27.21	
C30 (8)	true/35mm	8/3/2010	28	150	8.2	2429.63	674.3	29.97	

APPENDIX D –

ASTM E 488-96

STANDARD TESTING METHOD FOR STRENGTH OF ANCHOR IN CONCRETE AND MASONRY ELEMENTS [8]

Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements[1]

This standard is issued under the fixed designation E 488; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense.

1. Scope

1.1 These test methods cover procedures for determining the static, seismic, fatigue and shock, tensile and shear strengths of post-installed and cast-in-place anchorage systems in structural members made of concrete or structural members made of masonry. Only those tests required by the specifying authority need to be performed.

1.2 These test methods are intended for use with such anchorage devices designed to be installed perpendicular to a plane surface of the structural member.

1.3 Whereas combined tension and shear as well as torsion tests are performed under special conditions, such tests are not covered in the methods described herein.

1.4 While individual procedures are given for static, seismic, fatigue and shock testing, nothing herein shall preclude the use of combined testing conditions which incorporate two or more of these types of tests, (such as seismic, fatigue and shock tests in series), since the same equipment is used for each of these tests.

1.5 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

E 4 Practices for Force Verification of Testing Machines[2]

E 171 Specification for Standard Atmospheres for Conditioning and Testing Flexible Barrier Materials[3]

E 468 Practice for Presentation of Constant Amplitude Fatigue Test Results for Metallic Materials[2]

E 575 Practice for Reporting Data from Structural Tests of Building Constructions, Elements, Connections, and Assemblies[4]

3. Terminology

3.1 *Definitions of Terms Specific to This Standard:*

3.1.1 *adhesive anchor*—a post-installed anchor that derives its holding strength from the chemical compound between the wall of the hole and the anchor rods. The materials used include epoxy, cementitious material, polyester resin, and other similar types.

3.1.2 *anchor spacing*—the distance between anchors measured centerline to centerline, in mm (in.); also, the minimum distance between reaction points of the test frame.

3.1.3 *cast-in-place anchor*—an anchor that is installed prior to the placement of concrete and derives its holding strength from plates, lugs, or other protrusions that are cast into the concrete.

3.1.4 *displacement*—movement of an anchor relative to the structural member. For tension tests, displacement is measured along the axis of the anchor, and for shear tests, displacement is measured perpendicular to the axis of the anchor, in mm (in.).

3.1.5 *edge distance*—side cover distance or the distance from the centerline of an anchor to the nearest edge of a structural member, in mm (in.); also, minimum distance from the centerline to the test frame.

3.1.6 *embedment depth*—distance from the test member surface to the installed end of the anchor, in mm (in.), prior to the setting of the anchor.

3.1.7 *expansion anchor*—a post-installed anchor that derives its holding strength through a mechanically expanded system which exerts forces against the sides of the drilled hole.

3.1.8 *fatigue test*—a laboratory test that applies repeated load cycles to an anchorage system for the purpose of determining the fatigue life or fatigue strength of that system.

3.1.9 *LVDT*—a linear variable differential transformer used for measuring the displacement or movement of an anchor or anchor system.

---

[1] These test methods are under the jurisdiction of ASTM Committee E06 on Performance of Buildings and are the direct responsibility of Subcommittee E06.13 on Structural Performance of Connections in Building Construction .

Current edition approved May 10, 2003. Published June 2003. Originally approved in 1976. Last previous edition approved in 1996 as E 488 – 96.

[2] *Annual Book of ASTM Standards*, Vol 03.01.

[3] *Annual Book of ASTM Standards*, Vol 15.09.

[4] *Annual Book of ASTM Standards*, Vol 04.11.

3.1.10 *post-installed anchor*—an anchor that is installed after the placement and hardening of concrete.

3.1.11 *run-out*—a condition where failure did not occur at a specified number of load cycles in a fatigue test.

3.1.12 *safe working loads*—the allowable or design load obtained by applying factors of safety to the ultimate load of the anchorage device, kN (lbf).

3.1.13 *seismic test*—a laboratory test that applies load cycles of varying magnitude and frequency to an anchorage system for the purpose of simulating a seismic event (earthquake).

3.1.14 *shear test*—a test in which an anchor is loaded perpendicular to the axis of the anchor and parallel to the surface of the structural member.

3.1.15 *shock test*—a laboratory test that simulates shock loads on an anchorage system by the application of a short duration external load.

3.1.16 *static test*—a test in which a load is slowly applied to an anchor according to a specified rate such that the anchor receives one loading cycle.

3.1.17 *structural member*—the material in which the anchor is installed and which resists forces from the anchor.

3.1.18 *tensile test*—a test in which an anchor is loaded axially in tension.

3.1.19 *undercut anchor*—a post-installed anchor that derives its holding strength from an expansion of an embedded portion of the anchor into a portion of the hole that is larger in diameter than the portion of the hole between the enlarged section and the surface of the structural member. The enlarged diameter section of the hole is predrilled or enlarged by an expansion process during setting of the anchor.

3.2 *Symbols*:

h_{ef}	= effective depth of embedment of an anchor in mm (in.).
F_s	= safe working load in kN (lbf).
h	= thickness of the structural member in mm (in.).
h_v	= anchor embedment depth in mm (in.).
s	= anchor spacing in mm (in.) measured centerline to centerline.
c	= edge distance in mm (in.) measured from centerline of anchor to edge.
d	= nominal anchor diameter in mm (in.).
Δ_T	= uncorrected displacement for tension tests in mm (in.).
Δ_S	= uncorrected displacement for shear tests in mm (in.).
A_N and B_N	= instrument readings at a given load in mm (in.).
A_I and B_I	= initial instrument readings in mm (in.).
Δ_T	= average displacement at maximum load for tension tests in mm (in.).
Δ_S	= average displacement at maximum load for shear tests in mm (in.).
n	= number of test samples.
N_T	= total number of load cycles in tension fatigue test.
N_S	= total number of load cycles in shear fatigue test.
$\bar{N}_T$	= average number of load cycles in tension fatigue test.
$\bar{N}_S$	= average number of load cycles in shear fatigue test.
Δ_{FT}	= displacement of anchor occurring at maximum load for tension fatigue test mm (in.).
Δ_{FS}	= displacement of anchor occurring at maximum load for shear fatigue test mm (in.).
A_{fu} and B_{fu}	= maximum displacement instrument readings for fatigue tests mm (in.).
A_{fi} and B_{fi}	= initial displacement instrument readings for fatigue tests mm (in.).
$\overline{\Delta}_{FT}$	= average maximum displacement for tension fatigue tests mm (in.).
$\overline{\Delta}_{FS}$	= average maximum displacement for shear fatigue tests mm (in.).

4. Significance and Use

4.1 These test methods are intended to provide data from which applicable design data and specifications are derivable for a given anchorage device used in a structural member of concrete, masonry and related products and for qualifying anchors or anchorage systems.

4.2 The test methods shall be followed to ensure reproducibility of the test data.

5. Apparatus

5.1 *Equipment*:

5.1.1 *Laboratory*—Suitable equipment shall be used to perform tests to generate data required to publish load tables or to obtain listings from approval agencies, building officials, etc. Calibrated electronic load and displacement measuring devices which meet the sampling rate of loading specified herein shall be used. The equipment shall be capable of measuring the forces to an accuracy within ± 1 % of the anticipated ultimate load, when calibrated in accordance with Practices E 4. The load and displacement measuring devices shall be capable of providing data points at least once per second in order to produce continuous load versus displacement curves. A minimum of 120 data points per instrument shall be obtained and recorded for each individual test. The readings shall be obtained prior to reaching peak load. The instruments shall be positioned to measure the vertical movement of the anchor with respect to points on the structural member in such a way that the instrument is not influenced during the test by deflection or failure of the anchor or structural member. The testing device shall be of sufficient capacity to prevent yielding of its various components and shall ensure that the applied tension loads remain parallel to the axes of the anchors and that the applied shear loads remain parallel to the surface of the structural member during testing.

5.1.2 *Field Tests*—Suitable equipment shall be used to perform tests required to verify correct installation or provide proof loads on anchors installed at a specific job site. Calibrated load cells which meet the specified rate of loading given herein shall be used. The equipment shall be capable of measuring the forces to an accuracy within ± 2 % of the applied load, when calibrated in accordance with Practices E 4. For field tests which require displacement measurements, use either manually read dial gages or electronic load and displacement measuring devices, provided they are capable of generating a minimum of 50 data points prior to reaching peak load. For field tests requiring displacement measurements, the instrument(s) shall be positioned to measure the vertical movement of the anchor with respect to points on the structural member in such a way that the instrument is not influenced during the test by deflection or failure of the anchor or structural member. The testing device shall be of sufficient capacity to prevent yielding of its various components and shall ensure that the applied tension loads remain parallel to the axes of the anchors and that the applied shear loads remain parallel to the surface of the structural member during testing.

5.2 *Tension Test*—Examples of suitable systems for applying tension pull-out forces are shown in Figs. 1 and 2 in which a single anchor specimen is shown. The test system support shall be of sufficient size to prevent failure of the surrounding structural member. The loading rod shall be of such size to develop the ultimate strength of the anchorage hardware with minimal elastic elongation and shall be attached to the anchorage system by means of a connector that will minimize the direct transfer of bending stress through the connection.

5.3 *Shear Test*—Examples of suitable systems for applying shear forces are shown in Figs. 3 and 4 in which a single anchor specimen is shown. The components of the test fixture shall be of sufficient size and strength to prevent their yielding during ultimate capacity tests on the anchorage system.

5.4 *Loading Plate*—The thickness of the loading plate in the immediate vicinity of the test anchor shall be equal to the nominal bolt diameter to be tested, ± 1.5 mm (± 1⁄16 in.), representative of a specific application.

5.4.1 The hole in the loading plate shall have a diameter 1.5 mm ± 0.75 mm (0.06 mm ± 0.03 in.) greater than the test anchor. The initial shape of the hole in the loading plate shall correspond to that of the anchor cross section and shall be maintained throughout all tests. Worn or deformed holes shall be repaired. Insert sleeves of the required diameter shall be periodically installed in the loading plate to meet these requirements.

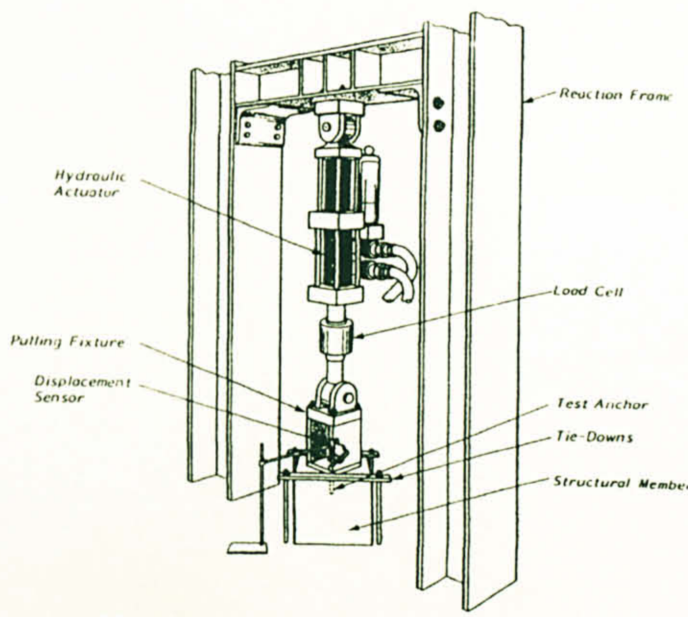


FIG. 2 Typical Seismic Tension Test Arrangement

5.4.2 For shear testing, the contact area between the loading plate through which the anchor is installed and the structural member shall be as given in Table 1, unless otherwise specified. The edges of the shear loading fixture shall be chamfered or have a radius to prevent digging in of the loading plate.

5.5 *Anchor Displacement Measurement*— For anchor tests that require displacement measurements, the displacement measurements shall be made using LVDT device(s) or equivalent which provide continuous readings with an accuracy of at least 0.025 mm (0.001 in.). Dial gages having an accuracy of

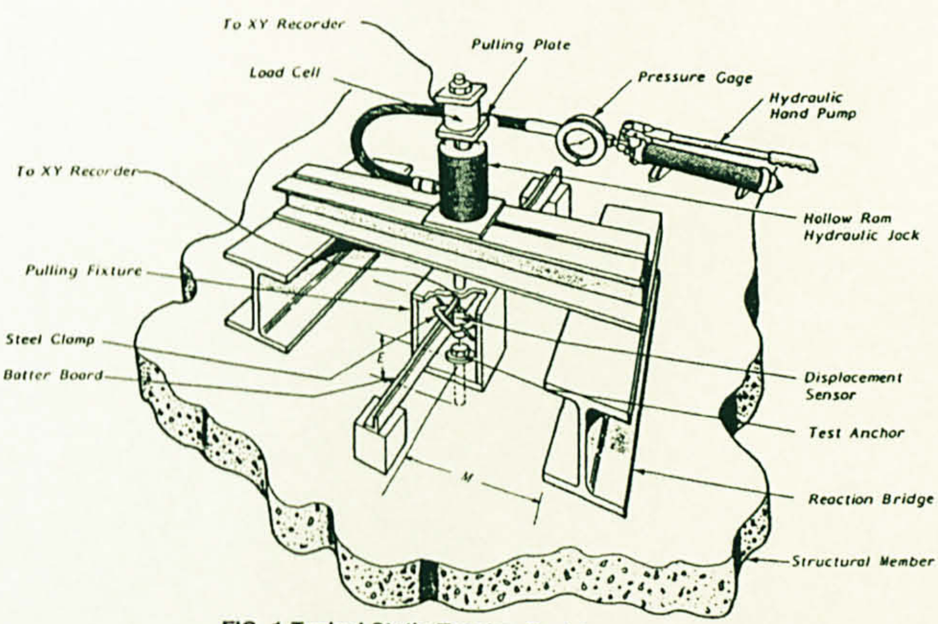


FIG. 1 Typical Static Tension Test Arrangement

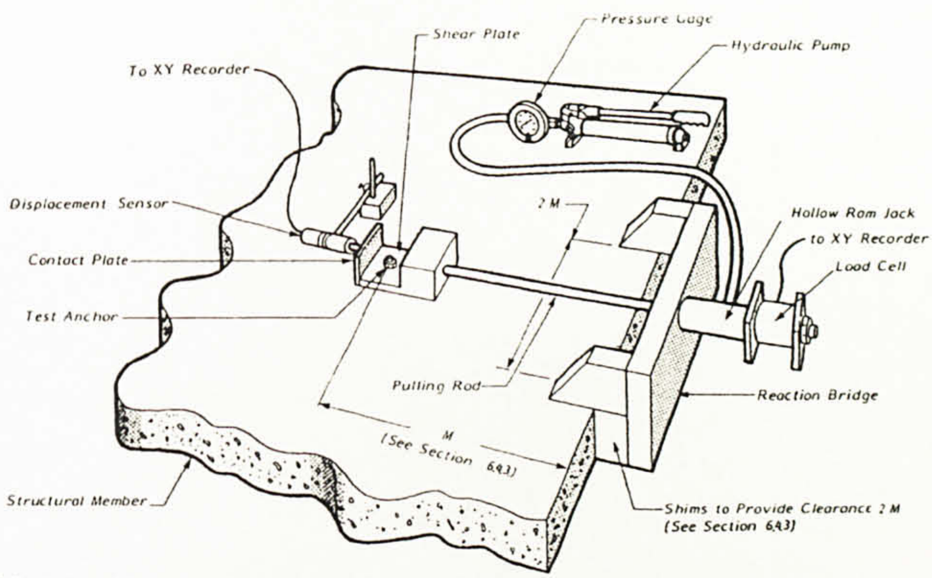


FIG. 3 Typical Method of Applying Shear Loads to Anchors Attached to Structural Members—Direct Loading Method

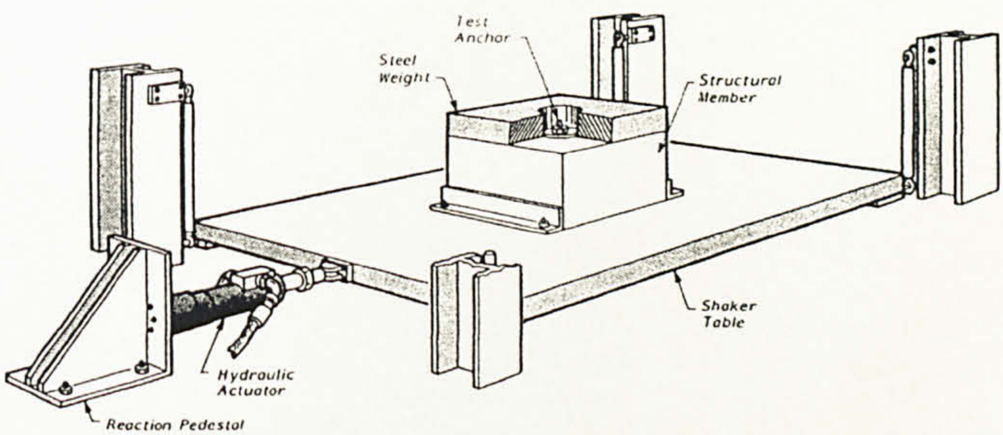


FIG. 4 Typical Seismic Shear Test Arrangement—Indirect Loading Method

TABLE 1 Shear Loading Plate Bearing Area as a Function of Anchor Diameter

Anchor Diameter, mm (in.)	Shear Loading Plate Contact Area, cm^2 ($in.^2$)
<10 (<3/8)	50–80 (8.00–12.40)
10–<16 (3/8 –<5/8)	80.01–120 (12.41–18.60)
16–<22 (5/8 –<7/8)	120.01–160 (18.61–24.80)
22–<51 (7/8 –<2)	160.01–260 (24.81–40.30)
>51 (>2)	260.1–400 (40.31–62.00)

0.025 mm (0.001 in.) are permitted in field testing or for general tests where precise displacement measurements are not required.

5.5.1 *Tension Test*:

5.5.1.1 *Single Anchor*—The displacement measuring device(s) shall be positioned to measure the vertical movement of the anchors with respect to points on the structural member in such a way that the device is not influenced during the test by deflection or failure of the anchor or structural member.

5.5.1.2 *Group of Anchors*—Displacement measurements shall be made on all anchors or group of anchors tested simultaneously except that only one set of instruments needs to be used for a group of anchors tested as a closely spaced cluster. Displacement measurements as described in 5.5 include components of deformation not directly associated with displacement of the anchor relative to the structural member. Include components of deformation such as elastic elongation of the loading rod anchor stem, deformation of the loading plate, sleeves, shims, attachment hardware, and local structural member material. Deduct all of the elongations from these sources from the total displacement measurements by using supplementary measuring devices or calibration test data for the installed test set-up with rigid specimen replacing the anchor to be tested. The displacement to be used for the evaluation of the findings is the average displacement indicated by both instruments mounted symmetrically equidistant from the centroid of the cluster as shown in Fig. 5.

5.5.2 *Shear Test*—The displacement measuring device(s) shall be positioned to measure displacement in the direction of the applied load. The device shall be placed on the structural member to allow the sensing element to bear perpendicularly on the anchor or on a contact plate located on the loading plate

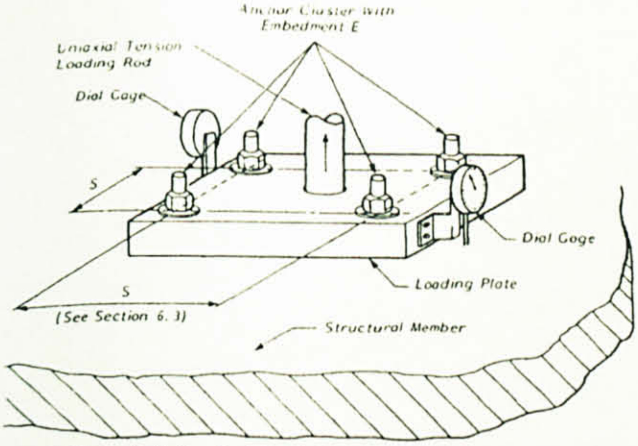


FIG. 5 Typical Method of Applying a Test Load to a Cluster of Anchors in the Test Area of a Structural Member

as shown in Fig. 3 or other method which prevents extraneous deflections. For tests on clusters of anchors, the instrument shall lie on a plane through the axis of the shear loading rod or plate. An extension of the axis of the shear loading rod or plate shall pass through the centric axis of the cluster of anchors.

6. Test Specimens

6.1 *Anchorage System*—The anchorage system shall be representative of the type and lot to be used in field construction and shall include all accessory hardware normally required for its use, that is, all attachment hardware.

6.2 *Anchor Installation*—Install the anchorage device in accordance with the manufacturer's procedures and tools, or, where specific deviation is justified, in accordance with good field methods.

6.3 *Anchor Placement*—Individually test all anchors as specified in the test program. The anchors shall be tested at distances equal to or greater than those given in Table 2. The distances in Table 2 are not intended for design of attachments. Table 2 test support requirements are not prohibited from being reduced for bonded anchors with embedments equal to or greater than 20 anchor diameters. For anchors intended to be field-installed at spacings less than specified in Table 2 in groups of two or more, test at the intended spacings or edge distances per the requirements of 8.3 at the selected spacing and edge distance intervals to assign reduction factors.

6.4 *Structural Member*—The structural member in which the anchor is to be embedded shall be representative of the materials and configuration intended for field use. The structural member is not prohibited from being steel-reinforced. The location and orientation of any reinforcement embedded in concrete or masonry members shall be evaluated. The overall size of the test specimen shall not be reduced unless the requirements in 6.4.1-6.4.3.1 are met.

6.4.1 The depth of the structural member shall be equal to the minimum member depth specified by the manufacturer. The structural member shall be at least 1.5 h_{ef} in thickness so long as the depth is suitable for normal installation of the anchor and does not result in premature failure of either the structural member or anchor, unless the specific test application requires a lesser thickness. The structural member will act as a beam if the spacing between reaction supports is greater than the thickness of the member. A structural member with a thickness of at least 1.5 h_{ef} will minimize bending during the application of the tensile load to the test anchor. In general, the thickness of the test member shall be equal to the minimum member depth specified by the manufacturer.

6.4.2 The length and width of the structural member shall ensure that no shear or tension failure spall intersects either the outside edges of the structural member or the bearing contact points of the test frame. The overall size of the test specimen shall only be reduced when the minimum requirements in 6.4.1 are met.

6.4.3 *Surface Finish*—The surface of the structural member where the loading fixture or loading plate bears on the member shall be a form-work or steel-trowel finish unless otherwise specified.

6.4.3.1 For static shear tests, a sheet of tetrafluoroethylene (TFE), polytetrafluoroethylene (PTFE), fluorinated ethylene propylene (FEP), or perfluoroalkoxy (PFA) of 0.5 ± 0.1 mm (0.020 ± 0.004 in.) thickness and corresponding to the area required according to Table 1 shall be placed between the shear plate and the surface of the structural member.

TABLE 2 Minimum Clearance Requirements for Test Equipment Supports

Adhesive Anchors		All Other Anchors	
Spacing between Test Supports	Minimum Distance to Edge or Test Frame	Spacing between Test Supports	Minimum Distance to Edge or Test Frame
Tension Loads			
$2.0h_{ef}$	$1.0h_{ef}$	$4.0h_{ef}$	$2.0h_{ef}$
Shear Loads			
$4.0h_{ef}$	$2.0h_{ef}$	$4.0h_{ef}$	$2.0h_{ef}$

7. Conditioning

7.1 *Specimen Conditioning and Curing*—When aging, seasoning, or curing conditions affect the performance and capacity of the installed anchor, take appropriate measures to age, season, or cure the installed anchoring system in accordance with appropriate procedures prior to testing. Describe such conditions in detail. Cast-in-place concrete, grout-set, and epoxy-set anchors are some examples of anchorage systems that require provisions for aging or curing.

7.2 *Specimen Moisture and Temperature*—When moisture and temperature conditions affect the performance of the anchorage system, they shall be kept constant during the testing of the anchorage system. The choice of the controlled conditions shall simulate the conditions under which the anchors will be used. Simulate field moisture and temperature conditions or use standard conditions of 23°C (73°F) and 50 % relative humidity, as provided in Specification E 171. Testing shall begin only after the test specimens have reached at least an appropriate stable condition with regard to temperature and moisture content.

8. Static Tests

8.1 *Equipment*—Use any suitable testing or loading system specified in the Apparatus section.

8.2 *Number of Test Specimens*—For determining the average tension or shear resistance, a minimum of five anchors per size shall be tested and the five test results averaged. Where steel failures occur, only a minimum of three anchors per size shall be tested and the three test results averaged.

8.3 *Number of Test Specimens for Statistical Data*—For determining statistical data, such as coefficient of variation or spacing and edge-distance reduction factors (with a ± 10 % accuracy), a minimum number of tests (n) shall be performed in accordance with Table 3.

8.3.1 This procedure shall be repeated for each variation in anchor type, size, embedment depth, and location. This procedure shall also be repeated for each variation in the structural member.

8.4 *Static Test Procedure*:

8.4.1 *Tension Test*:

8.4.1.1 Position the loading system, in such a way that the placement of the test system supports meet the requirements of Table 2 (see Figs. 1 and 2). Position the loading device in such a way that it is centered over the anchor to be tested. Provide uniform contact between the surface of the structural member and the support system. In the final alignment of the support system, ensure that the forces to be applied through the loading rod are perpendicular to the surface of the structural member section. The amount of torque or pretension applied to the anchor by the attaching nut or locking device shall be uniform for each series of tests.

8.4.1.2 Position and attach the loading rod so that the load is applied through the center of a single anchor, or through the centroid of a cluster of anchors. Whenever a loading plate is required in the testing of a cluster of anchors, ensure uniform loading of the individual anchors of the cluster.

8.4.2 *Shear Test*:

8.4.2.1 Position the loading system in such a way that the placement of the test system supports meet the requirements of Table 2 (see Fig. 3). A reaction bridge is not required along the edge of the structural member if the edge distance is larger than 4 h_{ef} in all directions.

8.4.2.2 Position and fasten the structural member in the support system in such a way that the test surface of the structural member is parallel to the loading plate and the axis of the pulling rod. Place the loading plate-rod assembly onto the structural member and secure it in place with the appropriate nut or other locking device typically used for the particular anchor installation to be tested. The amount of force exerted on the loading plate by the attaching nut or locking device shall be uniform for each series of tests performed.

8.5 *Initial Load*—Apply an initial load up to 5 % of the estimated maximum load capacity of the anchorage system to be tested, in order to bring all members into full bearing.

8.6 *Rate of Loading*—Two loading rates are given. For tests that require precise anchor load-displacement data for calculating stiffness or assessing proper functioning, the continuous load application method is required. The first method requires a continuous increase in load up to failure or up to a maximum specified load or displacement. The second is a step-loading method in 15 % increments of the expected ultimate load.

8.6.1 *Continuous Load Application*—Apply loading to the anchor at a uniform rate that will produce a failure as defined in the Failure Criteria section. A loading rate of 25 to 100 % of the ultimate anchor capacity per minute shall be used except that a minimum 1-min total test time and a maximum 3-min total test time is allowed when the test equipment provides accurate recording of load and displacement readings.

8.6.2 *Incremental Load Application*—In step loading during sustained constant-level load increments up to a maximum load, each increment load shall not exceed more than 15 % of the estimated maximum test load and shall be maintained for a 2-min period. Plot the initial and 2-min readings of the measurement devices in the form of load-displacement curves. Maintain complete load-displacement records throughout the test or plot after completion of the test. The data records shall include a time record of the beginning and end of each increment of constant load.

8.6.3 *Load Application for a Given Period*—If application of a given load is required for a certain period, such as 24 h, deformation readings shall be taken at the beginning, during, and end of the period to allow the satisfactory plotting of a time-displacement curve for the complete period.

8.7 *Calculations*:

8.7.1 *Load-Displacement Data*:

8.7.1.1 Determine the uncorrected displacements Δ_T and Δ_S at any given load for an individual test in the following manner:

For tension tests:

$$\Delta_T = ½ (A_N - A_I + B_N - B_I) \quad (1)$$

For shear tests:

$$\Delta_\Sigma = A_N - A_I \quad (2)$$

where: A_N and B_N are instrument readings at the given load, and A_I and B_I are initial instrument readings.

8.7.1.2 Obtain the corrected displacement by plotting the uncorrected displacement versus the applied loads and extrapolating a smooth curve through the data points back to zero load. The corrected displacement at maximum or at any other test load is observed from the plot relative to the adjusted zero-load displacement value.

8.7.1.3 Obtain the average displacement at maximum (Δ_T or Δ_S) or any other load for each test series as the arithmetic mean of all individual displacement determinations at a given load in a given series.

9. Seismic Tests

9.1 These tests demonstrate the capability of an anchor to withstand a simulated seismic event.

9.2 *Equipment*—Any suitable testing or loading system shall be used as provided for in the Apparatus section.

9.3 *Number of Test Specimens*—For determining the average tension or shear capability of the anchorage system, perform at least five tests per anchor size and type unless otherwise specified.

TABLE 3 Size for Statistical Evaluation of Test Data

Coefficient of Variation, %	Minimum Test Sample Size Required, *n*
Up to 12	5
12 to 15	10
>15	30

9.4 *Seismic Test Procedure*:

9.4.1 *Tension Test*—Position the loading system as described in 8.4.1.

9.4.2 *Rate and Level of Loading*—Apply test loads and cycles in accordance with a specified program to simulate seismic requirements.

9.4.3 *Shear Test*:

9.4.3.1 *Shear Test Direct-Loading Procedure*—Position the loading system in accordance with 8.4.2 as shown in Fig. 3.

9.4.3.2 *Shear Test Indirect Loading Procedure*—A structural member with a predrilled hole or an installed anchor is secured to the shaker table with bearing angles and tie-down bolts. The steel weight is then set over the hole or the installed anchor. This weight is secured to the structural member by the anchor (see Fig. 4).

9.4.4 *Rate of Loading*:

9.4.4.1 *Direct Loading and Rate Procedure*—Apply test loads and cycles in accordance with a specified program to simulate seismic requirements.

9.4.4.2 *Indirect Loading Rate Procedure*—Use the seismic shear test specified program given by the specifying authority. Inspect the anchor for failure or any suspected damage after each set of cycles.

9.4.4.3 *Instrumentation for Indirect Shear Test*—Accelerometers attached to the shaker table, structural member, and steel weight are used to monitor the input and output acceleration-level forces.

9.4.4.4 Once the cyclic test has been completed, the anchor shall be subjected to a shear test to determine its residual strength in accordance with the Apparatus and the Failure Criteria section, if required.

10. Fatigue Tests

10.1 *Equipment*—Any testing machine as described in the Apparatus section shall be used provided the requirements of specific loading rate and accuracy are met. The configuration of the test systems shall be such that no resonant vibrations are produced during the tests.

10.2 *Number of Test Specimens*—The number of test specimens shall be based on the purpose of the test. If the objective is to obtain runout at or below the endurance limit (that is, 2×10^6 cycles) at a given load, then three samples that reach runout are sufficient. If the test objective is to determine the maximum load that will reach runout (the endurance limit), then tests in accordance with Practice E 468 shall be performed.

10.3 *Fatigue Test Procedure*—Apply the specified fatigue test program, including the method, load levels, frequency, and number of cycles.

10.4 Once the cyclic test has been completed, apply a static tension load in accordance with the section on Static Tests to determine its residual strength and failure mode in accordance with the section on Failure Criteria.

11. Shock Test

11.1 *Equipment*—This test method is not intended to prohibit the use of any testing or loading device which provides the performance described in the Apparatus section.

11.2 *Number of Test Specimens*—The purpose and type of the shock test will determine the number of test specimens.

11.2.1 If the purpose is to determine if an anchorage system will withstand a specified shock load (magnitude and duration), at least three anchors shall be tested per anchor size at a given load and duration.

11.2.2 If the purpose is to determine the maximum shock loading an anchorage system is capable of withstanding without failure, a suitable test method such as a staircase method shall be used to obtain an anchorage failure. Three separate anchor tests at a given load without failure shall be sufficient to establish the maximum shock capacity of the anchorage system.

11.3 *Shock Test Procedure*:

11.3.1 *Tension Test*—Position the loading system as described in 8.4.1.

11.3.2 *Shear Test*—Position the loading system as described in 8.4.2.

11.4 *Rate of Loading Tension or Shear*—Apply a specified number of shocks to each anchor in a triangular (ramp) loading rate with a total application of 30 ms per shock, or as otherwise specified. After application of the shock loads, the anchors shall be tensile tested in accordance with the Static Tests section to measure residual static tensile capacity, if required.

12. Failure Criteria

12.1 *Load and Displacement at Failure*—Determine the maximum test load and the corresponding displacement for each assembly tested.

12.2 *Failure Modes*—Failure occurs by one or more of the following modes:

12.2.1 Failure of the structural member in a shear-cone mode.

12.2.2 Failure of the structural member with or without cracking that radiates outward from the location of the anchorage device, resulting in a pullout of the anchor.

12.2.3 Pullout of the anchor.

12.2.4 Failure of the bond between the anchor and the structural member. Displacement failure is evidenced by continuous displacement associated with a constant or decreasing applied load.

12.2.5 The fracture of any component of the anchoring device including hardware accessories shall constitute failure. Some anchorage systems require deformation to become effective. This provision does not apply to that deformation.

13. Report

13.1 Report the applicable information listed in Practice E 575, all information pertinent to the type of test performed (static, seismic, fatigue or shock, cracked or uncracked concrete), and specifically include the following:

13.1.1 Dates of test and date of report;

13.1.2 Test sponsor and test agency;

13.1.3 Identification of anchors tested: manufacturer, model type, material, finish, shape, dimensions, and other pertinent information, such as cracks and other defects;

13.1.4 Description of the anchorage system tested and physical description of the structural member, including dimensions, installed reinforcing, etc.;

13.1.5 Detailed drawings or photographs of test specimens before and after testing if not fully described otherwise;

13.1.6 Physical strength properties of the structural member into which the anchor(s) are embedded including mix design of the concrete, aggregate type, 28-day compressive strength, compressive strength at time of test, and age of the structural member at time of test;

13.1.7 Description of the procedure, tools and materials used to install the anchorage system, and any deviation from those specified;

13.1.8 Age, in hours or days of anchorage system, since installation, where applicable;

13.1.9 Moisture condition, at time of test of structural member in percent of oven-dry weight where applicable. The moisture content of the structural member at time of test is determined by several methods, including drying of small samples to constant weight or use of moisture meters;

13.1.10 Temperature conditions at time of installation and at time of testing and any other temperature experience which affects anchor performance;

13.1.11 Embedment depth of the installed anchors in mm (in.);

13.1.12 Amount of torque applied to the anchor prior to testing;

13.1.13 Description of test method and loading procedure used and actual rate of loading;

13.1.14 Number of replicate specimens tested;

13.1.15 Individual and average maximum load values, in kN (lbf), per embedded anchor, standard deviations and coefficients of variation, where applicable;

13.1.16 Individual and average displacement values at ultimate loads (Δ_T, Δ_S, or both), in mm (in.) and standard deviations, or where appropriate load versus displacement curves, as plotted directly, or as reprinted from data acquisition systems;

13.1.17 Description of the nature and type of failure exhibited by each anchor tested, including where appropriate, individual and average fatigue life values ($\bar{N}_T$, $\bar{N}_S$) in numbers of fatigue load cycles or the runout number of fatigue load cycles;

13.1.18 Photographs, sketches, or word descriptions of the failure modes observed;

13.1.19 Summary of findings; and

13.1.20 Listing of observers of tests and signatures of responsible persons.

14. Precision and Bias

14.1 No statement is made on the precision or bias of these test methods, since the test results indicate only whether there is conformance to given criteria and since no generally accepted method for determining precision and bias of these test methods is currently available. The information provided herein on the specimens, instrumentation, and procedures makes the results intractable to calculation of meaningful values by statistical analysis for precision and bias at this time.

15. Keywords

15.1 anchors; cast-in-place anchors; chemical anchors; concrete elements; expansion anchors; fatigue; masonry elements; post-installed anchors; seismic; shock; static; tensile/shear strengths; test methods

APPENDIX E –

ASTM C 900 -01

STANDARD TEST METHOD FOR PULL-OUT STRENGTH OF HARDENED CONCRETE [9]

Designation: C 900 – 01

INTERNATIONAL

Standard Test Method for Pullout Strength of Hardened Concrete[1]

This standard is issued under the fixed designation C 900; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope *

1.1 This test method covers determination of the pullout strength of hardened concrete by measuring the force required to pull an embedded metal insert and the attached concrete fragment from a concrete test specimen or structure. The insert is either cast into the fresh concrete or installed in hardened concrete.

1.2 The values stated in SI units are to be regarded as the standard. The values given in parentheses are for information purposes only.

1.3 The text of this test method references notes and footnotes which provide explanatory material. These notes and footnotes (excluding those in tables and figures) shall not be considered as requirements of this test method.

1.4 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 39/C 39M Test Method for Compressive Strength of Cylindrical Concrete Specimens[2]

C 670 Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials[2]

E 4 Practices for Force Verification of Testing Machines[3]

E 74 Practice of Calibration of Force Measuring Instruments for Verifying the Load Indication of Testing Machines[3]

3. Summary of Test Method

3.1 A metal insert is either case into fresh concrete or installed into hardened concrete. When an estimate of the in-place strength is desired the insert is pulled by means of a jack reacting against a bearing ring. The pullout strength is determined by measuring the maximum force required to pull the insert from the concrete mass.

4. Significance and Use

4.1 For a given concrete and a given test apparatus, pullout strengths can be related to compressive strength test results. Such strength relationships depend on the configuration of the embedded insert, bearing ring dimensions, depth of embedment, and level of strength development in that concrete. Prior to use, these relationships must be established for each system and each new combination of concreting materials. Such relationships tend to be less variable where both pullout test specimens and compressive strength test specimens are of similar size, compacted to similar density, and cured under similar conditions.

NOTE 1—Published reports (**1-16**)[4] by different researchers present their experiences in the use of pullout test equipment. Refer to ACI 228.1R (**14**) for guidance on establishing a strength relationship and interpreting test results. The Appendix provides a means for comparing pull-out strengths obtained using different configurations.

4.2 Pullout tests are used to determine whether the in-place strength of concrete has reached a specified level so that, for example:

(1) post-tensioning may proceed;

(2) forms and shores may be removed; or

(3) winter protection and curing may be terminated.

In addition, post-installed pullout tests may be used to estimate the strength of concrete in existing constructions.

4.3 When planning pullout tests and analyzing test results, consideration should be given to the normally expected decrease of concrete strength with increasing height within a given concrete placement in a structural element. The measured pullout strength is indicative of the strength of concrete within the region represented by the conic frustum defined by the insert head and bearing ring. For typical surface installations, pullout strengths are indicative of the quality of the outer zone of concrete members and can be of benefit in evaluating the cover zone of reinforced concrete members.

---

[1] This test method is under the jurisdiction of ASTM Committee C09 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.64 on Nondestructive and In-Place Testing .

Current edition approved Aug. 10, 2001. Published October 2001. Originally published as C 900 – 78 T. Last previous edition C 900 – 99.

[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] *Annual Book of ASTM Standards*, Vol 03.01.

[4] The boldface numbers refer to the list of references at the end of this test method.

*A Summary of Changes section appears at the end of this standard.

4.4 Cast-in-place inserts require that their locations in the structure be planned in advance of concrete placement. Post-installed inserts can be placed at any desired location in the structure provided the requirements of 6.1 are satisfied.

4.5 This test method is not applicable to other types of post-installed tests that, if tested to failure, do not involve the same failure mechanism and do not produce the same conic frustum as the cast-in-place test. **(16)**.

5. Apparatus

5.1 The apparatus requires three basic sub-systems: a pull-out insert, a loading system, and a load-measuring system (Note 2). For post-installed inserts, additional equipment includes a core drill, a grinding wheel to prepare a flat bearing surface, a milling tool to undercut a groove to engage the insert, and an expansion tool to expand the insert into the groove.

NOTE 2—A center-pull hydraulic jack with a suitable pressure gage and bearing ring have been used satisfactorily.

5.1.1 Cast-in-place inserts shall be made of metal that does not react with cement. The insert shall consist of a cylindrical head and a shaft to fix embedment depth that is attached firmly to the center of the head (see Fig. 1). The insert shaft shall be threaded to the insert head so that it can removed and replaced by a stronger shaft to pullout the insert, or it shall be an integral part of the insert and also function as the pullout shaft. Metal components of cast-in-place inserts and attachment hardware shall be of similar material to prevent galvanic corrosion. Post-installed inserts shall be designed so that they will fit into the drilled holes, and can be expanded subsequently to fit into the grooves that are undercut at a predetermined depth (see Fig. 2).

NOTE 3—A successful post-installed system uses a split ring that is coiled to fit into the core hole and then expanded into the groove.

5.1.2 The loading system shall consist of a bearing ring to be placed against the hardened concrete surface (see Figs. 1 and 2), and a loading apparatus with the necessary load-measuring devices that can be readily attached to the pullout shaft.

5.1.3 The test apparatus shall include centering features to ensure that the bearing ring is concentric with the insert, and that the applied load is axial to the pullout shaft, perpendicular to the bearing ring, and uniform on the bearing ring.

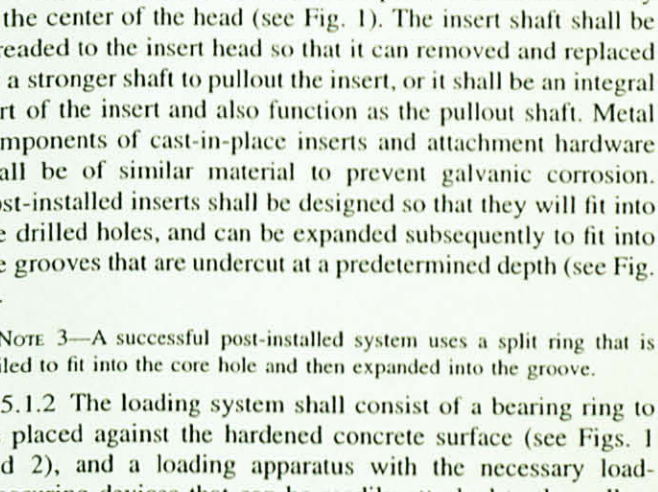


FIG. 1 Schematic Cross Section of Cast-in-Place Pullout Test

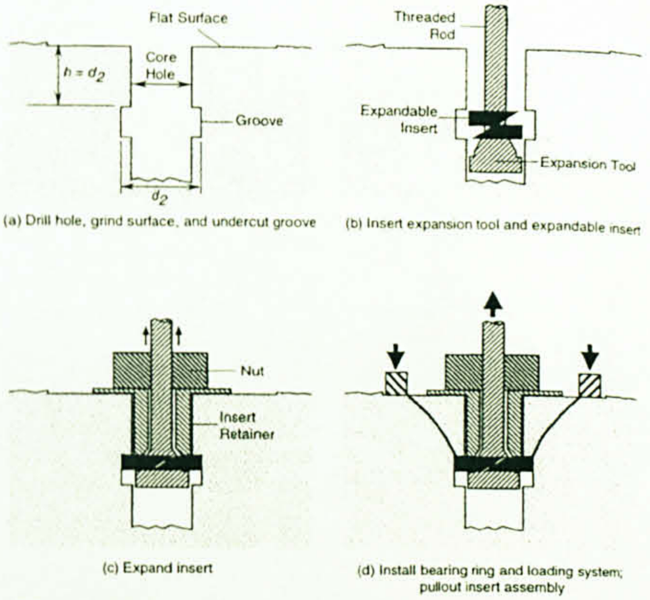


FIG. 2 Schematic of Procedure for Post-Installed Pullout Test

5.2 Equipment dimensions shall be determined as follows (see Figs. 1 and 2):

5.2.1 The diameter of the insert head (d_2) is the basis for defining the test geometry. The thickness of the insert head and the yield strength of the metal shall be sufficient to avoid yielding of the insert during test. The sides of the insert head shall be smooth (see Note 5). The insert head diameter shall be greater than or equal to ⅔ of the nominal maximum size of aggregate.

NOTE 4—Typical insert diameters are 25 and 30 mm (1 and 1.2 in.), but larger diameters have been used **(1, 3)**. Tests **(15)** have shown that nominal maximum aggregate sizes up to 1.5 times the head diameter do not have significant effects on the strength relationships. Larger aggregate sizes may result in increased scatter of the test results because the particles can restrict normal pullout of the conic frustum.

NOTE 5—Cast-in–place inserts may be coated with a release agent to minimize bonding with the concrete, and they may be tapered to minimize side friction during testing. The insert head should be provided with the means, such as a notch, to prevent rotation in the concrete if the insert shaft has to be removed prior to performing the test. As a further precaution against rotation of the insert head, all threaded hardware should be checked prior to installation to ensure that it is free-turning and can be easily removed. A thread-lock compound is recommended to prevent loosening of the insert head from the shaft during installation and during vibration of the surrounding concrete.

5.2.2 For cast-in–place inserts, the length of the pullout insert shaft shall be such that the distance from the insert head to the concrete surface (h) equals the diameter of the insert head (d_2). The diameter of the insert shaft at the head (d_1) shall be no more than 0.60 times the head diameter.

5.2.3 For post-installed inserts, the groove to accept the expandable insert shall be cut so that the distance between the groove and concrete surface equals the insert diameter after expansion (d_2). The difference between the diameters of the undercut groove and the core hole (d_1) shall be sufficient to prevent localized failure and ensure that a conic frustum is extracted during the test (see Note 6). The expanded ring shall bear uniformly on the entire bearing area of the groove.

NOTE 6—A core hole diameter of 18 mm (0.71 in.) and an undercut groove diameter of 25 mm (1 in.) have been used successfully.

5.2.4 The bearing ring shall have an inside diameter (d_3) of 2.0 to 2.4 times the insert head diameter, and shall have an outside diameter (d_4) of at least 1.25 times the inside diameter. The thickness of the ring (t) shall be at least 0.4 times the pullout insert head diameter.

5.2.5 Tolerances for dimensions of the pullout test inserts, bearing ring and embedment depth shall be ±2 % within a given system.

NOTE 7—The limits for dimensions and configurations for pullout test inserts and apparatus are intended to accommodate various systems.

5.2.6 The loading apparatus shall have sufficient capacity to provide the loading rate prescribed in 7.4 and exceed the maximum load expected.

NOTE 8—Hydraulic pumps that provide a constant loading rate may give more uniform test results than pumps that apply the load intermittently.

5.2.7 The gage to measure the pullout force shall have a least division not larger than 5 % of the minimum value in the intended range of use.

NOTE 9—For the most accurate results, gages should have a maximum value indicator that preserves the value of the ultimate load when ultimate failure and subsequent stress release occur.

5.2.8 Pullout apparatus shall be calibrated in accordance with Annex A1 at least once a year and after all repairs. Calibrate the pullout apparatus using a testing machine verified in accordance with Practices E 4 or a Class A load cell as defined in Practice E 74. The indicated pullout force based on the calibration relationship shall be within ±2 % of the force measured by the testing machine or load cell.

6. Sampling

6.1 Pullout test locations shall be separated so that the clear spacing between inserts is at least eight times the pullout insert head diameter. Clear spacing between the inserts and the edges of the concrete shall be at least four times the head diameter. Inserts shall be placed so that reinforcement is outside the expected conical failure surface by more than one bar diameter, or the maximum size of aggregate, whichever is greater.

NOTE 10—A reinforcement locator is recommended to assist in avoiding reinforcement when preparing post-installed tests. Follow the manufacturer's instructions for proper operation of such devices.

6.2 When pullout test results are used to assess the in-place strength in order to allow the start of critical construction operations, such as formwork removal or application of post tensioning, at least five individual pullout tests shall be performed as follows:

6.2.1 For a given placement, every 115 m^3 (150 yd^3), or a fraction thereof, or

6.2.2 For slabs or walls, every 470 m^2 (500 ft^2), or a fraction thereof, of the surface area of one face.

Inserts shall be located in those portions of the structure that are critical in terms of exposure conditions and structural requirements.

6.3 When pullout tests are used for other purposes, the number of tests shall be determined by the specifier.

7. Procedure

7.1 *Cast-in-Place Inserts*:

7.1.1 Attach the pullout inserts to the forms using bolts or by other acceptable methods that firmly secure the insert in its proper location prior to concrete placement. All inserts for the same tests shall be embedded to the same depth and each shaft shall be perpendicular to the formed surface.

NOTE 11—Inserts may be manually placed into unformed horizontal concrete surfaces. The inserts should be embedded into the fresh concrete by means that ensure a uniform embedment depth and a plane surface perpendicular to the axis of the insert shaft. Installation of inserts should be performed or supervised by experienced personnel. Experience indicates that pullout strengths are of lower value and more variable for manually-placed surface inserts than for inserts attached to the formwork.

7.1.2 When the concrete is to be tested, remove all hardware used for securing the pullout inserts in position. Before mounting the loading system, remove any debris or surface abnormalities to ensure a smooth bearing surface that is perpendicular to the axis of the insert.

7.2 *Post-Installed Inserts*:

7.2.1 The selected test surface shall be flat to provide a suitable working surface for drilling the core and undercutting the groove. Drill a core hole perpendicular to the surface to provide a reference point for subsequent operations and to accommodate the expandable insert and associated hardware. The use of an impact drill is not permitted.

7.2.2 If necessary, use a grinding wheel to prepare a flat surface so that the base of the milling tool is supported firmly during test preparation and so that the bearing ring is supported uniformly during testing. The ground surface shall be perpendicular to the axis of the core hole.

7.2.3 Use the milling tool to undercut a groove of the correct diameter at the correct depth in the core hole. The groove shall be concentric with the core hole.

NOTE 12—To control the accuracy of these operations, a support system should be used to hold the apparatus in the proper position during these steps.

7.2.4 If water is used as a coolant, remove free-standing water from the hole at the completion of the drilling and undercutting operations. Protect the hole from ingress of additional water until the completion of the test.

NOTE 13—Penetration of water into the failure zone could affect the measured pullout strength; therefore, water must be removed from the hole immediately after completion of drilling, grinding, and undercutting operations. If the test will not be completed immediately after preparation of the hole, water must not be allowed to enter the hole before completing the test.

7.2.5 Use the expansion tool to position the expandable insert into the groove and expand the insert to its proper size.

7.3 *Bearing Ring*—Place the bearing ring around the pullout insert shaft, connect the pullout shaft to the hydraulic ram, and tighten the pullout assembly snugly against the bearing surface.

checking to see that the bearing ring is centered around the shaft and flush against the concrete.

7.4 *Loading Rate*—Apply load at a uniform rate so that the nominal normal stress on the assumed conical fracture surface increases at a rate of 70 ± 30 kPa/s (Note 14). If the insert is to be tested to rupture of the concrete, load at the specified uniform rate until rupture occurs. Record the maximum gage reading to the nearest half of the least division on the dial. If the insert is to be tested only to a specified level for acceptance, load at the specified uniform rate until the specified pullout load is reached.

NOTE 14—The loading rate is specified in terms of a nominal stress rate to accommodate different sizes of pullout test systems. See Appendix X1 for the formula relating the nominal normal stress and the pullout load. For a pullout test system in which d_2 = 25 mm and d_3 = 55 mm, the specified stress rate corresponds to a loading rate of approximately 0.5 ± 0.2 kN/s. If this system is used, the ranges of the times to complete a test for different anticipated ultimate pullout loads would be as follows:

Anticipated Pullout Load, kN	Minimum Time, s	Maximum Time, s
10	14	33
20	29	67
30	43	100
40	57	133
50	71	167
60	86	200
70	100	233
80	114	267
90	129	300
100	143	333

7.5 *Rejection*—Reject a test result if one or more of the following conditions are encountered:

7.5.1 The large end of the conic frustum is not a complete circle of the same diameter as the inside diameter of the bearing ring;

7.5.2 The distance from the surface to the insert head (*h* in Fig. 1 or Fig. 2) is not equal to the insert diameter;

7.5.3 The diameter of the groove in a post-installed test is not equal to the design value;

7.5.4 The expanded insert diameter in a post-installed test is not equal to the design value; or,

7.5.5 A reinforcing bar is visible within the failure zone after the conic frustum is removed.

8. Calculation

8.1 Convert gage readings to pullout force on the basis of calibration data.

8.2 Compute the average and standard deviation of the pullout forces that represent tests of a given concrete placement.

9. Report

9.1 Report the following information:

9.1.1 Dimension of the pullout insert and bearing ring (sketch or define dimensions),

9.1.2 Identification by which the specific location of the pullout test can later be determined,

9.1.3 Date and time when the pullout test was performed.

9.1.4 For tests to failure, maximum pullout load of individual tests, average, and standard deviation, kN (lbf). For tests to a specified load, the pullout load applied in each test, kN (lbf).

9.1.5 Description of any surface abnormalities beneath the reaction ring at the test location,

9.1.6 Abnormalities in the ruptured specimen and in the loading cycle,

9.1.7 Concrete curing methods used and moisture condition of the concrete at time of test, and

9.1.8 Other information regarding unusual job conditions that may affect the pullout strength.

10. Precision and Bias

10.1 *Precision*—Based on the data summarized in ACI 228.1 R (**14**) for cast-in-place pullout tests with embedment of about 25 mm (1 in.), the average coefficient of variation for tests made on concrete with maximum aggregate of 19 mm (¾ in.) by a single operator using the same test device is 8 %[5]. Therefore, the range in individual test results, expressed as a percentage of the average, should not exceed the following:

Number of Tests	Acceptable range, (percent of average)
5	31 %
7	34 %
10	36 %

Similar values of within-test variability have been reported for post-installed pullout tests of the same geometry as cast-in-place tests (**15**).

NOTE 15—If the range of tests results exceeds the acceptable range, further investigation should be carried out. Abnormal test results could be due to improper procedures or equipment malfunction. The user should investigate potential causes of outliers and disregard those test results for which reasons for the outlying results can be identified positively. If there are no obvious causes of the extreme values, it is probable that there are real differences in concrete strength at different test locations. These differences could be due to variations in mixture proportions, degree of consolidation, or curing conditions.

10.2 *Bias*—The bias of this test method cannot be evaluated since pullout strength can only be determined in terms of this test method.

11. Keywords

11.1 concrete strength; in-place strength; in-place testing; pullout test

[5] This number represents the (1s%) limit as described in Practice C 670.

ANNEX

(Mandatory Information)

A1. CALIBRATION OF PULLOUT-HYDRAULIC LOADING SYSTEM

A1.1 The objective of the calibration procedure is to establish a relationship between the reading of the pullout force measuring system and the tensile force in the shaft used to pullout the insert. This relationship is established using alternative approaches as indicated in Fig. A1.1. In general, calibration is achieved by correlating the gage reading of the pullout loading system with the force measured by a testing machine that has been verified in accordance with Practices E 4 or a Class A load cell that has been calibrated in accordance with Practice E 74. The time interval between testing machine verifications or load cell calibrations shall be as defined in Practices E 4 or E 74.

A1.2 Position the pullout loading system on the force measurement apparatus. Align all components so that the pullout force is concentric with the loading system and the force measurement system. Use spherical seats or other similar means to minimize bending effects in the loading system.

NOTE A1.1—When a compression-testing machine is used to measure the force, the bearing blocks should be protected against damage. Cold-rolled steel plate at least 13 mm (½ in.) thick is recommended.

A1.3 Using the pullout loading system, apply increasing loads over the operating range, and record the gage reading and the corresponding force measured by the testing machine or load cell. Take readings at approximately 10 load levels distributed over the operating range of the pullout loading system.

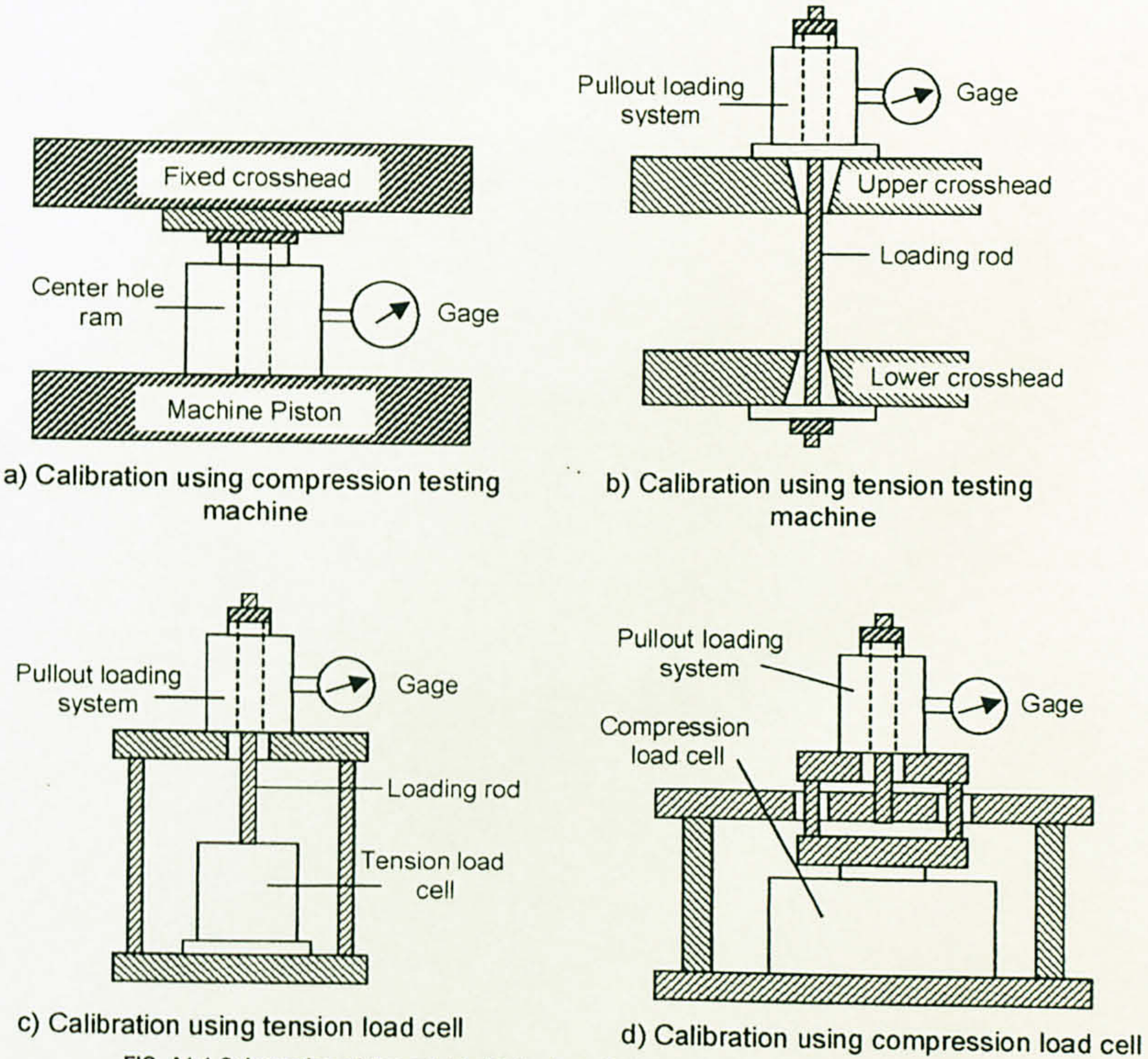


FIG. A1.1 Schematics of Acceptable Methods to Calibrate Pullout Load Measuring System

NOTE A1.2—Low values of force should be avoided in the calibration process because the effects of friction may introduce significant errors. The manufacturer should provide the operating range of the pullout loading system.

A1.4 Using readings obtained during calibration loading, calculate an appropriate regression equation using the least-squares curve-fitting method.

NOTE A1.3—Appendix X2 provides an example to illustrate the development of a calibration equation. Additional information is provided in Practice E 74.

A1.5 The difference between the force based on the regression equation and the force measured by the testing machine or the load cell shall not be greater than ±2 % of the measured force over the operating range. If this tolerance is not met, the pullout loading system shall not be used until this requirement is satisfied.

APPENDIXES

(Nonmandatory Information)

X1. STRESS CALCULATION

X1.1 When a stress calculation is desired, compute a nominal normal stress on the assumed conical fracture surface by dividing the pullout force by the area of the frustum and multiplying by the sine of one-half the apex angle (see Figs. 1 and 2). Use the following equations:

$$f_n = (P/A) \sin \alpha \tag{X1.1}$$

$$\sin \alpha = (d_3 - d_2)/2S \tag{X1.2}$$

$$A = \pi S (d_3 + d_2)/2 \tag{X1.3}$$

$$S = \sqrt{h^2 + ((d_3 - d_2)/2)^2} \tag{X1.4}$$

where:

f_n = nominal normal stress, MPa (psi),
P = pullout force, N (lbf),
α = ½ the frustum apex angle, or $\tan^{-1} (d_3 - d_2)/2h$,
A = fracture surface area, mm² (in.²),
d_2 = diameter of pullout insert head, mm (in.),
d_3 = inside diameter of bearing ring or large base diameter of assumed conic frustum, mm (in.),
h = height of conic frustum, from insert head to large-base surface, mm (in.), and
S = slant height of the frustum, mm (in.).

X1.2 The above calculation gives the value of the average normal stress on the assumed failure surface shown in Fig. 1. Because the state of stress on the conic frustum is not uniform, the calculated normal stress is a fictitious value. The calculated normal stress is useful when comparing pullout strengths obtained with different test geometries that fall within the limits of this test method.

X2. EXAMPLE TO ILLUSTRATE CALIBRATION PROCESS

X2.1 This appendix provides an example to illustrate the development of the calibration equation to convert the gage reading on the pullout loading system to the force acting on the insert. Table X2.1 shows data that were obtained using the procedure in the annex. The first column shows the gage reading and the second column is the measured force.

TABLE X2.1 Example of Calibration Data and Residuals After Regression

Gage Reading, kN	Measured Force, kN	Residuals, kN
2.0	1.6	0.03
5.0	4.8	0.09
10.0	10.5	−0.16
15.0	15.8	−0.02
20.0	21.2	0.03
25.0	26.7	−0.03
30.0	32.0	0.12
35.0	37.4	0.16
40.0	42.8	0.21
45.0	48.6	−0.14
50.0	54.2	−0.30
55.0	59.4	−0.06
60.0	64.5	0.29

X2.2 Fig. X2.1 shows a plot of the data in Table X2.1 along with the best-fit straight line to the data. A straight line was fitted using a commercial computer program for graphing and statistical analysis. The equation of the line is shown in the table of results on the graph and is as follows:

$$P\ (\text{kN}) = -0.55 + 1.089\ G\ (\text{kN}) \tag{X2.1}$$

where:

P = estimated pullout force, kN, and
G = pullout force indicated by gage of pullout loading system, kN.

The column labeled "error" in the table shown within Fig. X2.1 represents the standard deviation of the estimated intercept and slope. The low values of these standard deviations indicate that the intercept is not zero and that the slope is not equal to 1.00.

X2.3 Fig. X2.2 is a plot of the residuals of the best-fit line as a function of the measured force. These residuals are shown

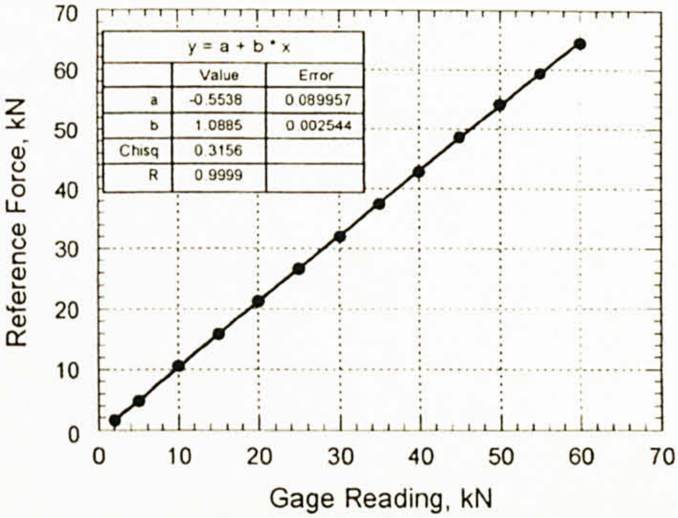


FIG. X2.1 Plot of Calibration Data from Table X2.1 and Best-Fit Straight Line

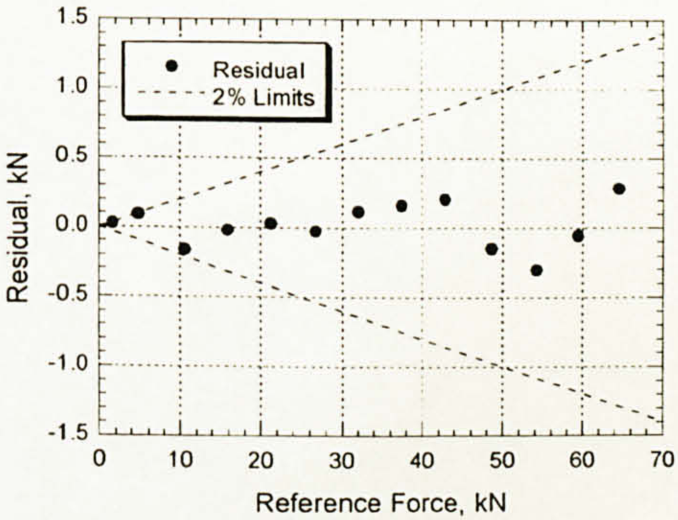


FIG. X2.2 Residuals of Best-Fit Equation as a Function of Measured Force

in the third column of Table X2.1 and are the differences between the estimated force based on the best-fit equation and the measured force (Column 2 in Table X2.1). Also shown in Fig. X2.2 are the ±2 % limits required in accordance with 5.2.8. It is seen that with the exception of the first three points, the residuals are well within the permitted tolerance. Thus, the calibration relationship for this particular apparatus satisfies the requirements of 5.2.8 provided that the pullout force is greater than about 10 kN.

X2.4 Fig. X2.2 shows that the residuals are not randomly distributed but appear to have a periodic variation with the level of force. This indicates that the true calibration equation is not a straight line. However, because the residuals are well below the ±2 % limits, it is not necessary to try to fit a higher order (polynomial) equation, and the straight line is adequate. Additional discussion on fitting higher order equations is provided in Practice E 74.

REFERENCES

(1) Richards, O.," Pullout Strength of Concrete," *Reproducibility and Accuracy of Mechanical Tests, ASTM STP 626*, ASTM 1977, pp. 32–40.

(2) Kiekegaard-Hansen, P., *Lok-Strength*, Saertryk af Nordisk Betong 3: 1975.

(3) Malhotra, V. M. and Carrette, G., "Comparison of Pullout Strength of Concrete with Compressive Strength of Cylinders and Cores, Pulse Velocity and Rebound Number," *Journal*, American Concrete Institute, Vol. 77, No. 3, May–June 1980, pp. 161–170.

(4) Bickley, J. A., "The Variability of Pullout Tests and In-Place Concrete Strength," *Concrete International*, American Concrete Institute, Vol 4, No. 4, April 1982, pp. 44–51.

(5) Dilly, R. L. and Ledbetter, W. B., "Concrete Strength Based on Maturity and Pullout," *ASCE Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 110, No. 2, Feb. 1984, pp. 354–369.

(6) Stone, W. C. and Giza, B. J., "The Effect of Geometry and Aggregate on the Reliability of the Pullout Test," *Concrete International*, American Concrete Institute, Vol 7, No. 2, Feb. 1985, pp. 27–36.

(7) Hindo, K. R. and Bergstrom, W. R., "Statistical Evaluation of the In-Place Compressive Strength of Concrete," *Concrete International*, American Concrete Institute Vol. 7, No. 2, Feb. 1985, pp. 44–48.

(8) Yener, M. and Chen, W. F., "On In-Place Strength of Concrete and Pullout Tests," Cement, Concrete, and Aggregates, CCAGDP, Vol. 6, No. 2, Winter 1984, pp. 90–99.

(9) Bickley, J. A., "The Evaluation and Acceptance of Concrete Quality by In-Place Testing," *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, 1984, pp. 95–109.

(10) Carrette, G. G. and Malhotra, V. M., "In Situ Tests: Variability and Strength Prediction of Concrete at Early Ages," *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, 1984, pp. 111–141.

(11) Khoo, L. M.," Pullout Technique—An Additional Tool for In Situ Concrete Strength Determination," *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, 1984, pp. 143–159.

(12) Vogt, W. L., Beizai, V. and Dilly, R. L.,"In Situ Strength of Concrete with Inserts Embedded by 'Finger Placing'," *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, 1984, pp. 161–175.

(13) Parsons, T. J. and Naik, T. R., "Early Age Concrete Strength Determination Pullout Testing and Maturity," *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, 1984, pp. 177–199.

(14) ACI 228.1R-95,"In-Place Methods to Estimate Concrete Strength," *ACI Manual of Concrete Practice-Part 2*, 1998, American Concrete Institute, Farmington Hills, MI.

(15) Petersen, C.G., "LOK-TEST and CAPO-TEST Pullout Testing, Twenty Years Experience," *Proceedings of Conference on Nondestructive Testing in Civil Engineering*, J.H. Bungey, Ed., British Institute of Nondestructive Testing, U.K., Liverpool, 8–11 April 1997, pp. 77–96.

(16) Carino, N.J., "Pullout Test," Chapter 3 in *Handbook on Nondestructive Testing of Concrete*, V.M. Malhotra and N.J. Carino, Eds., CRC Press, Boca Raton, FL, 1991, pp. 39–82.

SUMMARY OF CHANGES

The following changes to this test method have been incorporated since the last issue:

(1) 5.2.8 was revised to indicate the minimum accuracy of the measured pullout force.

(2) 7.4 was revised and specifies the loading rate in terms of the nominal tensile stress. Note 14 was added to provide guidance on implementing the new requirement.

(3) The Annex was revised and includes more guidance on acceptable calibration methods.

(4) A new Appendix X2 was added to illustrate the treatment of calibration data.

APPENDIX F –
GANTT CHART OF PROJECT

GANTT CHART

Semester		1 (FYP I)																								2 (FYP II)																							
Month		7				8				9				10				11				12				1				2				3				4				5				6			
No.	Activities\ Week	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
1	Submission of project proposal				■																																												
2	Data collection & revision					■	■	■	■	■	■	■	■																																				
3	Submission of progress report I						■																																										
4	Analyze data for literature review					■	■	■	■	■	■	■	■																																				
5	Preparation of experimental setup								■	■																																							
6	Submission of progress report II									■																																							
7	FYP I oral presentation																	■																															
8	Preparation of concrete foam work																							■	■																								
	Actual progress																									■	■	■	■																				
9	Mixing and casting of concrete																									■	■	■	■																				
	Actual progress																													■	■	■	■																
10	Curing																													■	■	■	■																
	Actual progress																																	■	■	■	■												
11	Submission of progress report																																		■														
12	Poster presentation																																						■										
13	Submission of dissertation report																																								■								
14	Pul-out test																																					■	■										
	Actual progress																																												■	■			
15	FYP II oral presentation																																														■		
16	Submission of report (hardbound)																																															■	■