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ANALYSIS OF SHEAR STRESS ON MARINE PILES USING SHEAR FRICTION THEORY

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ABSTRACT

The current research investigation is focused on estimating the theoretical capacity of a rehabilitated steel marine pile. The old steel pile can be rehabilitated by installing new concrete encasement (jacket). The new concrete jacket can be easily connected to the old steel pile using shear friction between old pile and new concrete jacket or additional mechanical or welded connection. The under-water welding process is a very expensive task and considerable saving can be realized by eliminating this process.

A previous experimental investigation was conducted to evaluate the behavior of the rehabilitated steel pile. The maximum load and the load-slip deformation data were recorded for all of the tested specimens. The test results indicated that the marine pile can be efficiently rehabilitated by installing a concrete jacket using shear friction principles or the bolted connection to avoid the expense of welding under-water.

The theoretical study included the investigation of surface friction, shear friction mechanism and cohesion on the bond capacity. The effect of the bolted anchor on increasing the effective cross section of the rehabilitated pile is examined. After investigating the predicted values of various equations developed by various researchers, the shear friction mechanical model developed by CSA 1994 is recommended to be used as the most effective formula that can provide an accurate prediction for the rehabilitated pile capacity.

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TABLE OF CONTENTS

AUTHOR'S DECLARATION.....	ii
ABSTRACT.....	iii
ACKNOWLEDGEMENTS.....	iv
TABLE OF CONTENTS.....	v
LIST OF FIGURES.....	ix
LIST OF TABLES.....	xii
LIST OF SYMBOLS.....	xiii

CHAPTER 1: INTRODUCTION

1.1 General.....	1
1.2 Problem.....	2
1.3 Objective and Scope of the test.....	2
1.4 The Outline of the report.....	4

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction.....	6
2.2 Deterioration and Rehabilitation System of piles	
2.2.1 Deterioration of Marine Piles.....	6
2.2.2 Pile Rehabilitation Systems.....	8
2.2.3. Experimental study on Rehabilitation using Concrete Jacket.....	11
2.3. Review of Shear friction equations.....	14

2.3.1. Hanson (1960).....	15
2.3.2 Birkeland and Birkeland (1966)	15
2.3.3 Mast (1968)	16
2.3.4 Hofbeck, Ibrahim and Mattock (1969).....	18
2.3.5 Mattock (972)	19
2.3.6 Mattock (1974)	19
2.3.7 Rath (1977).....	22
2.3.8 Loov (1978).....	23
2.3.9 Shaikh (1978).....	24
2.3.10 Walraven, Frenay and Pruijsere (1987).....	25
2.3.11 Mau and Hsu (1988)	25
2.3.12 Canadian Code (CSA,1994).....	26
2.3.13 Kriski and Loov (1996)	27
2.3.14 Ali and White (1999)	28
2.3.15 ACI code (1999).....	31
2.3.17 The ACI Commentary, ACI (2002).....	32
2.3.17 Kahn and Mitchell (2002)	34
2.3.18 Canadian Code (2004).....	35

CHAPTER 3 : EXPERIMENTAL INVESTIGATION

3.1 Introduction.....	37
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3.2 Test Specimens.....	37
3.3 Concrete Mix Design.....	38
3.4 Steel Piles and Reinforcement	39
3.5 Test set-up.....	40
3.6 Test Results and Discussion	
3.6.1 Steel Piles and Concrete Jacket without Connection	
3.6.1.1a Round Steel Pipe Shape.....	41
3.6.1.1b Wide Flange Steel Shape	42
3.6.1.2 Concrete Strain	42
3.6.1.3 Steel Strain	43
3.6.2 Steel Piles and Concrete Jacket with Connection	43
3.7 Summary.....	45

CHAPTER 4: TEST RESULT AND DISCUSSION

4.1 Introduction	47
4.2. Proposed Models	47
4.3 Comparison of different types of specimens	49
4.4. Discussions on Proposed Models	
4.4.1. Formula # 1 (Birkeland).....	52
4.4.2. Formula # 2 (Mattock, 1972)	53
4.4.3. Formula # 3 (Shaikh 1978)	54
4.4.4. Formula # 4 (Walraven, 1987)	55
4.4.5. Formula # 5 (Modified Mattock ,1988).....	57

4.4.6. Formula # 6 (Canadian code, 1994)	58
4.4.7. Formula # 7 (Canadian Standard Association, 2004)	59
4.4 Selected Model for the	60
4.5 Summary	61
 CHAPTER 5: CONCLUSION & RECOMMENDATIONS.....	63
 REFERENCES.....	66
APPENDIX A.....	70
APPENDIX B.....	77

Figure A.11: Concrete jacket cracked (a) New concrete jacket without connection to steel pile(b)
New concrete jacket with mechanical connection to steel pile.....76

LIST OF TABLES

Table 4.1.1: Calculated shear friction strength using formula # 1.....	53
Table 4.1.1: The results of V_t/V_{sf} for formula # 1.....	53
Table 4. 2 .1: Calculated shear friction strength using formula # 2.....	54
Table 4. 2 .2: The results of V_t/V_{sf} for formula # 2.....	54
Table 4.3.1: Calculated shear friction strength using formula #3.....	55
Table 4.3.2:The results of V_t/V_{sf} for formula # 3.....	55
Table 4.4.1: Calculated shear friction strength using formula # 4	56
Table: 4.4.2: The results of V_t/V_{sf} for formula # 4.....	56
Table 4.5.1: Calculated shear friction strength using formula # 5.....	57
Table: 4.5.2: The results of V_t/V_{sf} for formula # 5.....	57
Table 4.6.1: Calculated shear friction strength using formula # 6.....	58
Table 4.6.2: Value of shear resistance (KN), using formula # 6.....	58
Table 4.7.1: Wide flange with a clamp connection.....	59
Table 4.7.2: Wide flange without a clamp connection.....	60
Table 4.7.3.Circular tube without clamping connection.....	60
Table 4.7.4: Circular tube column with a clamp connection.....	60

Table 4.8.1: Value of shear resistance (KN), using CSA, 1994 with changed k values.....61

Table 4.8.2: The results of V_t/ V_{sf} for Canadian Code 1994 (F3) with changed k values.....61

Table A 3.1: Test results of Steel pile with concrete jacket and no connection..... 70

Table A.3.2: Steel piles with Concrete Jacket and Mechanical connection.....70

Table A.3.3: Steel piles with Concrete Jacket and Welded connection.....71

LIST OF SYMBOLS

v_n = Shear strength on the interface of the cracked concrete (MPa)

V = transverse shear force at location under consideration (MN)

A_s = the total area of shear plane (mm^2)

f_y = the yield stress of reinforcement (MPa)

ρ_v = Area ratio of steel to concrete

f_c' = compressive strength of concrete (MPa)

ϕ = angle of friction (degrees)

V_n = the shear resistance of a precracked surface (kN)

A_{vf} = the area of reinforcement crossing the surface (mm^2)

K_1 = the empirical factor for concrete contribution in shear friction (MPa)

A_{sa} = cross sectional area of one bar that is crossing the shear plane at angle θ (m^2)

A_{sb} = cross sectional area of one bar B, crossing the shear plane at angle $(90^\circ + \theta)$ (m^2)

s_a = space between rebar A, inclined in θ to the shear plane (m)

s_b = space between rebar B, inclined in $90^\circ + \theta$ to the shear plane (m)

K = coefficient relating to maximum and average shear stresses

f_{sa} = stress in rebar A, at ultimate (MP)

f_{sb} = stress in rebar B, at ultimate (MP)

f_y = yield strength of rebar (MP)

w = width of the shear plane (m)

C_s = constant used for the effect of concrete density

μ = the coefficient of friction

λ = constant used to account for concrete density

b_v = width of the interface (m)

k = shear friction factor

ϕ = strength reduction factor

ϕ_c = resistance factor for concrete

ϕ_s = the resistance factor for the reinforcement

σ = the normal stress on plane

α = the angle between the shear friction reinforcement and the shear plane (degrees)

δ_u = relative displacement of the face of the crack in the direction of the reinforcement (m)

CHAPTER 1

INTRODUCTION

1.1 General

Deterioration of marine piles is a serious problem affecting structures due to the aggressive marine environment. This deterioration may be a result of wetting and drying cycles, freeze-thaw cycles, or chloride attack. The key factors that contribute to early deterioration of steel pile are penetration of chloride ions that cause corrosion of the steel structures in marine environment.

Many marine structure components are now in need of urgent repair to extent their service life. In Canada, alone hundreds of millions of dollars are spent years for the repair or rehabilitation of marine infrastructure. Steel piles normally rust after about thirty years of exposure. The rate of rusting can vary with location. However, in most cases piles experience severe deterioration to the point where the structure is unsafe to use. The sever corrosion is typically limited to about one meter band approximately at the water level known as splash zone. The section above and below the water level, have much slower rate of corrosion.

The steel pile can be easily repaired by encasing the rusted pile into a concrete jacket. A special fiberglass bag is formed after the pile has been thoroughly cleaned by high-pressure water blasting to prepare the old steel piles. The technique involves constructing jacket using concrete and steel elements.

It has been shown that the method improves the bending strength, the shear capacity, the stiffness, the ductility and axial load carrying capacity of the structural element.

1.2 Problem Definition

Attaching the concrete jacket above water is a simple task usually achieved by welding steel brackets to provide composite action between the old steel pile and the new concrete encasing. Experienced diver is needed to carry out welding the steel reinforcement below water. Welding the new steel reinforcement and brackets under water is the most expensive task of the rehabilitation process.

Therefore, using welded shear connectors in the marine piles during the rehabilitation is uneconomical. It also causes extra afford to make any surface treatment to enhance the interface shear between the old and new layers.

This report describes how to use a mechanical anchorage system to increases the interface shear and friction resistance of the rehabilitated piles. The mechanical connectors increase the area of shear surface and enhance the capacity of the pile.

The parameters for this investigation are the concrete compressive strength of the new jacket, the amount of radial steel reinforcement, the shape of the rehabilitated steel pile and the type of connection between the old rehabilitated steel pile and the new concrete jackets.

1.3 Objective and Scope of the work

Placing fresh concrete against steel surface is routine task in building construction. For some time now, the placement of the concrete always has been gaining importance because of the increasing need for rehabilitation and strengthening the existing structures.

The transfer of internal stress across the bond interface between the concrete and steel is critical aspect. Various formulas had been developed in the past to calculate the interface shear force for

a treated surface of concrete and steel. However, the current research is focused to find a suitable formula for calculating the interface shear friction force between concrete and steel, where there is no surface treatment.

The method has the following features:

1. It is based on simple, but rational probabilistic contact models.
2. It is considered the interface roughness and concrete strength through an introduced relationship between crack roughness and fracture energy.
3. Reasonable correlation between the suggested method and existing experimental result for both normal and high strength should be achieved.

The experimental investigation included testing of twenty four specimens to evaluate the behavior of the rehabilitated steel pile. The maximum load and the load-slip deformation curve were recorded for all of the tested specimens.

Different factors affect the friction capacity between the old steel pile and the new concrete jacket. The round steel pile shapes and wide flange steel pile shapes are the two types of pile used in this investigation. Surface friction is the most important factor for the case of round steel pile shape. The surface friction includes local roughness, global roughness and confinement pressure. The wide flange steel pile has bigger friction due to the combination of surface friction and the shear concrete friction.

Two types of concretes were used namely; normal and high strength concrete jackets. The concrete shear and friction can contribute a lot of resistance to the total friction between the wide flange pile and the new concrete jacket.

The rehabilitated pile was tested without connection between the old steel pile and the new concrete jacket. Other specimens were tested using mechanical or welding connections. The results were compared to the theoretical interface due to stress.

The results are applicable to a wide variety of rehabilitation and repair projects involving the placement of new concrete over existing steel for structural purposes. The conventionally reinforced concrete around the damaged pile can protect the steel and strengthen the structure.

The investigation revealed that the underwater expensive welding process can be eliminated and two bolted strap connections can be designed to provide adequate mechanical connection between the old steel pile and the new concrete jacket. The installation is quick, easy and economical. The disadvantage of this system is the concrete deteriorates leaving the steel pile exposed and subjected to rust.

1.4 Outline of the report

Chapter 2 contains the literature review that is relevant to this investigation. A brief study of the cause of corrosion and preventions is presented. Researches done on marine pile rehabilitation system using reinforced concrete cover were discussed. A number of the previous work concerning the interface shear and friction between new and old layer of concrete and steel, models and expressions, developed to estimate the interface shear resistance were presented.

Chapter 3 represents exclusive data from the experimental test performed by previous researchers. The description of test specimens and the results were discussed on this chapter.

Chapter 4 contains the charts that represent the results of calculated interface shear resistance using various formulas to compare the interface shear resistance, developed from the test

specimens. The main point of interest in this chapter is to determine the most preferable proposal of the desired investigation. Depending on the values of the charts, the most adequate formula was selected for calculating the capacity of the rehabilitated pile.

Chapter 5 includes the conclusion and recommendation of the report.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The main reason of deterioration of marine piles is corrosion, which develops from the action of the oxygen of saline water to the pile. As a result, the strength of element may be significantly reduced by the deterioration of the structure. The deterred marine pile can be rehabilitated by encasing it by reinforced concrete jacket. In 1990, this technique was used in Holland to repair the piles of a bridge across a river (Contec ApS Denmark, 2004). The concept of shear friction, which is based on the treatment of friction in mechanics, can be used to explain the behavior of shear transfer at the concrete to steel interface of the new pile. Extensive experimental research leads to the development of the shear friction equation. The previous studies performed on corrosion of marine piles, rehabilitation systems, development of shear friction equation and review of CSA (1994 and 2004) and ACI (2002) codes are presented in this chapter.

2.2 Deterioration and Rehabilitation System of piles

2.2.1 Deterioration of Marine Piles

In a previous study conducted by British Standard BS6349-1:2000 under Clause 5923, it has been reported that these metals are more prone to corrosion in a marine environment than inland primarily due to galvanic cell formation. According to their research, differential aeration, or strong currents along with the solution of salts in the water acting as the electrolyte, initiates the formation of anodes and cathodes in different metals or on the same metal and thus leads to corrosion. The other mentioned causes of corrosion are removal of rust products by wave action,

rubbing or cyclic deflections and inadequacy of planned preventative inspection and/or maintenance by owners.

Figure 2.1: Deteriorated Marine piles (ICE briefing sheet, 2000)

Figure 2.2: Concentrated corrosion on a Universal Bearing Pile (ICE briefing sheet, 2000)

The repeated removal of the protective corrosion product layer causes the area of the pile to become anodic to the unprotected areas. The steel that is electrically connected to metals having nobler potentials, or where weld metals are significantly less noble than the parent metal, causes bi-metallic corrosion. As described by British Standard (BS6349-1:2000), microbiological

activity in concentrations around (and below) the low water zone will cause accelerated corrosion.

It was recommended by the same researchers that a specific solution will be developed using a thicker steel section. A higher grade of steel or the addition of steel plates using a high quality protective coating and an electrical bonding system with cathodic protection will ensure high bending moments to occur in areas away from anticipated concentrated corrosion sites.

2.2.2 Pile Rehabilitation Systems

The 'Reinforced Concrete Pile Repair System' (Contec ApS Denmark, 2004) is a structural underwater repair system developed in 1990, for the piles of a bridge crossing a river in Holland. This technique can be used for both concrete and steel piles. The reinforcement or rings are welded or glued against the steel to get maximum bending high performance steel pile. Steel and/or glass fiber reinforced resin forms with a smooth finish are used as a temporary formwork around the piles. Inside the formwork, the pile is secured by pre-cast distance keeper concrete. Just before placing the formwork, the pile must be cleaned again with high-pressure water to remove marine growth on the pile.

After the preparations of the surface, concrete pump must be connected to the formwork and the formwork can be filled with the high performance or the ultra high performance concrete until the necessary level. After approximately 36 hours of curing, the formwork can be removed.

Depending on the type of construction, type of damage, type of loading and if it is necessary to re-strengthening a structure a high performance concrete (HPC) or an ultra high performance concrete (UHPC) can be used. The used concrete exhibits very low shrinkage, thus concrete and surrounding steel and /or underlying steel or concrete remain in physical contact after curing.

Due to heat of hydration, the concrete product can eliminate the risk of thermal cracking and deterioration, even when casting in large volumes. High performance concrete (HPC) or ultra high performance concrete (UHPC) is pump able up to several 100 meters through 50 mm (2 in) and 75 mm (3 in) hoses. Due to viscosity and high inner cohesion of the mixed Concrete, there is no risk of washing out cement particles, separation or mixture with water when cast below water level.

Figure 2.3: Formwork shells for repairing piles (Contec ApS Denmark, 2004)

Figure 2.4: Connection formwork for repairing piles (Contec ApS Denmark, 2004)

Due to ultra high strength and durability, the fatigue of the used concrete strength is outstanding compared to normal concrete. Application of fibers in combination with reinforcement rods, the tensile and bending strength reaches considerable levels as the material shows significant

ductility and high fracture energy. The chloride penetration of high performance concrete (HPC) or ultra high performance concrete (UHPC) is very low compared to traditional concrete. Due to the high electric resistivity and the impermeability of the high performance, traditional reinforcement and high quality reinforcement like prestressing cables are well protected against corrosion.

Figure 2.5: Reinforced HPC for structural strengthening of pile
(Contec ApS Denmark, 2004)

Figure 2.6: Casted new piles, (Contec ApS Denmark, 2004)

Even with a thin cover of 10 – 15 mm, the reinforcement will be protected against corrosion in an aggressive environment for more than 50 years.

2.2.3 Experimental study by Vandoros and Dritsos on Rehabilitation using Concrete Jacket (2006)

Vandoros and Dritsos (2006) presented an experimental investigation of strengthening concrete column by placing concrete jacket in their study of shear transfer across interface between new and existing concrete. The concrete jacketing offers a significant enhancement to the structural characteristics (stiffness, strength and displacement capacity) of RC elements.

A monolithic specimen M (Figure 2.7) was used for the experiment that had the same cross sectional dimensions and the same longitudinal and stirrup reinforcement as the strengthened specimens. Three different methods of strengthening techniques were used to construct half height full-scale ground floor columns. The methods were: welding the jacket stirrup ends together placing dowels (specimen E) ; jacket stirrup end welding (specimen N) ; placing bent down steel connector bars welded to the original column longitudinal reinforcement bars and the respective bars of the jacket (specimen W). The construction method was different for every strengthened specimen. For specimen N (figure 2.8), the end of the four lowest stirrups of jackets were welded together. For specimen E (figure 2.9), the long branch of the L-shaped dowel was placed 100 mm into the original column and protruded 50 mm to the concrete jacket. In addition, the ends of the four lowest stirrups of the jacket were welded together, as was performed for specimen N. Poured concrete was used for specimen N, E and M. However, shotcrete concrete was used for specimen W (figure 2.10).

For all strengthened specimens, the thickness of the jacket of was 75 mm and there was no special treatment at the interface between the original column and the jacket in order to eliminate the influence of roughening in the effectiveness of the test method.

The control specimen M (figure 2.7) achieved the largest maximum load capacity and had good deformability and high ductility. The maximum strength of specimen N was similar to that of specimen W (which also had a low jacket concrete strength) but was significantly lower than that of specimen E. This poor behavior occurs due to poor interface connection between the jacket and the original column and the use of poured concrete instead of shotcrete for constructing the jacket. The poor performance of specimen N cannot be considered as acceptable in earthquake loading situations.

Figure 2.7: Geometrical characteristics of the monolithic specimen (M)

(Vandoros and Dritsos , 2006)

The jacket reinforcement in specimen N and E did not buckle since the stirrup ends were welded together. The loss of bond between the jacket and the original column resulted in all the damaged being restricted to the jacket, while the original column remained undamaged.

Figure 2.8: jacket stirrup end welding (specimen N)

(Vandoros and Dritsos , 2006)

Figure 2.9: welding the jacket stirrup ends together placing dowels (specimen E)

(Vandoros and Dritsos , 2006)

Due to presence of dowels in specimen E (figure 2.9), there were less damage for the jacket. No buckling was observed and this can be attributed to the use of welded stirrups.

The bond between the jacket and the original column in specimen W was very good and was significantly better than that of specimen N and E. This can be attributed to the shotcrete rather than cast poured concrete. The final condition of the jacket was very good and the damage was limited to the lower part of the jacket. Specimen W experienced very good ductility due to the significant contribution of the steel bent down connectors.

Figure 2.10: Bent down steel connector geometry of specimen W

(Vandoros and Dritsos , 2006)

2.3 Review of Shear friction equations

The concept of shear friction was first introduced to the U.S.A. from Europe in 1958 and was further developed by Birkeland (1966) and others. The ACI building code at that time allowed

design for shear transfer to be based on the shear friction hypothesis proposed by Birkeland (1966) and Mast (1968). Many researchers have proposed equations based on their own test results since then. Most of these equations are reviewed below.

2.3.1 Hanson (1960)

Hanson (1960) made a study to correlate push-off tests and beam tests by comparing shear slip relations of the two types of specimens. He concluded that push-off tests are representative of beam tests for “rough bonded” connections.

According to Hanson (1960), the maximum horizontal shear strength is reached at a slip of 0.13 mm (0.005 in). However, the idea to fix a limit for slip was quite controversial. According to Grossfield and Birnstiel (1962), larger shear strengths would be recorded if larger slips were permitted. Patnaiks (1992) stated that this limit greatly restricts the full utilization of strength of composite concrete beams.

2.3.2 Birkeland and Birkeland (1966)

Birkeland (1966) was the first to introduce a parabolic function for shear strength along an interface. In this shear friction concept, shear failure was considered as slip not as a tension crack in the usual sense. In this investigation, the shear friction hypothesis was built up and shear friction equation was proposed. According to his research work, the ultimate shear resistance can be written as:

$$V_u = A_s f_y \tan \phi \quad (2.1)$$

Where, A_s is the total area and f_y is the yield stress of reinforcement. If the equation is divided by the area of shear plane, the relationship expressed in terms of stress:

$$v_u = \rho_v f_y \tan \phi \quad (2.2)$$

Where, A_s is the total area; f_y is the yield stress of reinforcement and ϕ is the angle of internal friction.

$\tan \phi = 1.7$ for monolithic concrete

$\tan \phi = 1.4$ for artificially roughened construction joint

$\tan \phi = 0.8$ to 1.0 for ordinary construction joints and for concrete to steel interface

The above formula was formed for an interface of a crack in monolithic concrete, but the concept is still eligible for artificially roughened or smooth construction joints and for concrete to steel interface. The shear is developed by friction rather than by bond. The reinforcement across the interface is subjected to tension stress, with dowel action presumed insignificant. Reinforcement needs to be properly anchored on both sides of the failure plane to develop yield in steel. The cohesive strength was neglected in this shear friction theory to form a lower bond for test data.

2.3.3 Mast (1968)

The shear friction hypothesis was explained in more detail in this work. According to this experimental report, reinforcement should be provided for any externally applied tension across the crack, in addition to the reinforcement required by shear friction theory. According to the investigation, the full tensile anchorage provided on both sides of the crack should be reliable to develop the yield strength of the steel along with finite amount of slip and subsequent separation

of the concrete pieces on each side of the crack must take place. The value of angle of internal friction (ϕ) was assumed to be independent of concrete strength and stress level. Mast's shear friction equation is the same as equation 2.1. However, maximum shear stress was set as:

$$v_u = \rho_v f_y \tan \phi \leq 0.15 f'_c \tan \phi \quad (2.3)$$

The value of friction coefficient $\tan \phi$ was considered more carefully by test results.

$\tan \phi = 1.4$ to 1.7 for a crack in monolithic concrete

$\tan \phi = 1.4$ for a rough interface between precast and cast-in-place concrete

$\tan \phi = 1.0$ for concrete cast against steel

$\tan \phi = 0.7$ for concrete cast against smooth concrete

Birkeland (1966) suggested another alternate equation for shear transfer strength, where a constant value was suggested for the equation 2.1. The equation is:

$$v_n = 2.78 \sqrt{\rho_v f_y} \text{ (MPa)} \quad (2.4)$$

This equation is simple; however, is not considered as a reliable formula for the following reasons:

- 1) The coefficient 33.5 has an awkward unit of $\sqrt{\text{MPa}}$,
- 2) The apparent value of μ will vary with clamping stress from less than 1.0 to infinity, and therefore it has no logical relationship with the fitted curve, and
- 3) It does not present the designer with the visual impact and simplicity of a concept, which is embodied in the linear equation.

2.3.4 Hofbeck, Ibrahim and Mattock (1969)

Hofbeck, Ibrahim and Mattock (1969) reported a study of the shear transfer strength of reinforced concrete both with and without crack existing along the shear plane. In this experimental investigation, reinforcement across the shear plane was in form of closed stirrups, anchored by wrapping round the longitudinal reinforcement. Crack was assumed to occur along the shear plane, reinforcement was assumed to cross the crack at right angle. The faces of the crack were rough and irregular, so that when slip occurred along the crack, the pieces of the concrete on either side of the crack separated slightly, which created tension in the steel and compression along the concrete across the crack. This compressive stress provided resistance to slip along the crack by virtue of friction between the rough and irregular faces of the crack. Taking $\tan\phi$ as the coefficient of friction between the faces of the crack, the shearing resistance along the crack was calculated by the equation 2.2.

In their study reported that due to the effect of concrete strength on the shear transfer strength, it appears that equation 2.2 would start to become unconservative for concrete of strengths of 27.5 MPa (4000 psi) or over, when $\rho_v f_y$ exceeds 4 MPa (600 psi). For the cracked specimens the values of $\rho_v f_y$ in excess of 1.4 MPa (200-psi) the shear strength is a combination of cohesion and friction effects, with an upper limit of strength of $0.3 f_c'$. The study indicates that, using $\tan\phi$ equal to 1.4, the shear friction theory is reasonably conservative for values of $\rho_v f_y$ less than $.15f_c'$ or 4 MPa (600 psi) (whichever is less), using reinforcement with specified yield point of 400 MPa (60 ksi) or less.

It is recommended in the report a lower value of $\tan\phi$ and different upper limit to $\rho_v f_y$. Suitable values would be $\tan\phi$ equal to 1.0, and $\rho_v f_y$ not greater than $0.3 f_c'$ or 10 MPa (1.500 ksi), (whichever is less).

2.3.5 Mattock (1972)

Mattock's (1972) Shear friction (or the modified shear friction) theory considered the effect of concrete strength using a constant (K_1). However, Mattock's (1972) equation does not include any cohesive term. He suggested the following formula for shear strength of precracked surface:

$$V_n = 0.8A_{vf}f_y + A_c K_1 \quad (2.5)$$

Where, V_n is the shear resistance of a precracked surface (MN); A_{vf} is the area of reinforcement crossing the surface (mm^2); f_y is the reinforcement yield strength (MPa); A_c is the area of concrete surface (mm^2) and K_1 is the empirical factor for concrete contribution in shear friction. For the calculation, the value of K_1 is used as 2.8 MPa for normal weight concrete. This equation did not provide good result for the following reason:

1. Mattock's equation does not include any cohesive term.
2. It also does not provide an accurate representation of the effect of concrete strength in the bond strength.
3. The theory does not consider the contribution of the contact stresses across the crack, nor does it treat the contact stresses sustainable by the concrete matrix.

2.3.6 Mattock (1974)

Mattock (1974) suggested the formulae for shear transfer in concrete having reinforcement at an angle θ to the shear plane. To determine the Shear transfer behavior of initially untracked concrete with orthogonal reinforcement at an angle θ to the shear plane, the stress in the reinforcement was assumed proportional to the component of the relative displacement (δ_u) of

concrete on the two sides of the shear plane in the direction of the reinforcement. The relative displacement (δ_u) at ultimate was assumed to be constant and strain in reinforcement normal to the shear plane. Therefore, the shear transfer strength equation is:

$$v_u = \frac{\sin 2\theta}{2} \left[\frac{A_{sb} f_y}{s_b w} - \frac{A_{sa} f_{sa}}{s_a w} \right] + K \tau_{xy} \quad (2.6)$$

Where, A_{sa} is the cross sectional area of one bar that is crossing the shear plane at angle θ (m^2) and A_{sb} is the cross sectional area of one bar B, crossing the shear plane at angle $(90^\circ + \theta)$ (m^2); s_a is the space between rebar A, inclined in θ to the shear plane; s_b is the space between rebar B, inclined in $(90^\circ + \theta)$ to the shear plane (m). K is the coefficient relating to maximum and average shear stresses; f_{sa} is the stress in rebar A, at ultimate (MPa); f_{sb} is the stress in rebar B, at ultimate (MPa); f_y is the yield strength of rebar (MPa); w is the width of the shear plane (m).

The shear transfer behavior of initially uncracked concrete in which the shear plane crossed at angle θ by a parallel array of bars will be exactly similar to the case of orthogonally reinforced concrete. In this case, the shear transfer strength (MPa):

$$v_u = \frac{A_s f_s}{2 s w} \sin 2\theta + K \tau_{xy} \quad (2.7)$$

For the calculation of shear transfer behavior of initially cracked concrete with orthogonal reinforcement at angle θ to the shear plane, the strain in the reinforcement was assumed proportional to the component of the relative displacement (δ_u) of the face of the crack in the direction of the reinforcement. The relative displacement at ultimate was assumed to be constant

and strain in reinforcement normal to the shear plane. The relative displacement (δ_u) takes place at a direction at angle ϕ to the crack. Where, ϕ was assumed to be equal to $\tan^{-1} \mu$ and μ is the coefficient of friction between the crack faces. $\mu = 0.8$, hence $\phi = 38.7^\circ$. Therefore, the shear transfer strength equation is:

$$v_u = 400 + 0.8 f_y, \text{ but } > 0.3 f'_c \quad (2.8)$$

Figure 2.11: Shear transfer in (a) initially uncracked concrete with orthogonal reinforcement (b) initially cracked concrete with orthogonal reinforcement (Mattock, 1974)

Shear transfer behavior of initially cracked concrete with parallel reinforcement at an angle θ to

the shear plane is similar to that of orthogonally reinforced concrete; providing the angle θ is large enough so that tension is developed in the reinforcement. The relative displacement (δ_u) was assumed to be placed in a direction at angle $(90+\theta)$ to the crack, when θ is less than $(90-\phi)$ and in a direction at an angle ϕ to the crack, when θ is equal to or greater than $(90-\phi)$. Therefore, the equation becomes as below:

$$v_u = 400 \sin 2\theta + \frac{A_s f_s}{s w} [0.8 \sin^2 \theta - 0.5 \sin 2\theta] \quad (2.9)$$

The resistance to shear along the crack due to dowel action was assumed to vary directly with $\sin\theta$. Whereas, shear along the crack is resisted by the component of the dowel force parallel to the crack that will be $\sin\theta$ times the dowel force normal to the bar.

2.3.7 Rath (1977)

In order to obtain a straight-forward design expression for use in every day design practice, Rath (1977) suggested a parabolic shear strength equation for a monolithically cast direct shear interface that is similar to Birkeland's equation 2.2 and is expressed as follows:

$$v_u = C_s (1000\mu) \sqrt{\rho_v f_y} \quad (2.10)$$

Where, C_s is a constant used for the effect of concrete density

$C_s = 1.0$ for normal weight concrete

$C_s = 0.85$ for sand light-weight concrete

$C_s = 0.75$ for all light-weight concrete

In addition, μ is the coefficient of friction

$\mu = 1.4$ for monolithically cast

$\mu = 0.6$ for smooth concrete

So that, the equation for monolithically cast concrete can be written as:

$$v_u = 3.11 C_s \sqrt{\rho_v f_y} \text{ (MPa)} \quad (2.11)$$

2.3.8 Loov (1978)

Loov first considered the influence of concrete strength on the shear transfer strength equation.

The researcher proposed a shear transfer equation in the following form:

$$\frac{v_u}{f_c'} = k \sqrt{\frac{\sigma}{f_c}}$$

It can be written as:

$$v_u = k \sqrt{(\sigma f_c')} \quad (2.12)$$

Where, k is the constant, v_u is the shear stress and σ is the normal stress on the shear plane at failure. In the shear friction hypothesis, this normal compressive stress is equal to the tensile stress in reinforcement across the crack. Equation 2.13 can be written as:

$$v_u = k \sqrt{\rho_v f_y f_c'} \quad (2.13)$$

The author considered that the concrete strength of most specimens at that time were around 20 to 30 MPa and suggested the value of k equal to 0.5 for an initially uncracked shear interface. This equation is almost the same as the equation of Birkelands (1966).

2.3.9 Shaikh (1978)

Shaikh (1978) suggested an equation that was expressed as in terms of reinforcement ratio, yield strength of reinforcement and the effect of friction and shear. The frictional constant is expressed in terms of λ . Where, λ is a constant used to account for concrete density. The equation is given below:

$$v_u = \phi \rho_v f_y \mu_e \quad (2.14)$$

Where,

$$\mu_e = \frac{1000 \lambda^2}{v_u} (\text{psi}) \quad (2.15)$$

The value of λ is 1.0 for normal weight concrete; 0.85 for sand lightweight concrete and 0.75 for all lightweight concrete. If the equation for v_u and μ_e (equations 2.14 and 2.15) are combined; a parabolic equation for v_u with respect to the clamping force is obtained:

$$v_u = \lambda \sqrt{6.9 \phi \rho_v f_y} \leq 0.25 f_c' \lambda^2 \text{ and } 6.9 \lambda^2 (\text{MPa}) \quad (2.16)$$

Where, the strength reduction factor, ϕ is 0.85 for shear. Equation does not include the concrete strength variation.

2.3.10 Walraven, Frenay and Pruijsere (1987)

Concrete in the contact area may crush before the slip could mobilize enough crack dilation for the yielding of the crossing reinforcement. Walraven (1987) formalized the problem using a consistent contact model with a probabilistic basis. Based on 88 push-off tests with a wider range of concrete strength, they came up with a shear transfer strength equation for a precracked shear interface:

$$\begin{aligned}v_n &= C_1 (\rho f_y)^{C_2} \\ C_1 &= 0.878 f_c'^{0.406} \\ C_2 &= 0.167 f_c'^{0.303}\end{aligned}\tag{2.17}$$

2.3.11 Mau and Hsu (1988)

In their discussion on the paper by Walraven, Frenay and Pruijsers (1987), Mau and Hsu (1988) suggested that the non-dimensional web reinforcement index, $\rho_v f_y / f_c'$, was the dominant factor in the normalized shear strength. Also based on statical analysis, a shear strength equation was proposed as:

$$\frac{v_u}{f_c'} = 0.66 \sqrt{\frac{\rho_v f_y}{f_c'}} < 0.3\tag{2.18}$$

This equation is very similar to the equation proposed by Loov (1978). The researchers chose 0.66 (k in Loov's equation 2.10) for both initially cracked and initially uncracked shear interface.

2.3.12 Canadian Code (CSA, 1994)

Loove and Patnaik (1994) performed the study for horizontal shear stress by testing 16 beams. Three different concrete mixes were used. The concrete strength of f'_c of the web and the flange concrete was evaluated using standard cylinders. Shear friction equations assumed yielding of reinforcement crossing the interface. Beams were simply supported and were loaded with a point load at the center span. These beams were intended to be very strong in diagonal shear and flexure so that they would fail in horizontal shear prior to fail in any other mode. Different beams failed in different ways. Some of them faced cracking or slip at the interface. Some of them failed in horizontal shear and along the half of the beam, the interface crack deviated through the narrow portion of the web rather than following along the joint. Slip was insignificant up to horizontal shear stress of 1.5 to 2 MPa (220 to 290 psi).

Figure 2.12: Typical roughness achieved for the test beams

Thereafter, the horizontal shear stress increases with slip, up to a slip ranging from 0.3 to 0.8 mm (0.01 to 0.03 in). Based on equation 2.14, the horizontal shear friction stress were calculated using the following equation:

$$v_n = k\lambda\sqrt{((0.1 + \rho f_y) \times f'_c)} \quad (2.19)$$

Where, k is the shear friction factor; λ is the empirical coefficient for surface roughness effect on shear friction capacity; ρ_v is the reinforcement ratio; f_y is the reinforcement yield stress (MPa) and f_c' is the concrete compressive strength (MPa). It provides common basis for comparison, and was included in the CSA Code (1994). One advantage of this equation is that any consistent system of units can be used without changes.

2.3.13 Kriski and Loov (1996)

Loov and Kriski (1996) found the shear resistance in composite beams was a function of the concrete strength and the amount of reinforcement crossing the failure crack. The researchers compared their test result to the shear friction stress calculated according to CSA A23.3-94 .The equation for shear friction, based on their recent work is:

$$v_f = \lambda \phi_c k \sqrt{\sigma f_c'} + \phi_s \rho_v f_y \cos \alpha \quad (2.20)$$

Where, v_f is the factored shear stress resistance (MPa). The shear friction factor k is 0.5 for concrete placed against hard concrete and 0.6 for concrete placed monolithically. In this equation λ is a factor to account for low density concrete, ϕ_c is resistance factor for concrete, ϕ_s is the resistance factor for the reinforcement, σ is the normal stress on plane, f_c' is the specific compressive concrete strength, ρ_v is the transverse reinforcement ratio, f_y is the specific yield strength of the reinforcement and α is the angle between the shear friction reinforcement and the shear plane.

In this model, the stirrups and longitudinal reinforcement are assumed to provide a clamping

force thereby increasing the friction force, which can be transferred across a potential failure crack. For beams, the clamping stress is provided by longitudinal reinforcement as well as by stirrups but neither is necessarily at yield. The force at a crack is limited to force that can be develop by bond or anchorage.

In the experimental study to determine the strength of beam based on shear friction done by Kriski and Loov (1996), twelve rectangular concrete beams were tested to investigate the applicability of the proposed shear friction to intermediate length beams. The shear friction model predicts a higher strength in smaller stirrup spacing because the increase in number of stirrups across the shear plane increases the resisting friction force along the crack. The average V_t/V_{sf} was 1.24, where V_t represents the failure load using shear friction. The load V_t is on the mid span. A slightly lower value of strength reduction factor k makes the predicted capacity conservative to the test result. In this research, it was suggested that for routine design, it is desirable to use slightly lower shear strength to allow for the possibility of smoother interfaces. Therefore, for composite construction, a factor of 0.5 for k is suggested. For monolithic construction, there is no joint at the shear interface. Therefore, the additional level of safety to account for the possible variation in roughness is not warranted and the value of k can be kept as 0.6. The factor varies for lightweight concrete as per ACI 318-02 stated in Section 2.2.1.

2.3.14 Ali and White (1999)

Ali and White (1999) stated in their work that the shear friction is the term used to describe the mechanism of shear transfer along a concrete-to-concrete interface subjected to simultaneous shear and normal compression (Figure 2.13). Other terms are used to describe the same phenomenon: concrete-to-concrete friction, interface shear transfer, and aggregate interlock.

When shear is applied to an initially cracked surface, or a surface formed by placing one layer of concrete on an existing layer of hardened concrete, relative slip of the layers is accompanied by the separation of the surfaces (Figure 2.14).

Figure 2.13: Crack profile and shear friction deformation of (a) Normal strength concrete & (b) High strength concrete (Ali & White, 1999)

The factors that affect shear friction capacity are global roughness, local roughness, concrete strength, reinforcement crossing the interface, pressure applied normal to the surface etc. Global

roughness increases tortuousness that results in increasing both contact area and a contact angle. Local roughness increases roughness of the aggregates and the fractured matrix results in increasing frictional force under the same applied compression. If there is reinforcement across the crack, its elongation across the crack provides compression forces on the interface (Figure 2.15), and crack slip causes bending and shear forces in the reinforcement.

Hence, shear is transmitted across the crack by interlock of aggregate protrusions on the cracked surface, and friction that is caused by asperities combined with dowel action of reinforcement crossing the crack surface. Shear friction increases with the decrease of shear reinforcement ratio. According to Duthinh (1997), the proportion of shear force, carried by the shear friction increases with the decreases of the shear reinforcement.

Ali and White (1999), in their research work stated that sometime pressure applied normal to the surface increases the resisting friction force by increasing both the interlocking contact area and the contact pressure and consequently increase frictional resistance.

Crushing occurs when these aggregates in the interface reach their compressive capacity, which is directly related to the compressive strength. Moreover, in high strength concrete global roughness of intentionally roughened cracks is less than for concrete with lower strength.

Figure 2.14: Contact stresses along deformed crack (Ali & White, 1999)

In other studies made by Walraven (1995) and Walraven and Stroband (1994), it was stated that shear transfer across the interface depends on the compressive stress across the crack, the crack width, the crack slip, the strength of the mortar and the concrete, the coefficient of friction between the mortar and the aggregate, the maximum size and volumetric percentage of aggregate.

Figure 2.15: Force Developed in reinforcement (ACI-ASCE Committee, 1973 & Gregor, 1988)

2.3.15 ACI code (1999)

According to ACI (1999), shear friction is the shear force transferred across a joint by friction between the surfaces. The frictional force is a function of the normal force applied and the coefficient of friction μ between the surfaces. The normal clamping force may result from the transverse shear reinforcement, and the shear is given by Equation 2.21.

$$V_n = \mu A_v f_y \quad (2.21)$$

Where, V_n is the nominal shear strength and μ is the coefficient of friction; A_v is the area of shear reinforcement across shear plane and f_y is yield stress of reinforcement (≤ 413.7 MPa, ACI 1999); μ is the coefficient of friction. The value of μ is 1.4 for a monolithic concrete connection; 1.0 for a cold joint with 6 mm roughness amplitude; 0.7 for concrete-steel interface; and 0.6 for a cold joint at a smooth concrete interface.

If shear friction is expressed in terms of shear stress, this expression is obtained by dividing V_n by the area of the concrete shear, interface A_c .

$$v_n = \mu \rho_v f_y \quad (2.22)$$

Where, v_n is the nominal shear stress V_n/A_c (≤ 5.5 MPa); ρ_v is the shear friction reinforcement ratio A_v/A_c ; and A_c is the area of concrete interface. Birkeland and Birkeland (1966) proposed this fundamental shear friction equation.

2.3.17 The ACI Commentary (2002)

The researcher acknowledged that changes in strength, size, and spacing of the reinforcement affect the shear strength only in so far as they change the value of the reinforcement parameter f_y for $f_y < 445.4$ MPa. An externally applied compressive stress acting transversely to the shear plane is additive to f_y in calculations of the ultimate shear transfer strength of both initially cracked and uncracked concrete. Conclusively, the shear transfer strength of initially cracked concrete with moderate amounts of reinforcement is developed primarily by frictional resistance

to sliding between the faces of the crack and by dowel action of the reinforcement crossing the crack. When large amounts of reinforcement, or sufficient externally applied compression stresses normal to the shear plane are provided, then the crack in the shear plane locks up and shear transfer strength is developed as in initially uncracked concrete. In their discussion of the report, Walraven et al. (1987) and Mattock (1988) modified the cohesion contribution in the modified shear friction theory to include the effect of concrete strength

$$v_u = 0.467f_c'^{0.545} + 0.8 \rho_v f_y \text{ (psi)} < 0.3 f_c' \quad (2.23)$$

The ACI Commentary ACI (2002) states that in this equation, the first term represents the contribution of friction to shear transfer resistance (0.8 representing the coefficient of friction) and the second term represents the sum of the resistance to shearing of protrusions on the crack faces and the dowel action of the reinforcement. This comment is a little ambiguous. In this equation, it would seem that the contribution of the dowel action should be considered part of the first term, which contains the area of the shear reinforcement and the yield strength of the bar. In addition, the contribution of the friction and aggregate interlock (or shearing of protrusions) are so intertwined or combined that it is difficult to determine the effect of each separately.

Another conclusion made by Mattock (1972) concerning horizontal shear design is that, within their range of applicability, the shear friction provisions of ACI 318-71 that are slightly different from the provisions of ACI 318-02, yield a conservative estimation of the shear transfer strength of reinforced concrete whether a crack exists in the shear plane. Also for higher shear transfer strengths than the upper limit of 5.52 MPa (800 psi) or $0.2 f_c'$, Mattock (1972) suggested that his equation may be used to proportion shear reinforcement.

2.3.16 Kahn and Mitchell (2002)

Kahn and Mitchell performed Shear Friction Tests with High-Strength Concrete and developed the equation given below:

$$v_u = 0.05f_c' + 1.4 \rho_v f_y \leq 0.2 f_c' \text{ (psi)} \quad (2.24)$$

This equation gives a simple, accurate, and conservative prediction of the shear capacity of a cold joint or an uncracked joint for a wide range of concrete strengths. Equation 2.20 has a component in the form of the original equation proposed by Birkeland and Birkeland (1966).

The equation, however, is also in agreement with the rationale of Basler and Witta (1966), Mattock and Hawkins (1972), and Mattock (1976). The equation incorporates a frictional component ($\mu = 1.4$) and a component for bond and asperity shear ($0.05 f_c'$). By taking the component for bond and asperity shear as a percentage of the concrete compressive strength, the equation better predicted results for both normal- and high-strength concretes as compared with a constant value.

Kahn and Mitchell (2002) performed the study of shear Friction Tests with high-strength Concrete in. The push off test was performed with the specimens identically designed to those used by Hofbeck, Ibrahim, and Mattock (1996) and by Anderson (1960) so that the results of the current experiments could be compared directly to previous tests. The 50 push off specimens had concrete design strengths of 27.6, 48.3, 69.0, and 96.5 MPa (4, 7, 10, and 14 ksi). The shear plane was 5 in. wide x 12 in. long (127 x 305 mm); the transverse reinforcing ratio ρ_v varied from 0.37 to 1.47%.

Figure 2.16: Typical loading and failure of precracked specimen (Kahn and Mitchell, 2002)

The surface was left as cast; it was not floated or intentionally roughened. Generally, the cold joint surface was rough, approximately 6 mm (1/4 in). Some specimens had a very high slump mixture in which the as-cast surface was smooth. All other specimens were cast monolithically, so the shear plane was vertical. All cracks were smooth and straight. The compression load was applied through the pins at the top and bottom of the specimens. For a yield stress limited to 413.7 MPa, the actual ultimate shear stress was greater than that predicted in all cases.

2.3.18 Canadian Code (2004)

Clause 11.1.3 in the previous CSA code (Canadian Standards association, 1994) indicates that shear friction may be used to consider interface between element such as webs and flanges, between dissimilar materials and between concrete cast at different times or at existing or potential major cracks along which slip can occur.

According to CSA (2004), a crack shall be assumed to be occur along the shear plane and relative displacement shall be considered to be resisted by cohesion and friction maintained by

the shear friction reinforcement crossing the crack. The factored shear stress resistance of the plane can be computed by:

$$v_s = \lambda \phi_c (c + \mu \sigma) + \phi_s \rho_v f_y \cos \alpha_f \quad (2.25)$$

Here, v_f is the factored shear stress resistance. In this equation, λ is a factor to account for low density concrete; ϕ_c is resistance factor for concrete; ϕ_s is the resistance factor for the reinforcement; σ is the normal stress on plane; ρ_v is the transverse reinforcement ratio; f_y is the specific yield strength of the reinforcement and α_f is the angle between the shear friction reinforcement and the shear plane.

CHAPTER 3

EXPERIMENTAL INVESTIGATION

3.1 Introduction

This investigation was designed and performed by Marzouk and Bridger (2005) to test the capacity of the rehabilitated steel pile. The load history, shape of pile, concrete strength, the confinement reinforcement, and the type of connection between the concrete jacket and the old pile are investigated. The study parameters were evaluated under monotonic loading in compression. The internal steel strain in the new reinforcement cage and the concrete close to the contact surface area was measured and recorded using strain gauges. A summary of the test program and the test results is presented in tables A.1 to A.3 in appendix A.

Twenty-four test specimens were tested in the laboratory. The first set of specimens was focused on testing steel pile without any connection. The new concrete jacket was cast, using normal or high strength concrete with different amounts of circular reinforcements. The second set was designed to evaluate the behavior of rehabilitated piles using mechanical connection between the new concrete jacket and the old steel pile. The third group of specimens was devoted to evaluate the performance of welded connection between the old steel pile and the new concrete jacket.

3.2 Test Specimens

The test specimen consisted of the original steel pile confined with a new circular concrete jacket. The original steel pile can be of wide flange cross section (W shape) or round steel cross section (pipe shape) of hollow structural section (HSS shape). The total length of the test pile

specimen was 1000 mm, confined at the middle with 600 mm long concrete jacket. The test set up details is shown in figure A.1 in appendix A.

The concrete confinement jacket can be normal strength concrete or high strength concrete. The outside diameter of the concrete jacket is 300 mm diameter and the total length of the jacket is 600 mm. The steel reinforcement cage is composed of six 15 mm longitude steel bars equally spaced on circular steel ties of 240 mm diameter. All the horizontal steel reinforcement ties are 10 mm in diameter and equally spaced on the length of the jacket. The total numbers of horizontal ties vary between three, four and six ties per jacket as shown in figure A.2 in appendix A. A typical cross sectional details of the tested specimen including the new concrete jacket is shown in figure 4.2. All the tested specimens were fully instrumented to measure steel and concrete strain gauges. The steel strains were recorded on the longitudinal and circular horizontal steel reinforcement as shown in figure A.3. The concrete strain gauges were positioned on the outside perimeter of the concrete jacket. A typical test specimen used in this investigation before casting is shown in figure A.4.

3.3 Concrete Mix Design

The concrete mixture contained a normal Portland cement, type I in accordance with Canadian Standards Association CSA (type 10), and 6% of silica fume used on the basis of weight for the high strength concrete mix only. The cement content ranged between 450 to 470 Kg/m³ for high strength concrete and concrete and 320 to 350 Kg/m³ for the normal strength concrete mix. The water cement ratio was 0.29 and 0.35 for high and normal strength, respectively. A non-chloride water-reducing agent, retarder and super plasticizer were used in the high strength concrete mix.

The target compressive strengths for the high strength and normal strength concrete specimens was 75 and 30 MPa, respectively.

Local aggregates were used for the concrete mix design. The coarse aggregate was mostly crushed quartzite sandstone with a maximum nominal size of 20 mm. The fine aggregate was identical in composition to the coarse aggregate with a minor percentage of sandstone and shale. The tested specimens were moist cured for 28 days at the concrete laboratory, and then were transferred for the structural laboratory for testing. A typical specimen before testing is shown in Figure A.5.

3.4 Steel Piles and Reinforcement

The steel used to represent the W shapes piles were all of grade Canadian Standard Association CSA-G40.21-M-350W. The wide flange shape specimens were all of W150×14. The circular steel pile shape used in the testing program was a hollow structural steel round section HSS 114×8 confirming to grade CSA-G40.21-M-380W. The reinforcing bars were steel conforming to Canadian Standard Association CSA-G40.20-M94. Two samples of each bar size with diameter of 10 and 15 mm were tested for tensile strength. A hydraulic testing actuator was used to apply load and electrical resistance strain gauges were used to measure the strain up to yield. The recorded yield stress of the steel bars was found between 450-475 MPa. The 10 mm bars were used as radial ties and the 15 mm were used as longitudinal reinforcements for the new concrete jacket.

The second set was designed to evaluate the behavior of rehabilitated piles using the strapped bolted connection between the new concrete jacket and the old steel pile shown in figure A.6 of appendix A. The third set of specimens was devoted to evaluate the performance of welded

connection between the old steel pile and the new concrete jacket shown in figure A.7 of appendix A.

3.5 Test Set-up

A test frame was designed to carry out the lab experimental program. Some additional accessory parts were designed and fitted to facilitate the examination of the rehabilitated pile strength investigation figure A.8 shows the test set-up with a specimen mounted. The vertical loading frame consisted of two main vertical W-shape columns connected by two horizontal cross channels. An electro-hydraulically controlled testing actuator with capacity 700 kN was used to apply monotonic tensile and compression load. A load cell was attached to the actuator to measure the load. The load cell was connected to an internal amplifier via the controller. The output voltage from the controller was feed into the input channel of the data acquisition system. All tests using the lab actuator were run under displacement control. The displacement was measured at the end of the loaded steel bar by the use of the built in Linear Variable Differential Transducer (LVDT) in the actuator and confirmed by the external Linear Potential Differential Transducer (LPDT) at the unloaded end of the bar. The typical displacement rate at 1.51 mm/minute was used for loading the specimens. The slip is the relative displacement between the steel pile and the concrete jacket.

Few of the specimens failure load exceeded the actuator loading capacity of 700 kN, for these specimen two hydraulic jacks and a load cell were used. The loading capacity of the two hydraulic jacks was 900 kN. Four specimens were tested using hydraulic jacks. The main objective of the testing program was to record the maximum load that the rehabilitated pile can resist and the load displacement curve during the duration of the testing. All the specimens were

tested over a three-month testing period at the structural laboratory of Memorial University of Newfoundland.

3.6 Test Results and Discussion

3.6.1 Steel Piles and Concrete Jacket without Connection

3.6.1.1a Round Steel Pipe Cross-Section

The maximum magnitudes of slip and load for each specimen are shown in tables A.1 to A.3 in appendix A. A typical bond load-slip curve for normal strength concrete specimen for the pipe shape using three stirrups (NS-P/3) is shown in figure A.10. In all of the tested specimens, the main cracks developed in the longitudinal direction, while some minor cracks also developed in the transverse direction. The test results indicated a linearity of the bond load-slip behavior in the ascending portion of the curve, especially close to maximum stress. All results confirmed the gradual drop of the load level at the beginning of the descending section of the curve. The slope of the curves in the ascending section was nonlinear for all the specimens. The total bond energy between the steel round and the new concrete can be calculated from the area under the curve.

The number of horizontal ties in the new concrete jacket did not influence the bond load-slip curve of the specimens. However, the use of high strength concrete resulted in slight increase in the maximum bond load and great reduction in the amount of the slip.

The maximum deflection recorded for the normal strength concrete specimens ranged between 8.6-9.2 mm compared to 0.82-0.56 mm for high strength concrete specimens. The maximum bond load for the pipe shape type can be calculated based on the surface shear as detailed in appendix-A. The experimental test results provided a good agreement with the calculated values.

3.6.1.1b Wide Flange Steel Shape

The maximum load recorded for the wide flange specimens was much higher for the wide flange specimens. A typical bond load-slip curve for normal strength concrete specimen for the wide flange shape using three stirrups (NS-W/3) is shown in figure A.10. The maximum-recorded load for the normal concrete specimens with the round shape ranged between 575-473 kN and 625-674 kN for the wide flange specimens. The same trend was observed for high concrete specimens, the maximum load for the round shape ranged between 415-455 kN for the round and 705-711 kN for the wide flange section. The maximum deflection recorded for the normal concrete round shape specimens ranged between 8.6-9.2 mm compared to 4.6-0.17 mm for wide flange shape normal concrete specimens. The amount of slip of all high strength concrete specimens and wide flange shape was less than 1 mm. It is evident that the wide flange shape provided much higher bond load and smaller deflection rather than the round shaped section. The main mechanism for bond between the steel round and concrete jacket is surface friction related to the contact surface area and cohesion coefficient. However, for the wide flange shape the combination of the surface friction and the shear friction is attributed for the strong bond forces. The maximum capacity of the bond of the W shape types can be calculated based on the surface shear and shear friction assumptions in figure A.11 (a) and equations discussed in chapter 4.

3.6.1.2 Concrete Strain

The concrete strain reading recorded on the concrete surface was not very significant; the results did vary from one specimen to the other according to its proximity and the location of the bond

failure surface. The result indicated that when secondary cracks started to open, the concrete strain gauges were damaged.

3.6.1.3 Steel Strain

The steel strain gauge was glued to the steel longitudinal bar at the middle of the specimen and to top steel ring reinforcement. The steel stress vs. strain curve demonstrates the increase of steel strain proportional to the load and the decrease of the strain at the beginning of the descending portion of the load-slip curve. The steel strains for the longitudinal reinforcement of the new concrete jacket of the tested specimens are shown in figure A.9 (b) and A.10 (b) for the specimen without and with connection, respectively. The steel strains for the radial reinforcement of the new concrete jacket of the tested specimens are shown in figure A.9 (c) and A.10 (c) for the specimen without and with connection, respectively.

3.6.2 Steel Pipes and Concrete Jacket with Connection

a) Mechanical Connection

Two 100 mm wide and 4 mm thick steel straps were connected to the top and the bottom of the test specimens as shown in figure A.6. Each strap was connected to the round shape using two 20 mm diameter bolts. A typical bond load-slip curve for normal strength concrete specimen for the round shape using 3 stirrups and mechanical straps (NS-P/3/M) is shown in figure A.10. The maximum load recorded for the round steel pipe shape specimens increased compared to the same section without mechanical connection shown in figure A.10 (a). The increase in capacity can be attributed to the contribution of the shear friction and direct shear resistance of the top and bottom bolted straps. The specimen reached a maximum load of 786 kN and corresponding 4

mm of slip. The maximum bond load for the round shape with mechanical straps can be calculated based on the enhanced shear friction planes shown in figure A.11 (b) and equations in appendix-A.

The same trend was evident for the wide flange shape specimen. The two mechanical top and bottom bolted straps enhanced the resistance capacity due to the direct shear on the straps. The maximum-recorded load was 895 kN and no slip was recorded as shown in figure A.10 (a).

b) Welding Connection

The final set of specimens was conducted to examine the effectiveness of the bolted strap connection verses the welded connection that is regularly used. The welded connection provided a very similar trend to the mechanical connection as shown in figure A.10. A typical bond load-slip curve for normal strength concrete specimen for the pipe shape using 3 stirrups and welding (NS-P/3/WD) is shown in figure A.10. The maximum recorded capacity for the steel round shape with welded shear studs was 675 kN compared to 786 kN for the bolted straps. The bolted straps for the case of round shape pile increased the shear surface area that resulted in an increase in the loading capacity.

Welding underwater is the most expensive task of the pile rehabilitation process. Welding underwater is much more expensive than installing bolted mechanical connection. The mechanical connection for the case of the steel pipe shape provided better results than the welded connection as shown in Tables 2 and 3. However, for the case of the wide flange shape both of the welded and bolted connection yielded the same load capacity (capacity of the actuator or loading jacks) for the rehabilitated pile. It is evident that for the case of steel piles with mechanical or welding connection, that radial steel reinforcement of the new concrete jacket was

mobilized and noticeable increase in the radial strain was recorded as shown in figure A.10 (c). In the mean time the longitudinal steel strains of the reinforcement cage of the steel jacket were also mobilized for both of the welded and mechanically connected concrete jacket. The increase in the bond load was proportional to the increase in steel strains as shown in figure A.10 (b).

3.7 Summary

Test results of the experimental investigation revealed that the rehabilitation of old steel marine piles is possible by encasing the deteriorated pile in a new concrete jacket. The concrete jacket can be made of normal strength concrete or high strength concrete for better durability of the new concrete. The change in the amount of the radial steel reinforcement of the concrete jacket did not influence the bond capacity of between the old steel pile and new concrete jacket. The bond capacity is more related to the shape of the old steel pile. The wide flange steel pile provided more bond resistance than the round shape pile due to the increase of the surface area. In the mean time, the wide flange steel pile provided additional shear friction by creating a new failure surface. The recorded slip of the wide flange pile was much smaller compared to the slip of the round steel pile.

Use of high strength concrete increases the strength of the pile 5% for circular pile and 8% for wide flange pile from the low strength concrete. The influence of confinement increases the circular pile capacity with a normal strength of concrete. However, for the higher strength concrete (wide flange section) the effect of number of ties is not significant. For the unclamped wide flange pile, the capacity of pile is 61% higher for normal strength concrete and 65% higher for high strength concrete compared to unclamped circular pile. However, for piles with bolted

clamp connection, the wide flange increases the pile capacity 17% than the circular pipe. In case of normal strength concrete, the increase of strength due to bolted clamp connection is 73% for circular pile and 31% wide flange pile, compared to unclamped connection. For piles with high strength concrete, the increase of strength due to clamp connection is 65% for circular pile and 21% for wide flange section compared to unclamped pile. The web flange section provides higher strength compare to the circular section as the crack-surface area is greater than that of circular pile.

The suggested mechanical bolted strapped connection provided a good performance compared to the welded connection. Therefore, the expensive process of welding old steel piles under water can be eliminated and a more efficient and much simpler mechanical bolted connection can be installed.

CHAPTER 4

TEST RESULT AND DISCUSSION

4.1 Introduction

Following the results formally tabulated in chapter 3 the next step is to analyze results with respect to how the results compare to those presented in the work of other researchers. This chapter contains the comparison of the interface shear resistance developed from the test specimens to the previous experimental interface shear strength researches by Mattock (1972), Loov (1978), Walraven (1987) and other researchers. The calculated value using Canadian Code (CSA, 1994) is also included in this comparison. The main point of interest in this chapter is to determine the most preferable proposal of the desired equation. To achieve our goal the following topics are discussed:

- Proposed Models
- Comparison of the result in form of tables
- The discussion about the tables in terms of proposed formulas
- Discussion about the selected formula to calculate the shear friction resistance of the tested specimens

4.2 Proposed Models

Eighteen models were discussed in chapter 2, developed for the calculation of shear friction strength by researchers. However; seven expressions were included for the analysis in chapter 4. It has been determined to exclude the formulae from the farther discussions those do not give probative values. The predicted values of shear strength vary from one formula to other due to the different parameters and coefficients used. The proposed formulas are as follows:

Formula # 1 (Birkeland, 1966):

$$v_n = 2.65 \rho f_y \quad (4.1)$$

Formula # 2 (Mattock, 1972):

$$V_n = 0.8 A_{vf} f_y + A_c K_I \quad (4.2)$$

Formula # 3 (Shaikh, 1978):

$$v_n = \mu (\phi \rho_v f_y) \quad (4.5)$$

Formula # 4 (Walraven, 1987):

$$v_n = C_1 (\rho f_y)^{C_2}$$

$$C_1 = 0.878 f_c^{0.406}$$

$$C_2 = 0.167 f_c^{0.303} \quad (4.2)$$

Formula # 5 (ACI, 2002):

$$v_n = 0.467 f_c^{0.545} + 0.8 (\rho_v f_y) < 0.3 f_c \quad (4.3)$$

Formula # 6 (CSA, 1994):

$$v_n = k * \lambda [(0.1 + \rho_v f_y) f_c]^{0.5} \quad (4.3)$$

Formula # 7 (CSA, 2004):

$$v_n = \lambda \phi_c (c + \mu \sigma) + \phi_s \rho_v f_y \cos \alpha \quad (4.7)$$

4.3 Comparison of different types of specimens

Data from previous tests was examined to determine the validity of applying the shear friction model to the test specimen. The comparison is made as the ratio of V_t/V_{sf} . Where, V_t is the value of shear from the test and V_{sf} is the shear predicted by the shear friction model. V_{sf} are reported in this chapter and all the values of V_t are provided in appendix A.

The test specimen consisted of the original steel pile confined with a new circular concrete jacket. The original steel pile can be of wide flange steel section (W shape) or round steel (pipe shape) of hollow structural section (HSS shape). The W shapes piles were all of W150X14 and the Hollow structural steel round sections were HSS 114X8. The total length of the test pile specimen as 1000 mm, confined at the middle with 600 mm long concrete fill. There were two forms of piles:

- Steel piles and concrete Jacket without clamp connection
- Steel piles and concrete Jacket with clamp connection

The detail cross-section for clamped and unclamped section of the test specimens are shown in figure 4.1. The Maximum load recorded for the Wide flange shape and round steel pipe shape specimens with a clamped connection increased compared to the section without mechanical connection shown. The increased capacity can be attributed to the contribution of the shear friction and direct shear resistance of the top and bottom bolted straps. The shear friction models predict higher strength with smaller stirrup spacing since the number of stirrups across the plane is increased, therefore increasing the resisting friction force along the crack. The assumed cracked surfaces for all specimens are presented in the figure A.11 in appendix A.

Figure 4.1: Concrete jacket (a) New concrete jacket without connection to steel pile

(b) New concrete jacket with mechanical connection to steel pile

(Marzouk and Bridger, 2005)

Figure 4.2(a): Steel reinforcement of new concrete jacket (Wide Flange pile)
(Marzouk and Bridger, 2005)

Figure 4.2(b): Steel reinforcement of new concrete jacket (Circular pile)
(Marzouk and Bridger, 2005)

Figure 4.3: Test specimen with a typical bolted connection (Marzouk and Bridger, 2005)

4.3. Discussions on Proposed Models

Various previous researches have been performed to establish the equations for shear resistance of concrete. To develop a proper equation for calculating the interface shear resistance we have chosen the following models:

4.3.1 Formula # 1: (Birkeland, 1966)

Birkeland (1966) obtained a parabolic correlation, which is:

$$v_u = 2.65 \rho f_y \quad (4.1)$$

In this equation, the concrete strength was not considered.

The values of V_t / V_{sf} are given in the chart. It shows that for specimen with wide flange section

obtains a satisfactory result that is greater than the value tested .However, it varies from the desired value. For the specimen with web flange section and a clamp connection we also get high values. For the specimen with circular tube without any clamp connection, low values of V_t / V_{sf} is observed. However, this formula is appropriate for the specimen with circular tube pile and web-flange, confined with 6 ties and with clamp connection.

Formulas	No of ties	W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
Formula # 1	3ties	487.0	474.1	494.7	487.1
Formula # 1	4ties	562.0	547.5	571.3	549.7
Formula # 1	6ties	688.8	670.5	699.7	673.3

Table 4.1.1: Calculated shear friction strength using formula # 1

W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
1.37	1.70	0.76	1.37
1.11	1.58	0.66	1.14
0.98	1.33	0.68	1.00

Table 4.1.2: The results of V_t / V_{sf} for formula # 1

4.3.2 Formula #2: Mattock (1972)

Mattock's (1972) shear friction (or the modified shear friction) theory considered the effect of concrete strength using a constant (K_1). However, Mattock's equation does not include any cohesive term. He suggested the following formula for shear strength of precracked surface

$$V_n = 0.8A_v f_y + A_c K_1 \quad (4.2)$$

Here, V_n is the shear resistance of a precracked surface (kN); A_v is the area of reinforcement crossing the surface (mm^2); f_y is the reinforcement yield strength (MPa); A_c is the area of concrete surface (mm^2) and K_1 is the empirical factor for concrete contribution in shear friction. For the calculation, the value of K_1 is assumed as 2.8 MPa for normal weight concrete. The reasons for deviation from the experimental results are the following:

- Mattock's equation does not include any cohesive term.
- It also does not provide an accurate representation of the effect of concrete strength in the bond strength.
- The theory does not consider the contact stresses across the crack, nor does it treat the contact stresses sustainable by the concrete matrix.

Formulas	No of ties	W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
Formula # 2	3ties	730	765	711.26	745
Formula # 2	4ties	766	801	747	781
Formula # 2	6ties	838	873	819.26	753

Table 4. 2 .1: Calculated shear friction strength using formula # 2

W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
0.87	1.05	0.53	0.61
0.82	0.80	0.50	0.53
0.80	0.90	0.58	0.56

Table 4.2.2: The results of V_f/V_{sf} for formula # 2

For the formula #2, the values of V_f/V_{sf} are given in the table 4.2.2. It demonstrates that for specimen with wide flange section the gives a satisfactory result, which is close and greater than the value, tested. For the specimen with wide flange section and clamp connection lower values of strength were obtained compared to the tested values. For the specimen with circular tube and with a clamp connection also give a satisfactory result. However, for the specimen with circular tube and without any clamp connection give larger value than tested.

4.3.3 Formula # 3: Shaikh (1978)

Shaikh (1978) suggested an equation, which can be expressed as:

$$V_n = \mu(\phi \rho_s f_y) \quad (4.3)$$

Where, ϕ is the constant for shear. The value of ϕ is 0.85 and the value of coefficient of friction (μ) is 25, which is constant. In this equation, only steel strength was considered, but the contribution of concrete was ignored. However, the effect of friction was also considered.

The values of V_t / V_{sf} are given in the table 4.3.2. It shows that for both specimens of web flange section with or without a clamped connection demonstrate higher value of shear strength using the formula # 3 than the tested value.

Formulas	No of ties	W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
Formula #3	3ties	419.3	431	413.0	424.0
Formula #3	4ties	484.1	498	491.4	490.0
Formula #3	6ties	592.9	610	601.9	600.0

Table 4.3.1: Calculated shear friction strength using formula # 3

W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
1.59	1.88	0.91	1.47
1.29	1.74	0.75	1.45
1.14	1.47	0.79	1.31

Table 4.3.2: The results of V_t / V_{sf} for formula # 3

For the specimen with circular tube without a clamp connection also obtain lower values. Nevertheless, for the specimen with circular tube and with clamp connection give larger values than tested, which is not satisfactory.

4.3.4 Formula # 4: Walraven (1987)

Concrete in the contact area may crush before the slip could mobilize enough crack dilation for the yielding of the crossing reinforcement. Walraven (1987) formalized the problem using a

consistent contact model with a probabilistic basis. Experimental and theoretical evidence suggested the influence of concrete strength (but not particularly HSC) on the results. To consider the effect of concrete strength (f_c), Walraven (1987) et al. suggested the following equation for predicting the ultimate stress for shear friction:

$$\begin{aligned}v_n &= C_I (\rho f_y)^{C_2} \\ C_I &= 0.878 f_c^{0.406} \\ C_2 &= 0.167 f_c^{0.303}\end{aligned}\tag{4.4}$$

Here ρ is the ratio of the area of reinforcement across the crack and the area of the cracked surface.

The values of V_t / V_{sf} are given in the chart 4.4.2 with respect to formula #4. It shows that for specimen with wide flange section clamped the gives a satisfactory result, which is close to the value, tested. Unclamped wide flange section variable significance of the formula according to the number of ties.

Formulas	No of ties	W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
Formula # 4	3ties	633	671	727	680
Formula # 4	4ties	710	765	763	798
Formula # 4	6ties	862	953	835	959

Table 4.4.1: Calculated shear friction strength using formula # 4

W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
1.05	1.21	0.52	0.92
0.88	1.22	0.49	0.90
0.78	1.04	0.57	0.82

Table 4.4.2: The results of V_t / V_{sf} for formula # 4

For the specimen with circular tube and without a clamp connection lower values were obtained from the model than the tested value. However, the specimen with circular tube with a clamp connection also gives a satisfactory result.

4.3.5 Formula # 5: Modified Mattock (1988)

Mattock modified the cohesion contribution in the modified shear friction theory to include the effect of concrete strength .Nevertheless, the nonlinearity of the cohesion term detracted from the modified-shear friction theory simplicity.

$$v_n = 0.467f_c^{0.545} + 0.8(\rho_v f_y) < 0.3f_c \quad (4.5)$$

Formulas	No of ties	W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
Formula # 5	3ties	781.7	868.6	726.5	890.0
Formula # 5	4ties	817.7	904.6	762.5	926.0
Formula # 5	6ties	889.7	976.6	834.5	998.0

Table 4.5.1: Calculated shear friction strength using formula # 5

W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
0.85	0.93	0.52	0.70
0.77	0.96	0.49	0.77
0.76	0.92	0.57	0.79

Table 4.5.2: The results of V_t / V_{sf} for formula # 5

Table 4.5.2 presents the comparison of V_t / V_{sf} with respect to formula # 5. It demonstrates that this formula is only suitable for the wide flange and clamped section. However, the values of other sections mentioned on the table predict V_t / V_{sf} values, much lower than 1.

4.3.6 Formula #6: Canadian code (CSA, 1994)

Canadian code 1994 provided an equation for the prediction of the shear friction capacity. In the equation both concrete compressive strength and yield strength of steel was considered. The contribution of empirical coefficient for surface roughness and reinforcement ratio (ρ) was also considered.

$$v = k * \lambda [(0.1 + \rho f_y) f_c']^{0.5} \quad (4.6)$$

Here, k is the reduction in initial contact area inclined at angle θ (here $\theta = 0^\circ$) because of crack deformations δ and w (geometric nonlinearity); λ is the empirical coefficient for surface roughness effect on shear friction capacity; ρ is the reinforcement ratio; f_y is the reinforcement yield stress (MPa) and f_c' is the concrete compressive strength (MPa).

In the actual formula of Canadian Code 1994 for interface shear the value of k is 0.6. This value is appropriate for the wide flange section with a clamp connection. But for the other three kinds of section we have to consider a lower value.

Formulas	No of ties	W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
Formula # 6	3ties	634	655.5	584	709.5
Formula # 6	4ties	707.5	735.5	668	812.8
Formula # 6	6ties	850.4	904.4	798	960.0

Table 4.6.1: Calculated shear friction strength using formula # 6

W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
1.05	1.24	0.64	0.88
0.88	1.18	0.56	0.87
0.79	0.99	0.59	0.82

Table 4.6.2: Value of shear resistance (kN), using formula # 6

The values of V_t/V_{sf} are given in the Table 4.6.2. It shows that for specimen with wide flange section with a clamped connection gives a satisfactory result that is close to the value tested.

Unclamped section with a wide flange pile gives satisfactory result in case of confinement with 3 ties. For the specimen with circular tube with a clamp connection and without clamp connection lower values from the model than the tested values were obtained.

The values are different from the proposed model because in the code the formula is provided for a treated (intentionally roughened) surface. But our purpose is to establish a formula for the surface where there is no provision for surface treatment.

4.3.7 Formula #7: Canadian Code (CSA, 2004)

The values calculated using the latest version of CSA (2004) does not comply with the test results. Because the formula (CSA, 2004) was formed for the surface that is intentionally treated. Therefore, it gives a different value. The factored shear resistance of the shear plane shall be computed from the following formula:

$$v_f = \lambda \phi_c (c + \mu \sigma) + \phi_s \rho_v f_y \cos \alpha \quad (4.7)$$

(Where σ and $\alpha=0$)

The values calculated using CSA 2004 and there comparisons to the tested values are presented in the following tables :

No of ties	Shear force calculated (kN)	Test results (kN)	ρ
3ties	131.1	895	.0013
4ties	168.2	895	.0018
6ties	244.7	895	.0026

Table 4.7.1. Wide flange with clamp connection using Canadian Code (CSA, 2004)

No of ties	Shear force calculated (kN)	Test results (kN)	ρ
3ties	141.0	665	.0014
4ties	179.3	625	.0018
6ties	255.8	674	.0027

Table 4.7.2: Wide flange without clamp connection using Canadian Code (CSA, 2004)

No of ties	Shear force calculated (kN)	Test results (kN)	ρ
3ties	128.4	625	.0013
4ties	167.9	710	.0018
6ties	244.8	786	.0026

Table 4.7.3: Circular tube column with clamp connection using Canadian Code (CSA, 2004)

No of ties	Shear force calculated (kN)	Test results (kN)	ρ
3ties	149.8	375	0.001392
4ties	188.0	375	0.001857
6ties	264.5	473	0.002785

Table 4.7.4: Circular tube without clamping connection using Canadian Code (CSA, 2004)

4.4 Selected Model for the analysis

According to the above discussion we can chose model, which taken from CSA, 1994. The provision to change values of k , the reduction factor helps us to get a satisfactory value. The actual value of k , as given in the CSA (1994) code may not be safe to use for different types of section. The variation of the test result in comparison to the values calculated from the formula lead us to change the k value.

- For wide flange without clamping $k=0.5$
- For wide flange with clamping connection $k=0.6$
- For circular tube without clamping connection $k=0.4$
- For circular tube with clamping connection $k=0.5$

W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
528.3	690	389	591.2
589.6	774.2	445	677.4
708.3	952	532	800.2

Table 4.8.1: Value of shear resistance (kN), using CSA 1994 with changed k values

W-unclamped	W-clamped	HSS-unclamped	HSS -clamped
1.26	1.17	0.96	1.06
1.06	1.12	0.84	1.05
0.95	0.94	0.89	0.98

Table 4.8.2: The results of V_t / V_{sf} for Canadian Code-1994 (formula# 3) with changed k values

The k values are different from the proposed model in the code. For wide flange with clamp connection and circular tube without clamp, we need the highest and lowest value of k respectively. Consequently, this formula gives the value of interface shear resistance, which is very close to the test results. The coefficient of variation is the lowest in comparison to the other formula. Typical variability in normal test results is usually within a coefficient of variation of 10 percent. Hence, the ultimate goal of any behavioral model is to result in a coefficient of variation close to 10 percent when compared with tests. Obviously, the exceptionally good correlation, the wide spectrum of values of involved all the rehabilitation parameters f_c , f_y , ρ , A_c , A_v etc.

4.5 Summary

From the above discussion, it is observed that mattock's equations (1972 and 1988) predicted the highest range of pile capacity. However, CSA (1994) provides predicted most satisfactory results compared to the test data. The interface shear resistance calculated by using the proposed formula of CSA (2004) anticipates very low values. Therefore, this formula was avoided and the

formula of CSA (1994) was recommended. The shear friction factor (k) included in the formula (CSA 1994) can be changed for calculating the capacity of piles according to the section property in order to get the most predictable values.

The investigation revealed that the new concrete jacket should have a mechanical connection to provide adequate bond between the old rehabilitated steel pile and the new concrete jacket. The calculated results, using the equation of shear friction strength from CSA (1994) shows that the bond capacity is increased by 51.5% for the round steel pile shape and by 32% for the wide flange steel pile. For unclamped specimens, the wide flange sections provide 34% higher strength than the circular pipe section. In case of clamp connection, the wide flange pile provides 17% higher strength than circular piles. In specimens with wide flange pile and with circular pile, the compressive strength of concrete is assumed to be around 35 MPa.

CHAPTER 5

CONCLUSION & RECOMMENDATIONS

In this investigation, the attention was centralized on developing a formula to determine the capacity of rehabilitated piles using the shear friction equation principles. Marine piles are subjected to corrosion due to the adverse effect of salted water. The deteriorated piles can be rehabilitated by encasing steel piles with a reinforced concrete. To ensure a proper bond between concrete and old steel pile some shear connectors need to be welded to the structure. However, welding under water is expensive and a difficult task. Moreover, any kind of surface preparation to increase the steel-concrete bond is not possible in this environment. Therefore, an alternative strengthening procedure can be applied by anchoring the concrete encasement to the old pile with mechanical bolted straps.

The installment of the concrete encasement around corroded steel pile will provide protection against corrosion. The concrete encasement decreases the diffusion of chemically deleterious substances, it provides a reduction in plastic cracking and it can resist rusting of steel piles. Concrete can serve to protect the steel as the highly alkaline nature of concrete that provides an environment that is beneficial to the embedded reinforcement. If the concrete is of a good quality, well compacted and with minimal cracking, it is difficult that a corrosion can take place. In many cases, reinforced concrete can provide better support against stresses experienced by the structure. As the bolted joint is surrounded by concrete jacket, confidence can be placed that no slip will occur in connection.

The bolted anchorage increases the strength of the pile by raising the structural stiffness with the composite action of the pile, clamp connection and the reinforcing steels. It increases the load

capacity and the ductility in comparison to the corresponding unanchored specimen. The bolted anchorage plate has its effects on the failure mode, as it is associated with a clamping force. As the plates are bolted to each other on increase in the confinement force is created to facilitate the rehabilitation work.

The investigation was based on a previous laboratory test evaluation of shear friction strength of the concrete covered rehabilitated pile samples. The pile-loads, obtained from the test results were compared with the calculated shear strength from equations, developed by the previous researchers. Among the formulas developed by the researchers, seven were chosen for the analysis proposed by Birkeland (1966), Mattock (1972 and 1988), Shaik (1978), Walraven (1987) and Canadian Concrete Standard Association (CSA, 1994 and 2004). It is observed that Mattock's (1988) equation anticipates the highest range of the value of pile capacity. However, CSA (1994) formula predicted the most satisfactory results compared to the test data. Moreover, the provision of changing the k value (empirical factor for concrete contribution in shear friction) for different types of samples made that expression more reliable. The main component of this equation is the area of reinforcement crossing the surface, the steel reinforcement yield strength and the area of concrete failure surface. The value of k is used to include the contribution of concrete for shear friction that varied, with the concrete strength. This formula is convenient, since the concrete strength can also be evaluated along with the reinforcing strength and the contribution of steel anchorage.

The use of anchorage plate also influences the shear strength as it provides different amount of cracked surface area. The area of cracked surface has to be calculated for each case; such as clamped or unclamped or wide flange steel pile or circular steel pile. In general, the wide flange

provides a greater surface area of shear plane; it possesses the greater pile capacity over circular pile.

The use of high strength concrete increases the strength of the rehabilitated pile. The increase of confinement increases the circular pile capacity with a normal strength of concrete. However, for wide flange pile with high strength concrete, the influence of number of ties is insignificant. The amount of increase of strength due to bolted clamp connection is higher in normal strength concrete and lower in high strength concrete. The wide flange section provides higher strength compared to the circular section as the failure surface area is greater than that of circular pile.

Repair and rehabilitation of piles for marine decks are important in order to restore structural and functional integrity of the facility providing. Epoxy coated steel can be used in the rebar and the steel anchorage system to protect surface against corrosive environment and to prevent the penetration of moisture and chloride salts. The high strength concrete increases the water tightness of the structure.

The bolted anchor strap plates are advantageous for improving the capacity of the rehabilitated pile and for avoiding early failure. Anchored piles are associated with lower maximum deflection and lower strain through the section due to increase in flexural rigidity of the section.

In case of normal strength concrete, the increase of strength due to bolted clamp connection is estimated by 73% for circular pile and 31% for the wide flange pile, compared to unclamped connection. For piles with high strength concrete, the increase of strength due to clamp connection is 65% for circular pile and 21% for wide flange section compared to unclamped pile. The marine pile can be efficiently rehabilitated by installing a concrete jacket using shear friction and bolted connection to avoid the expense of welding under water. The recommended formula can predict the capacity of the rehabilitated pile efficiently.

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APPENDIX A

Specimen Number	Pile Shape	Concrete Strength MPa	Type of Connection	No. of circular Ties	Max load KN	Max Deflection mm
NS-P/3	HSS 114×8	28	NONE	3	375	8.6
NS-P/4	HSS 114×8	28		4	375	9.5
NS-P/6	HSS 114×8	28		6	473	9.2
HS-P/3	HSS 114×8	46		3	455	.72
HS-P/4	HSS 114×8	51		4	415	.56
HS-P/6	HSS 114×8	51		6	418	.82
NS-W/3	W 150×14	31	NONE	3	665	.17
NS-W/4	W 150×14	31		4	625	.35
NS-W/6	W 150×14	31		6	674	4.6
HS-W/3	W 150×14	76		3	710	.3
HS-W/4	W 150×14	76		4	705	.54
HS-W/6	W 150×14	76		6	711	.85

Table A 3.1: Test results of Steel pile with concrete jacket and no connection

(Marzouk and Bridger, 2005)

Specimen Number	Pile Shape	Concrete Strength MPa	Type of Connection	No. of circular Ties	Max load KN	Max Deflection mm
NS-P/3	HSS 114×8	39	Mech Bolted	3	625	.35
NS-P/4	HSS 114×8	39		4	710	3.7
NS-P/6	HSS 114×8	39		6	786	0
NS-W/3	W 150×14	35	Mech Bolted	3	810	0
NS-W/4	W 150×14	35		4	865	0
NS-W/6	W 150×14	35		6	895	0

Table A.3.2: Steel piles with Concrete Jacket and Mechanical connection

(Marzouk and Bridger, 2005)

Specimen Number	Pile Shape	Concrete Strength MPa	Type of Connection	No. of circular Ties	Max load KN	Max Deflection mm
NS-P/3	HSS 114×8	39	Welding	3	692	8.45
NS-P/4	HSS 114×8	39		4	670	4.6
NS-P/6	HSS 114×8	39		6	673	7
NS-W/3	W 150×14	35	Welding	3	895	0
NS-W/4	W 150×14	35		4	895	0
NS-W/6	W 150×14	35		6	895	0

Table A.3.3 : Steel piles with Concrete Jacket and Welded connection

(Marzouk and Bridger, 2005)

FigureA.1: Test set-up (Marzouk and Bridger, 2005)

Figure A.2: Steel cages with different ties

(Marzouk and Bridger, 2005)

Figure A.3: Steel strain gauges attached to reinforcement cage
(Marzouk and Bridger, 2005)

Figure A.4: A Typical specimen before casting the concrete jacket
(Marzouk and Bridger, 2005)

Figure A.5: A typical specimen after casting
(Marzouk and Bridger, 2005)

Figure A.6: A Typical mechanical connection
(Marzouk and Bridger, 2005)

Figure A.7: A Typical welding connection
(Marzouk and Bridger, 2005)

Figure A.8: A Typical specimen during testing
(Marzouk and Bridger, 2005)

Figure A.9: Responses of steel piles and concrete jacket without connection (a) Load-deflection, (b) Load-longitudinal strain, (c) Load-radial strain (Marzouk and Bridger, 2005)

(a)

(b)

(c)

Figure A.10: Responses of steel piles and concrete jacket with connection

(a) Load-deflection, (b) Load-longitudinal strain, (c) Load-radial strain

(Marzouk and Bridger, 2005)

Surface friction

Surface friction and shear friction

(a)

Surface friction, shear friction and strap friction

Surface friction, shear friction and strap friction

(b)

Figure A.11: Concrete jacket assumed cracking (a) New concrete jacket without connection to steel pile (b) New concrete jacket with mechanical connection to steel pile (Marzouk and Bridger, 2005)

APPENDIX B

Mattock, 1972:

$$V_n = 0.8A_v f_y + A_c K_I$$

Wide Flange Section without Clamping Connection:

Calculation of area of shear plane in concrete A_c :

Length of the cracked surface = $(150+60+60+100)$ mm = 370 mm (See figure A.11.a)

Depth of the sample = 600 mm

$$A_c = (370 \times 600) \text{ mm}^2 = 222000 \text{ mm}^2$$

No of ties	Area of steel $A_v(\text{mm}^2)$	Area ratio of steel to concrete (A_v/A_c)
3	$3 \times 100 = 300$	$300/222000 = 0.00135135$
4	$4 \times 100 = 400$	$400/222000 = 0.0018018$
6	$6 \times 100 = 600$	$600/222000 = 0.0027027$

Table B.1 .a: Calculation of area ratio

$K_I = 2.78 \text{ MPa}$ & $f_y = 450 \text{ MPa}$ (Assumed)

Calculation of Shear Force:

No of ties	Interface Shear Force (V_n) kN
3	$0.8 \times 300 \times 450 + 222000 \times 2.8 = 729$
4	$0.8 \times 300 \times 450 + 222000 \times 2.8 = 765.6$
6	$0.8 \times 300 \times 450 + 222000 \times 2.8 = 838$

Table B.1.b: Calculation of Shear Force

Mattock 1972:

$$V_n = 0.8A_v f_y + A_c K_I$$

Wide Flange Section with Clamping Connection:

Calculation of area of shear plane in concrete A_c :

Length of the cracked surface = $[96+100+(75+60)+60]$ mm = 385 mm (See figure A.11.b)

Depth of the Sample = 600 mm

$$A_c = (385 \times 600) \text{ mm}^2 = 234600 \text{ mm}^2$$

No of ties	Area of steel $A_v(\text{mm}^2)$	Area ratio of steel to concrete (A_v/A_c)
3	$3 \times 100 = 300$	$300/234600 = 0.001279$
4	$4 \times 100 = 400$	$400/234600 = 0.001705$
6	$6 \times 100 = 600$	$600/234600 = 0.002558$

Table B.2.a: Calculation of area ratio

$K_I = 2.78 \text{ MPa}$ & $f_y = 450 \text{ MPa}$ (Assumed)

Calculation of Shear Force:

No of ties	Interface Shear Force (V_n) kN
3	$0.8 \times 300 \times 450 + 0.002558 \times 2.8 = 729$
4	$0.8 \times 300 \times 450 + 0.002558 \times 2.8 = 765.6$
6	$0.8 \times 300 \times 450 + 0.002558 \times 2.8 = 838$

Table B.2.b: Calculation of Shear Force

Mattock 1972:

$$V_n = 0.8A_v f_y + A_c K_I$$

Circular pipe without Clamping Connection:

Calculation of area of shear plane in concrete A_c :

Length of the Cracked surface = $(150+60+60+100)$ mm = 370 mm (See figure A.11.a)

Depth of the Sample = 600 mm

$$A_c = 215450 \text{ mm}^2$$

No of ties	Area of steel $A_v(\text{mm}^2)$	Area ratio of steel to concrete (A_v/A_c)
3	$3 \times 100 = 300$	$300/215450 = 0.00135135$
4	$4 \times 100 = 400$	$400/215450 = 0.00185658$
6	$6 \times 100 = 600$	$600/215450 = 0.00278487$

Table B.3.a: Calculation of area ratio

$K_I = 2.78 \text{ MPa}$ & $f_y = 450 \text{ MPa}$ (Assumed)

Calculation of Shear Force:

No of ties	Interface Shear Force (V_n) kN
3	$0.8 \times 300 \times 450 + 215450 \times 2.8 = 711.26$
4	$0.8 \times 300 \times 450 + 215450 \times 2.8 = 747$
6	$0.8 \times 300 \times 450 + 215450 \times 2.8 = 819.26$

Table B.3.b: Calculation of Shear Force

Mattock, 1972:

$$V_n = 0.8A_v f_y + A_c K_I$$

Circular Section with Clamping Connection:

Calculation of area of shear plane in concrete A_c :

Length of the cracked surface = $(114+30+85+63+57+30)$ mm = 385 mm (See figure A.11.b)

Depth of the sample = 600 mm

$$A_c = (385 \times 600) \text{ mm}^2 = 234600 \text{ mm}^2$$

No of ties	Area of steel $A_v(\text{mm}^2)$	Area ratio of steel to concrete (A_v/A_c)
3	$3 \times 100 = 300$	$300/227400 = 0.001319$
4	$4 \times 100 = 400$	$400/227400 = 0.001759$
6	$6 \times 100 = 600$	$600/227400 = 0.002639$

Table B.4.a: Calculation of area ratio

$K_I = 2.78 \text{ MPa}$ & $f_y = 450 \text{ MPa}$ (Assumed)

Calculation of Shear Force:

No of ties	Interface Shear Force (V_n) kN
3	$0.8 \times 300 \times 450 + 0.002558 \times 2.8 = 890.0555$
4	$0.8 \times 300 \times 450 + 0.002558 \times 2.8 = 926.0555$
6	$0.8 \times 300 \times 450 + 0.002558 \times 2.8 = 998.0555$

Table B.4.b: Calculation of Shear Force

CSA, 1994:

$$v = k \cdot \lambda \cdot ((0.1 + \rho f_y) f_c)^{0.5}$$

Wide Flange Section without Clamping Connection:

Calculation of area of shear plane in concrete A_c :

Length of the cracked surface = $(150 + 60 + 60 + 100)$ mm = 370 mm (See figure A.11.a)

Depth of the sample = 600 mm

$$A_c = (370 \times 600) \text{ mm}^2 = 222000 \text{ mm}^2$$

No of ties	Area of steel $A_v(\text{mm}^2)$	Area ratio of steel to concrete (A_v/A_c)
3	$3 \times 100 = 300$	$300/222000 = 0.00135135$
4	$4 \times 100 = 400$	$400/222000 = 0.0018018$
6	$6 \times 100 = 600$	$600/222000 = 0.0027027$

Table B.5.a: Calculation of area ratio

$k = 0.6$, $f_y = 450$ MPa, $f_c = 31$ MPa (Assumed)

Calculation of Shear Force:

No of ties	Interface Shear Strength (v_n) KN/mm ²	Interface Shear Force ($V_n = v_n \times A_c$) KN
3	$0.6 \times 1 \times ((0.1 + 0.0013 \times 450) \times 31)^{0.5} = 2.9$	634
4	$0.6 \times 1 \times ((0.1 + 0.0018 \times 450) \times 31)^{0.5} = 3.2$	707.5
6	$0.6 \times 1 \times ((0.1 + 0.0027 \times 450) \times 31)^{0.5} = 3.8$	992.0

Table B.5.b: Calculation of Shear Force

CSA, 1994:

$$v = k \cdot \lambda \cdot ((0.1 + \rho f_y) f_c)^{0.5}$$

Wide Flange Section with Clamping Connection:

Calculation of area of shear plane in concrete A_c :

Length of the cracked surface = $[96 + 100 + (75 + 60) + 60]$ mm = 385 mm (See figure A.11.b)

Depth of the sample = 600 mm

$$A_c = (385 \times 600) \text{ mm}^2 = 234600 \text{ mm}^2$$

No of ties	Area of steel A_v (mm ²)	Area ratio of steel to concrete (A_v/A_c)
3	$3 \times 100 = 300$	$300/234600 = 0.001279$
4	$4 \times 100 = 400$	$400/234600 = 0.001705$
6	$6 \times 100 = 600$	$600/234600 = 0.002558$

Table B.6.a: Calculation of area ratio

$k = 0.6$, $f_y = 450$ MPa, $f_c = 35$ MPa (Assumed)

Calculation of Shear Force:

No of ties	Interface Shear Strength (v_n) kN/mm ²	Interface Shear Force ($V_n = v_n \times A_c$) kN
3	$0.6 \times 1 \times ((0.1 + 0.0013 \times 450) \times 35)^{0.5} = 2.9$	690
4	$0.6 \times 1 \times ((0.1 + 0.0017 \times 450) \times 35)^{0.5} = 3.3$	774.2
6	$0.6 \times 1 \times ((0.1 + 0.0026 \times 450) \times 36)^{0.5} = 4.0$	952

Table B.6.b: Calculation of Shear Force

CSA, 1994:

$$v = k \lambda ((0.1 + \rho f_y) f_c)^{0.5}$$

Circular pipe without Clamping Connection:

Calculation of area of shear plane in concrete A_c :

Length of the cracked surface = (150+60+60+100) mm = 370 mm (See figure A.11.a)

Depth of the sample = 600 mm

$$A_c = 215450 \text{ mm}^2$$

No of ties	Area of steel $A_v(\text{mm}^2)$	Area ratio of steel to concrete (A_v/A_c)
3	$3 \times 100 = 300$	$300/215450 = 0.00135135$
4	$4 \times 100 = 400$	$400/215450 = 0.00185658$
6	$6 \times 100 = 600$	$600/215450 = 0.00278487$

Table B.7.a: Calculation of area ratio

$k = 0.6$, $f_y = 450 \text{ MPa}$, $f_c = 28 \text{ MPa}$ (Assumed)

Calculation of Shear Force:

No of ties	Interface Shear Strength (v_n) kN/mm^2	Interface Shear Force ($V_n = v_n \times A_c$) kN
3	$0.6 \times 1 \times ((0.1 + 0.0014 \times 450) \times 28)^{0.5} = 2.7$	584
4	$0.6 \times 1 \times ((0.1 + 0.0018 \times 450) \times 28)^{0.5} = 3.1$	668
6	$0.6 \times 1 \times ((0.1 + 0.0028 \times 450) \times 28)^{0.5} = 3.7$	798

Table B.7.b: Calculation of Shear Force

CSA, 1994:

$$v = k \lambda ((0.1 + \rho f_y) f_c)^{0.5}$$

Circular pipe with Clamping Connection:

Calculation of area of shear plane in concrete A_c :

Length of the cracked surface = $(114 + 30 + 85 + 63 + 57 + 30)$ mm = 385 mm (See figure A.11.a)

Depth of the sample = 600 mm

$$A_c = (385 \times 600) \text{ mm}^2 = 234600 \text{ mm}^2$$

No of ties	Area of steel $A_v(\text{mm}^2)$	Area ratio of steel to concrete (A_v/A_c)
3	$3 \times 100 = 300$	$300/227400 = 0.001319$
4	$4 \times 100 = 400$	$400/227400 = 0.001759$
6	$6 \times 100 = 600$	$600/227400 = 0.002639$

Table B.8.a: Calculation of area ratio

$k = 0.6$, $f_y = 450$ MPa, $f_c = 39$ MPa (Assumed)

Calculation of Shear Force:

No of ties	Interface Shear Strength (v_n) kN/mm^2	Interface Shear Force ($V_n = v_n \times A_c$) kN
3	$0.6 \times 1 \times ((0.1 + 0.00132 \times 450) \times 39)^{0.5} = 3.1$	709.488
4	$0.6 \times 1 \times ((0.1 + 0.0018 \times 450) \times 39)^{0.5} = 3.6$	812.8206
6	$0.6 \times 1 \times ((0.1 + 0.0026 \times 450) \times 39)^{0.5} = 4.2$	960.2313

Table B.8.b: Calculation of Shear Force