DESIGN OF DOWELS FOR SHEAR TRANSFER AT THE INTERFACE BETWEEN CONCRETE CAST AT DIFFERENT TIMES: A CASE STUDY

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Degree of Master of Engineering in Structural Engineering Design

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A Thesis submitted in partial fulfillment of the requirement for the degree of

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DECLARATION

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ABSTRACT

Enlargement of original cross-sections or replacement of defective concrete layers with new concrete are usual situations in strengthening operations of reinforced concrete structures. In these situations, the shear strength between concrete cast at different times is crucial for the monolithic behavior of the strengthened members. Most design standards for concrete structures present design procedure for estimating the shear resistance between concrete layers based on the shear friction theory.

The study includes three-dimensional and two-dimensional finite element model (FEM) analysis for calculation of shear stresses and comparison of three different code approaches, i.e. BS8110, ACI 318 and EN 1992, for determination of design shear resistance at an interface between concrete cast at different ages of a pile cap supported on precast concrete piles.

Based on the results of the analysis carried out, it can be stated that complicated three dimensional finite element model analysis is not always essential for analysis of structures, which are having complex geometrical shapes. It is possible to transform three-dimensional problems to a simplified two-dimensional problem based on the level of accuracy required.

For the selected surface characteristics and r/f percentage, the estimated design shear resistance based on recommendations of EN-1992-1-1-2004 was found be lower than the corresponding estimated value based on ACI 318M-11 recommendations. It was further observed that BS 8110-1-1997 recommendations gives the highest value for the design shear resistance independent of r/f percentage provided.

EN-1992-1-1-2004 can be used to compare contribution of concrete interface roughness and interface reinforcement on design shear resistance without any limitation of design shear stress as specified in ACI 318M-11. Furthermore, the EN-1992-1-1-2004 recommends a conservative value for design shear resistance compared to other two standards.

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LIST OF ABBREVIATIONS

EN-1992-1-1-2004

v_{Edi}	=	Design value of shear stress
v_{Rdi}	=	Design shear resistance
β	=	Ratio of the longitudinal forces
V_{Ed}	=	Design value of applied shear force
z	=	Lever arm of composite section
b_i	=	Width of the interface
c	=	Factor related to adhesion
μ	=	Coefficient of friction
ρ	=	Ratio (A_s/A_i)
f_{ctd}	=	Design value of concrete tensile strength
σ_n	=	Stress per unit area caused by external normal force
f_{yd}	=	Design yield strength of reinforcement
α	=	Angle
v	=	Strength reduction factor
f_{cd}	=	Design vale of concrete compressive strength
f_{ctk}		Characteristic axial tensile strength of concrete
γ_c	=	Partial factor for concrete
A_s	=	Area of the reinforcement crossing the interface
A_i	=	Aria of the joint
f_{ck}	=	Characteristic compressive cylinder strength of concrete at 28 days
α_{cc}	=	Coefficient
α_{ct}	=	Coefficient
b_w	=	Breadth of the web of the member
$\rho_{w.min}$	=	Nominal shear reinforcement ratio
A_{sw}	=	Area of nominal shear reinforcement

ACI 318M-2011

V_u	=	Factored shear force
ϕ	=	Strength reduction factor
V_{nh}	=	Nominal horizontal shear strength
b_v	=	Width of the cross section
d	=	Distance from extreme compression fiber to centroid of of longitudinal tension reinforcement
A_{vf}	=	Area of shear friction reinforcement
f_y	=	Yield strength of reinforcement
μ	=	Coefficient of friction
f'_c	=	Specified compressive strength of concrete
A_c	=	Area of concrete section resisting shear transfer
λ	=	Modification factor
s	=	Spacing of shear links
ρ_v	=	Ratio of tie reinforcement area to contact surface area
f_{yt}	=	Yield strength of transverse reinforcement
b_w	=	Web width, wall thickness
v_u	=	Design shear stress

BS 8110-1-1997

V_h	=	Horizontal shear force
C	=	Compressive force
T	=	Tensile force
v_h	=	Horizontal shear stress
l	=	Length between points of maximum moment and zero moment
A	=	Cross sectional area of nominal links
b_v	=	Width of the contact surface

$S_{l.max}$ = Maximum longitudinal spacing between links

$S_{t.max}$ = Maximum transverse spacing between links

f_{yk} = Characteristic yield strength of reinforcement

S_v = Spacing of shear links

h_f = Minimum thickness of the in-situ concrete

A_h = Total area of shear reinforcement

M = Bending Moment

b = Width of the section

d = Effective width of the tension reinforcement

f_{cu} = Characteristic cube strength of concrete

FEM = Finite Element Model
2D = Two Dimensional
3D = Three Dimensional
2D-L = Two Dimensional-Longitudinal
2D-T = Two Dimensional-Transverse
ULS = Ultimate Limit State
RC = Reinforced Concrete
SFD = Shear Force Diagram
BMD = Bending Moment Diagram

CHAPTER 1

INTRODUCTION

CHAPTER 1 – INTRODUCTION

1.1. General

Most of the reinforced concrete structural elements like building slabs, bridge decks, columns, beams are generally strengthened by adding a concrete overlay. Replacing of defective concrete and placing of new concrete overlay on top of reinforced concrete elements are also used in strengthening operations. In these situations, interfaces between old and new concrete play an important role in achieving monolithic behavior of the resulting composite reinforced concrete members.

Knowledge on modeling of cracked concrete related to shear forces has not yet fully understood, as the shear loading leads to complicated physical mechanisms, such as multy-axial stress conditions, interlocking of aggregates, dowel action, and reduced bond resistance of embedded bars.

The method of design across the shear strength at the interfaces where old concrete meet new concrete has changed over the years. Nowadays the majority of design codes have adopted expression based on shear friction theory. In this particular case study, it is mainly focused on to ACI 318M-2011, BS 8110-1-1997and EN-1992-1-2004 approaches for estimating interface shear resistance of concrete cast at different ages with dowels passing through the interface.

1.2. Background of the problem

Excessive delay in transportation and placing of concrete may influence to initial setting of concrete and cause it to become unusable. In order to avoid setting of concrete, generally a retarder, i.e. retarding admixture, is used which prolong the setting of concrete. While giving the permission to use of retarder, it should be ensured the suitability and dose of retarder after conducting necessary trials. (It may be noted that generally retarding effect of retarder is smaller at higher temperature and sometimes few retarders seem to be in-effective at extremely high temperature.)

Overdosing of admixture or incompatible admixture can couse excessive delay in hardening of concrete. Concrete with excessive delay in hardening can resulted in defective hardened concrete with reduced load carrying capacity with respect to the assumed value in the design stage.

One of the concrete pile cap (20 m x 2.2 m x 0.8 m) of the Bridge B20A (Figure 1) of Southern Expressway extension project was subjected to excessive delay in hardening of fresh concrete and that has resulted more porous top layer concrete which was identified by observing cores extracted from the pile cap.(Figure 3,4) Cores clearly indicate the conditions of the concrete especially around the reinforcement bars. There are voids in the upper layer of the pile cap especially below the reinforcement and coarse aggregate as a result of arching action of coarse aggregate due to plastic settlement of fresh concrete. The voids underneath the reinforcing bars can impair the anchorage of reinforcement as well as the durability.

As remedial measures, considering the durability of the structure, it is proposed to replace the defective concrete layer with new concrete layer over an existing concrete to restore the pile-cap to withstand the originally designed loads.

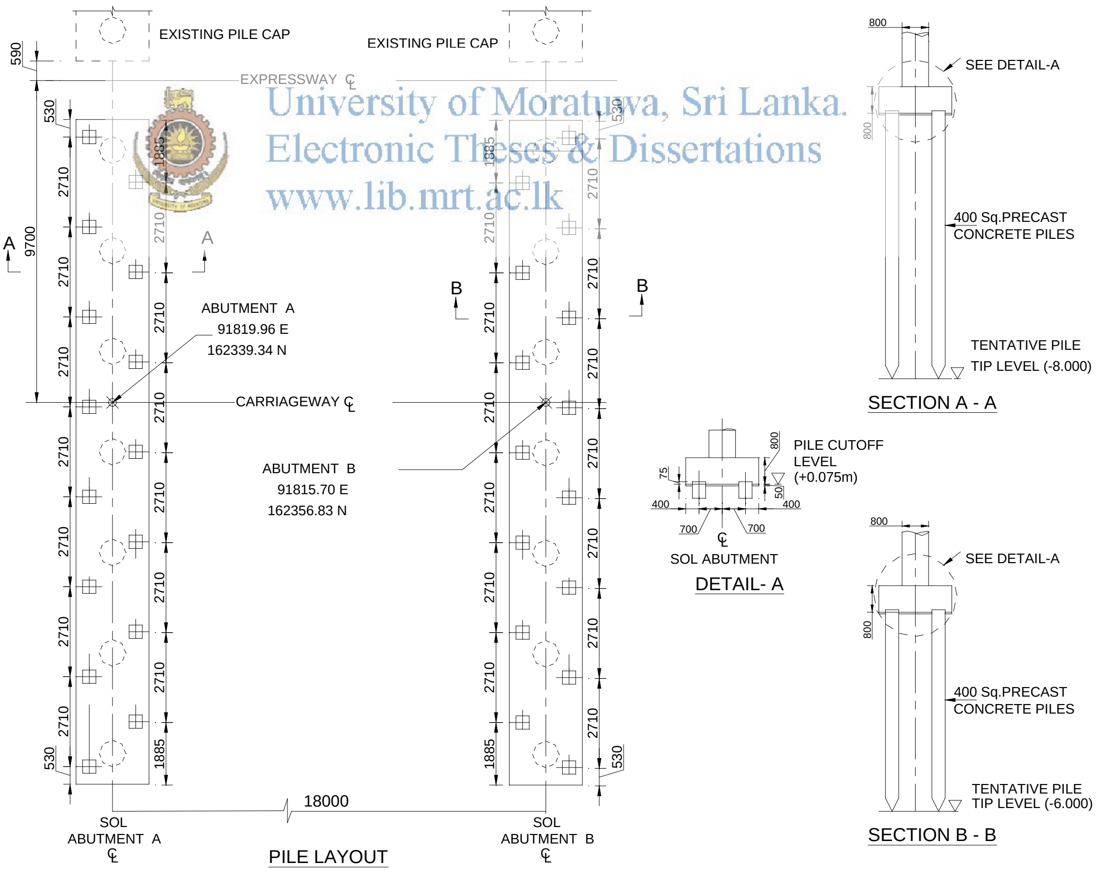


Figure 1: Layout of the piles and pile caps of Bridge B20A

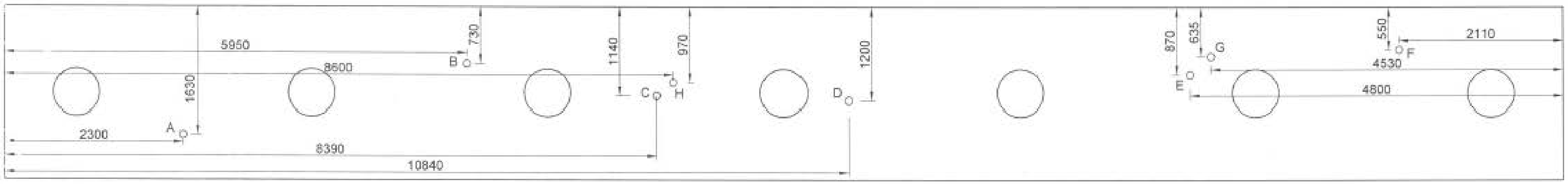


Figure 2: Core sample locations

Figure 3: Defects in core samples A, B, & C

Figure 4: Defects in core samples D, F, G, H &J

1.3. Objective of the study

The objective includes the evaluation of the interface shear stress by using finite element model analysis and the evaluation of shear resistance provided by various interface characteristics of reinforced concrete, where new concrete cast against an existing concrete surface. In addition to that, the study includes examining the influence of reinforcement crossing the interface between the existing concrete member and the concrete overlay by considering its shear transfer phenomena. (The reason is the two principle modes of shear forces to be transmitted across a crack are through the interaction between rough surfaces of the crack and through reinforcement crossing the interface. The dowel action contributes to the overall shear transfer. The tensile stiffness of the reinforcement normal to the crack plane also influences the shear stiffness of interface. At the same time, the shear displacement at the crack produces localized flexure in reinforcement in the reinforcement inside concrete, giving rise to combine flexural and shear stresses.)

1.4. Scope of the study

Scope of the study includes evaluation of shear stresses at the interface between old substrate concrete and an new overlay concrete of a pile cap by analyzing the three dimensional and simplified 2-Dimensional Finite Element Model (FEM) of the pile cap. The results obtained by two models are compared with each other to verify the necessity to perform complicated finite element model analysis.

The design and examination of required interface shear reinforcement are performed based on a shear-friction hypothesis by using three international standards, EN-1992-1-1-2004, ACI 318M-2011, and BS 8110-1-1997.

Finally, the results comparison is exercised for design interface shear resistance calculated based on the principles stipulated in the three design standards for various interface preparation techniques. In addition to that, detailing of interface shear dowels are performed with recommendations.

1.5. Methodology

Methodology was planned to do the study in a sequential way of completion in step-by-step arrangement as in the flow chart. (Figure 5)

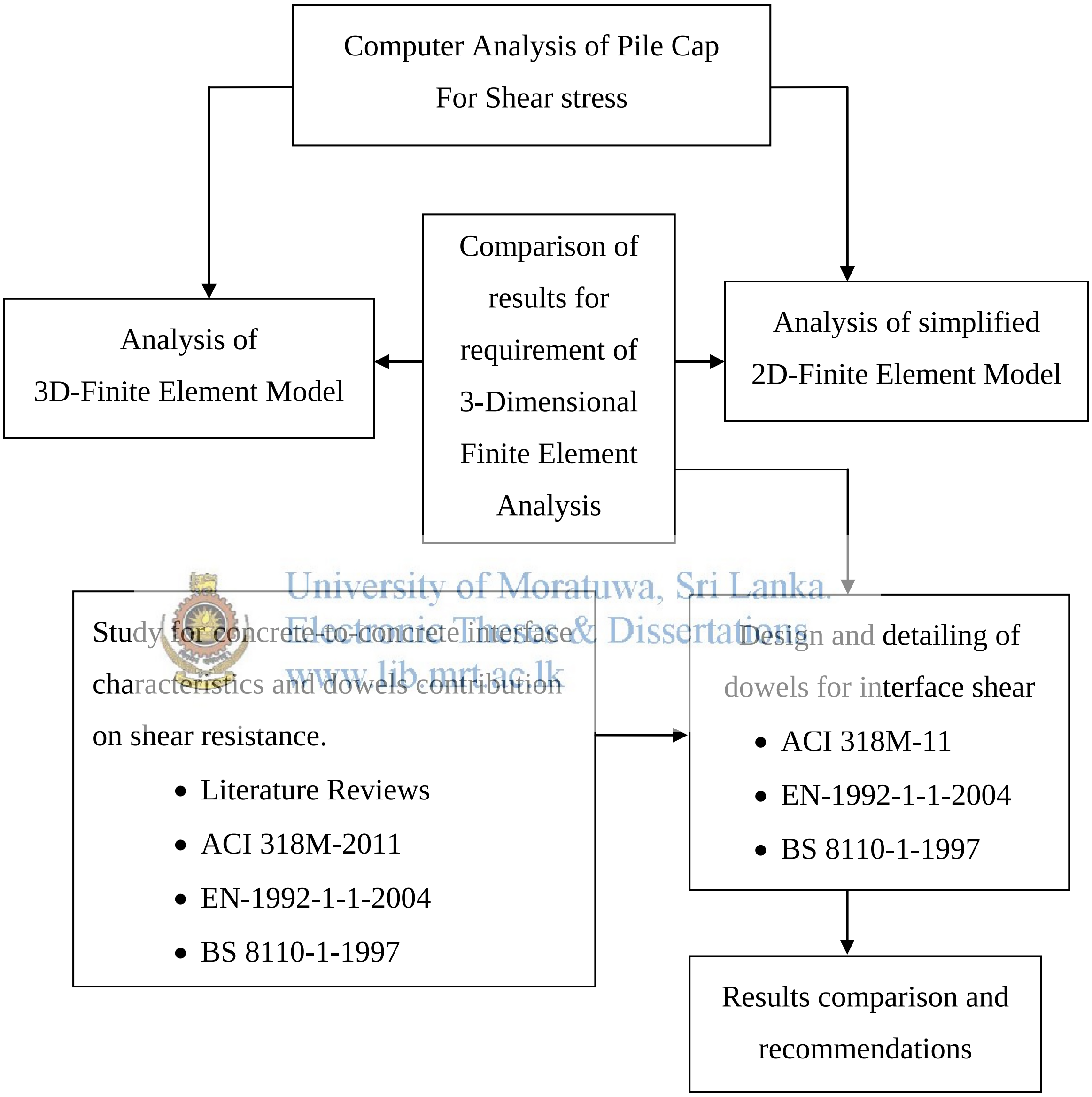


Figure 5: Flow chart for methodology

CHAPTER 2

LITERATURE REVIEW

CHAPTER 2 – LITERATURE REVIEW

2.1. Factors affecting interface shear resistance

The topic of shear transfer across reinforced concrete interfaces is very popular and has been the subject of several studies. The various components of this shear transfer mechanism have been well identified. Many researchers have given valuable contributions. The components, which have been identified as the contributing factors to the interface shear resistance are discussed in the following sections.

2.1.1. Effect of reinforcement crossing the interface

The previous research study performed by Robert et al.[1] based on the shear transfer across the existing and new concrete interfaces have concluded that generally, when the amount of reinforcement crossing the interface are increased, the shear capacities are also increased at large slip levels. The study further mentioned that the deeper embedment provides better development of the reinforcement and subsequently higher shear capacities at large slip levels. Also, the reinforcement detailing in the new concrete wall and the base block concrete member has not been contributed significantly on shear capacity.

Paulay et al. [2] have performed a study on the horizontal construction joints in the cast-in- placed reinforced concrete. The researchers imply that the shear and kinking as shown in figure 6 are the principle mechanisms of dowel action of the reinforcement crossing the interface and superior performance of the smaller bars probably results from the smaller development lengths required on either side of the plane of the joint. In addition, study has mentioned that the large displacements associated with significant dowel strength are likely to contribute to another form of distress in structural members subjected to shear. Therefore, the authors finally concluded that for design purposes, the contribution of the dowel action of the reinforcement should be ignored, as the significant dowel forces generates at the excessive slip along the joint. Also, for low steel percentages, failure consisted of yielding of interface reinforcement in the range of 0.3 percent. For higher steel percentages, failure consisted of crushing of concrete at the shear plane.

Mattock et al. [3] carried out an experimental study on shear transfer in reinforced concrete with and without a crack existing along the shear plane prior to the application of shear. The research concluded that the dowel action of reinforcing bars crossing the shear plane is insignificant in initially uncracked concrete, but it is substantial in concrete with a pre-existing crack along the shear plane.

Julio et al. [4] recently carried out the experimental study based on conclusions drawn from previous studies on longitudinal shear strength of strengthening concrete overlays. From the analysis of the experimental results, it was possible to conclude that

i. The reinforcement crossing the interface does not significantly increase the interface debonding stress.

ii. The shear strength of the interface increases with the increase of reinforcement crossing the interface

iii. For low reinforcing ratios, the shear strength of the interface corresponds to the debonding stress.

iv. For higher reinforcing ratios, the shear strength of the interface is not reached immediately on debonding but only after a considerable slip.

v. There is a difference of 6.6% to 8.3% having the reinforcement placed before casting the substrate and having it inserted into hardened substrate.

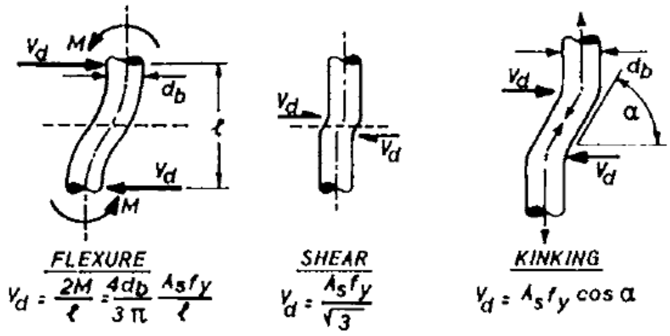


Figure 6: The mechanisms of dowel action [2]

White et al. [5] performed an investigation on dowel action and interface shear transfer under cyclic loading. It was found that the load slip behavior for dowel action alone is similar to that of interface shear transfer, except the lesser residual slip after unloading for dowel action alone. The application of axial tensile forces on the interface reinforcement also resulted in large increases in slip at the interface for a given applied load.

2.1.2. Effect of interface surface preparation

Generally, a rough surface along a construction joint considerably increased the interface shear capacity. The effect of interface surface preparation on shear resistance studied by Robert et al. [1] considered five base block surface preparation techniques. The following surface preparations were taken into account.

i. **Untreated** -As cast
ii. **Heavily sandblasted** - the heavy sandblasting resulted in exposure of the aggregate in the concrete along the interface and depth of surface roughness of about 1/8 inch.
iii. **Chipped to 1/4 inch amplitudes** - chipping to achieve 1/4 inch depth of surface roughness was done by hand, using a pickaxe
iv. **Shear Keys** - Two shear keys cut in to the base blocks along the interface using an electric jackhammer. The 8 x 8 in. keys have been cut in to a 1 inch depth. They have been positioned midway between the locations of the dowels used for interface reinforcement.
v. **Epoxy bonding agent** - The base-block interface coated with an epoxy-bonding agent

The experimental results concluded that for deep surface preparation techniques, such as chipping to 1/4 inch amplitudes and 1 inch deep shear keys has resulted in higher interface shear capacity with higher base block concrete strength. The study further mentioned that for sandblasted interfaces with shallower surface roughness, base block concrete strength has no significant effect on the interface shear capacity. In addition to that, results showed that the higher shear stress attained by the specimen with the interface epoxy would not appear to justify the use of a bonding agent, because the peak shear stresses attained by the specimens occur at a relatively high slip level where most of the bond has been already destroyed.

Paulay et al. [2] have performed a study on shear transfer by concrete to concrete alone with the following types of surfaces at construction joints.

i. **Smooth surface-** One hour after screening the surface has been finished with a steel trowel.
ii. **Rough surface**-One hour after screening the surface has been sprayed with a chemical retarder. The following day the surface over the construction joint area has been washed and scrubbed with a firm brush to expose the coarse aggregate particles.
iii. **Rough scraped-** The screeded surface has been scraped with a pointed trowel in a criss-cross fashion providing approximately ¾ inch (20 mm) deep grooves.
iv. **Rough washed**-Approximately four hours after screeding the surface has been washed and the mortar was removed between the coarse aggregate particles with aid of a brush , whenever necessary .
v. **Rough scabbled-**Four days after the surface has been screeded it has been chipped, using a chisel and hammer , to provide a rough surface.
vi. **Keyed-** Two 4 inch (102mm) long by 1 ½ inch (38mm) deep wooden blocks have been used to form two symentricallly placed recesses over the 66 inch (152mm) width of the 16 inch (406mm) long joint surface.

The experimental results concluded that all joints with rough surfaces and bond such as scrabbled, keyed, washed and trowelled (the upper four curves of Figure 7) showed satisfactory performance. In addition, it is mentioned that the specimen without a joint was not superior to the other samples with rough surface construction joint, as it depends on the strength of the concrete.

Juio et al. [6] have examined the bond strength with various surface roughening techniques which are most commonly used in practice. The following situations have been considered for the study.

i. Surface cast against steel formwork (to serve as reference)
ii. Surface prepared with steel brush
iii. Surface partially chipped
iv. As in iii. Plus water saturation 24h prior to concrete cast
v. Surface treated with sand-blasting

Situation (iv), considered to analyze the advantage of pre-wetting the original concrete surface before casting new concrete.

From the experimental results, it was concluded that the sand blasting was the preparation method of the substrate surface which presented the higher values of bond strength in shear and in tension, among all the considered techniques. It is further indicated that the influence of pre-wetting the substrate surface has no significant effect on bond strength.

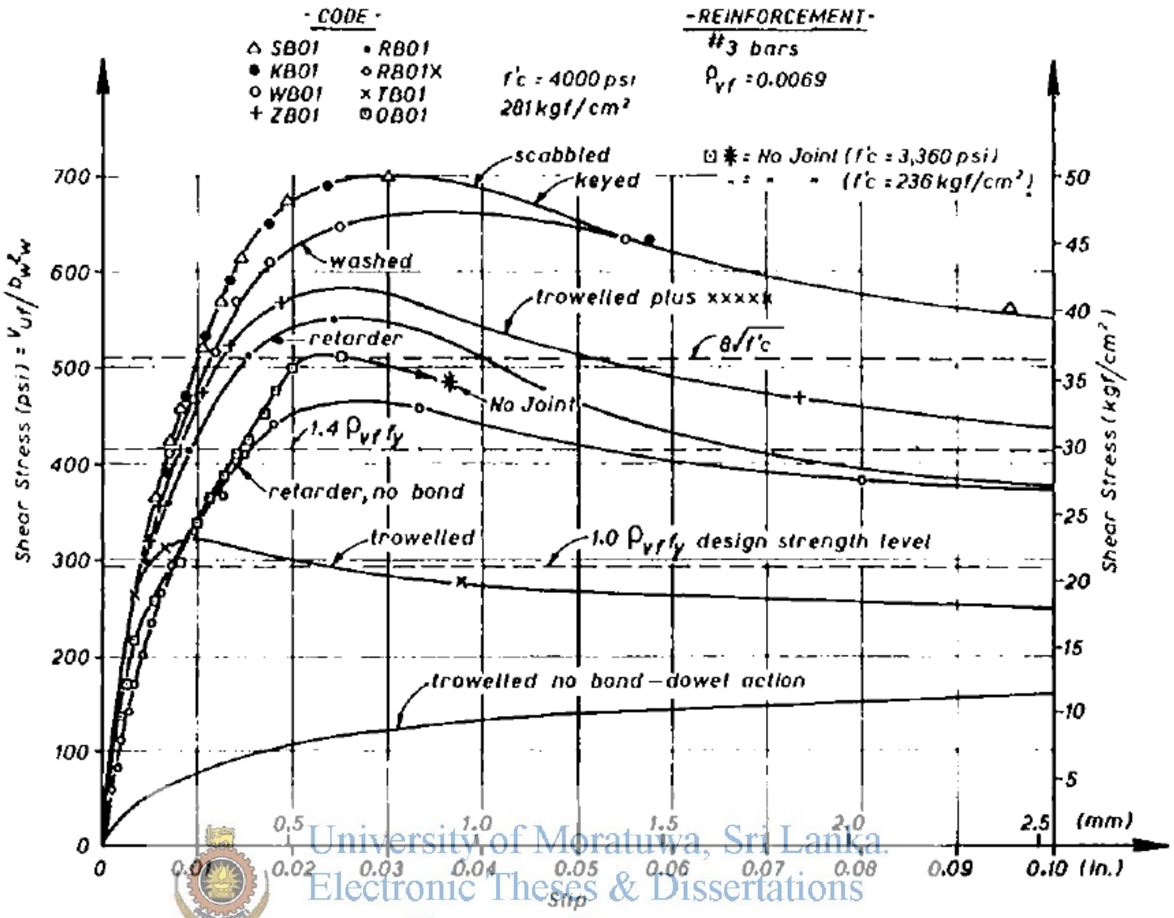


Figure 7: Load-slip curves of concrete shear transfer for various surface preparations [2]

There is no significant difference in the shear capacity of joints was found by Hason [7] when the maximum amplitude of surface roughness at the joint varies between 1/16 and 3/8 inch (3.2 and 9.5 mm). Further, it is found that the higher limit of roughness represented by shear keys are no more effective than rough surfaces with bond.

Loeber [8] has performed an investigation on shear transfer by aggregate interlock and found that the load displacement relationship for shear transfer by aggregate interlock across performed cracks was little affected when 3/8 or 3/4 inch (9.5 to 19mm) round or crushed coarse aggregate were used.

The United States Army Waterways Experiment Station [9] has performed an investigation on method of preparing horizontal construction joints in concrete and the following conclusions were emerged.

i. Wetting the face of the joints before placing concrete against it lowers the bond strength.
ii. Cement grout or a mortar layer placed upon the joints has no significant effect upon bond.
iii. Rough surfaces produced the greatest bond.
iv. The quality of bond is more important when tensile stresses are to be transferred across the joints.

These observations indicated that bond could have a relatively small effect on the interface shear transfer as long as a rough surface is provided.

2.1.3. Compressive strength of existing and new concrete members

Paulay et al. [2] have performed a study on horizontal construction joints in cast in place reinforced concrete and came up with important observations pertaining to the failure plane and concrete strength. In this series of tests, it was observed that, with the exception of the troweled and lightly reinforced rough joints, failure has not occurred along the plane of the joints. Based on the observations, authors concluded that in a well-performed design and executed construction joint, the plane of failure could be expected to be located below the level of the joint in a layer of inferior concrete. The strength capacity, therefore, would not be governed by the surface condition along a joint. The quality of the concrete immediately below the joint of actual structures is likely to be worse than that obtained in these tests. Therefore, author has instructed that the interpretation of the results obtained from this study should not be considered as overlay conservative.

Robert et al. [1] investigated the effect of compressive concrete strength of existing member and new materials. The researchers concluded that for sandblasted interfaces (shallower roughness), base-block concrete strength has no significant effect on the interface shear capacity.

The United States Army Waterways Experiment Station [9] has performed experiment to observe the effect of concrete strength on interface shear capacity. The conclusion states that the compressive strength of the concrete has little influence on the interface shear resistance.

The experiment performed by Julio et al. [10] have revealed that the added concrete, normally with higher compressive strength than the substrate concrete, may possibly have an influence on concrete-to-concrete interface shear strength.

2.2. Design considerations

2.2.1. Accuracy of design code expressions

Julio et al. [4] recently carried out the experimental study on accuracy of design code expressions for estimating longitudinal shear strength of strengthening concrete overlays. This study included an evaluation of shear strength between a sandblasted concrete substrate and a concrete overlay with different amount of transvers reinforcements at the interface. In addition to that, the following design codes were considered for assessing its accuracy for longitudinal shear strength based on shear friction theory.

i. Portuguese Code (REBAP)-1984
ii. Euro Code 2 (EC 2)-1-2008
iii. CEB-FIP Model Code 90 (MC 90) -1990
iv. Canadian Code (CSA)-A23.3-2004
v. USA Code(ACI 318)-2008
vi. British Code (BS 8110)-1997

Based on the study, the researchers have emerged the following conclusions.

i. substantial differences are registered between the values given by each of the codes considered.
ii. Comparing experimental/numerical values with code expressions, it can be stated that the values given by EC 2 and MC 90 for low reinforcing ratios are not safe. (Figure 8)

iii. the ratio of the shear strength to normal stress (the friction coefficient) is approximately 1.3 for experimental results and numerical analysis data; significantly higher than the corresponding ratio for code expressions which vary between 0.6 and 1.0 (Figure 8)

iv. Due to this fact, the codes with the exception of REBAP and MC 90, tend to predict the shear strength conservatively for higher levels of normal stress. (Figure 8)

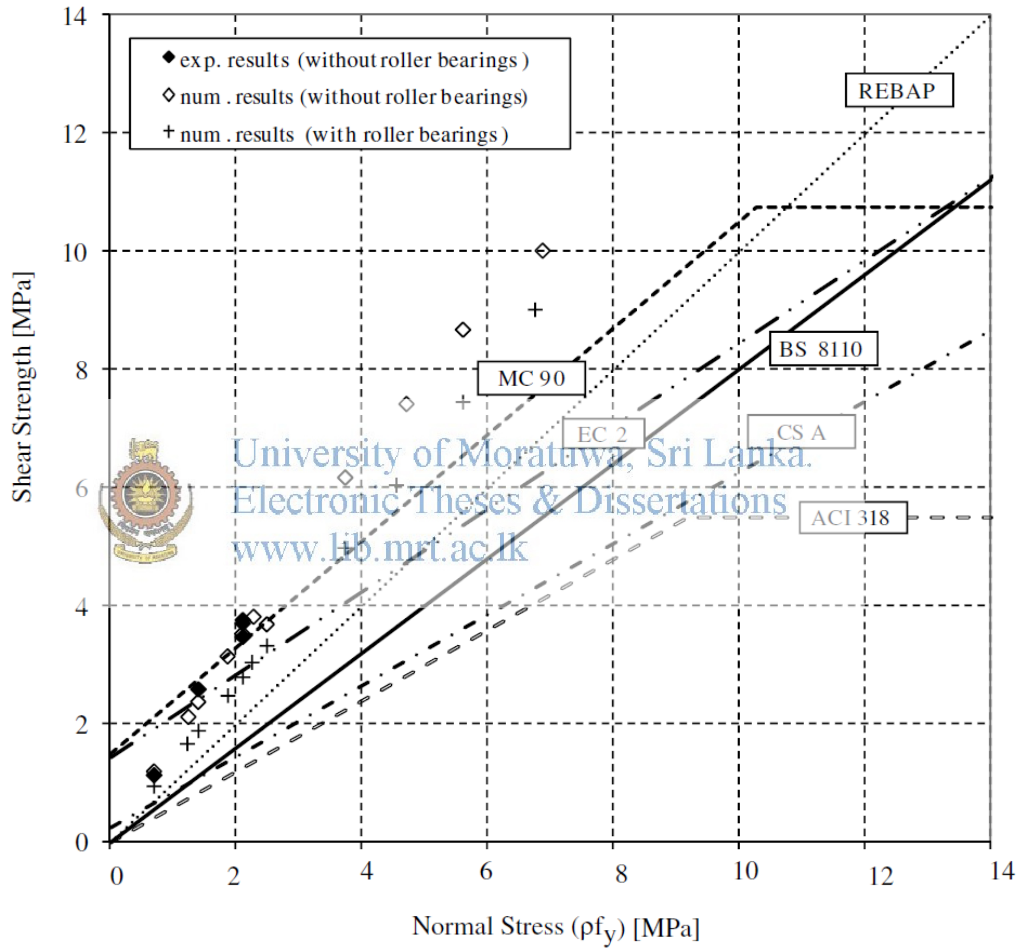


Figure 8: Experimental /numerical versus analytical ratios "shear strength/normal stress," according to different codes [3]

2.2.2. EN-1992-1-1-2004 [11] Approach for horizontal shear

Shear stresses at the interfaces between concrete elements cast at different times must be checked to ensure whether the two concrete components act compositely. The shear and bending designs of such members are based on this assumption. Clause 6.2.5 of the standard deals with this interface shear requirement, which must be considered in addition to the requirements of clauses 6.2.1- 6.2.4

Clause 6.2.5(1) and equation 6.23 states that the interfaces should be checked to ensure $v_{Edi} < v_{Rdi}$, where

v_{Edi} is the design value of shear stress in the interface and it is given by following equation,

$v_{Edi} = \beta V_{Ed} / (z b_i)$

where:

β is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or the tension zone, both calculated for the section considered

V_{Ed} is the total vertical shear force for the section

z is the lever arm of the composite section

b_i is the width of the interface shear plane

v_{Rdi} is the design shear resistance at the interface.

Equation 6.24 in the standard is used by assuming that all loads are carried on the composite section, which is adaptable with the design approach for ultimate flexure. The shear stress for design at the interface is related to the maximum longitudinal shear stress at the interface between compression and tension zones given by $V_{Ed} / (z\ b_i)$. considering equilibrium of the forces in either the tension or the compression zone.

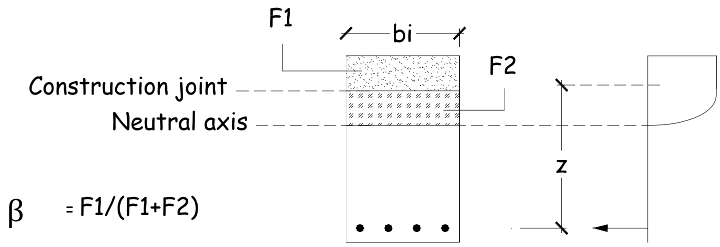


Figure 9: Determination of β

If the shear plane lies within either the compression or the tension zones, the shear stress from equation $v_{Ed} = V_{Ed} /(z\ b_i)$ may be reduced by the factor β mentioned previously. It is always conservative to take β = 1.0. As far as the flanged beams are concerned, much of the force is contained in the flanges. In that case, construction joints at the underside of flange will typically have $\beta \sim 1.0$. In other cases, β can be calculated from the forces F1 and F2 as shown in Figure 9 by flexural design. It is important to make sure the value to be used for the lever arm, z. The value of z shall reflect the stress block in the beam for the loading considered. Since the calculation of z is time-consuming, it is reasonable to use the same value obtained from the ultimate bending resistance analysis, as shown in Figure 4. For cracked sections at lower bending moments, the use of the ultimate bending resistance lever arm would slightly overestimate the actual lever arm.

The design shear strength at the interface is based on the EN-1992-1-1-2004 [7] provisions and is given in clause 6.2.5(1) as:

$v_{Rdi} = c\, f_{ctd} + \mu\sigma_n + \rho f_{yd}\,(\mu \sin\alpha + \cos\alpha) < 0.5\nu\, f_{cd}$ - Equation(6.25)

where

f_{ctd} = $\alpha_{ct}\,(f_{ctk,0.05}) / \gamma_c$

α_{ct} = coefficient taking account of long term effects on the tensile strength and of unfavorable effects, resulting from the way the load is applied

$\alpha_{ct} = 1.0)$

$f_{ctk,0.05}$ = $1.8\ (f_{ck}=25\ MP_a$, *Table 3.1 of* EN-1992-1-1-2004*)*

γ_c = partial safety factor for concrete for design situations of persistent & transient $(\gamma_c=1.5)$

c and μ are factors, which depend on the roughness of the interface.

Recommended values for c and μ are given in clause 6.2.5(2) and reproduced in Table1. Other factors are defined in clause 6.2.5(1).

Table 1: **c** and $\boldsymbol{\mu}$ factors for interfaces of concrete elements cast at different times

Interface characteristic	**c**	**μ**
Very smooth-(a surface cast against steel, plastic or specially prepared wooden moulds)	0.25	0.5
Smooth-(a slip formed or extruded surface, or a free surface left without further treatment after vibration)	0.35	0.6
Rough-(a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behavior)	0.45	0.7
Indented-(a surface with indentations complying with Figure 6.9 of EN-1992-1-1-2004)	0.50	0.9

σ_n = stress per unit area caused by the minimum external normal force across the interface that can act simultaneously with the shear force, positive for compression, such that $\sigma_n < 0.6\, f_{cd}$, and negative for tension. When σ_n is tensile cf_{ctd} should be taken as zero.

ρ = A_s / A_i

A_s = area of reinforcement crossing the interface, including ordinary shear reinforcement (if any),

A_i = area of the interface

b_i = width of the interface

α = defined in Figure 6.9 of EC2, and should be limited by $45° \leq \alpha \leq 90°$

v = strength reduction factor for concrete cracked in shear

= $0.6(1-f_{ck}/250)$

f_{ck} = Characteristic compressive cylinder strength of concrete at 28 days

f_{cd} = $\alpha_{cc}\,(f_{ck}/\,\gamma_c)$

γ_c = partial factor for concrete

α_{cc} = coefficient taken into account of long term effects on the compressive strength and unfavorable effects resulting from the way the load is applied (α_{ct} =1.0)

The first term in equation (6.25) represents the influence of adhesion between the surfaces, the second term relates to friction across the interface under the action of compressive stress, σ_n, and the third term represents the resistance provided by the reinforcement crossing the interface. The reinforcement provided for shear in accordance with clauses 6.2.1 to 6.2.4 may be considered in the reinforcement ratio ρ. The amount of interface reinforcement is considered as a whole including the ordinary shear reinforcement.

2.2.3. ACI 318M-2011 [12] Approach for horizontal shear

The ACI 318M-(2011) approach consists of seven different criteria for horizontal shear design including four different equations. The use of each of these equations depends on the magnitude of the factored shear force and on the conditions of the interface. The design for horizontal shear is based on the following fundamental equation:

$V_u \leq \phi V_{nh}$

Where

V_u = factored shear force (required strength)

V_{nh} = nominal horizontal shear resistance

ϕ = 0.75 (strength reduction factor for shear)

For the factored shear force, *ACI 318M-(2011)* allows the designer to choose either the factored vertical shear force or the actual change in compressive or tensile force in any segment.

The horizontal shear resistance is determined as follows:

If $V_u \geq \phi 3.5\ b_v d$ then

$V_{nh} = A_{vf} f_y \mu < \min\{0.2f'_c A_c \text{ or } (3.3+0.08f'_c)\ A_c \text{ and } 11A_c\}$

If $V_u < \phi 3.5\ b_v d$ then

1) $V_{nh} = 0.55\ b_v d$ if the contact surface is clean, free of laitance, intentionally roughened and no shear reinforcement is provided.

2) $V_{nh} = 0.55\ b_v d$ if minimum ties are provided $(A_v = 0.35 b_v s/f_y)$, contact surfaces are clean and free of laitance, but not intentionally roughened.

3) $V_{nh} = (1.8+0.6\rho_v f_y)\lambda\, b_v d < 3.5\, b_v d$ if contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude approximately 6 mm and more than the minimum amount of shear reinforcement is provided.

In the above equations:

b_v = width of the interface

d = distance from extreme compression fiber to centroid of tension reinforcement for the entire composite section

A_{vf} = area of shear reinforcement crossing the interface

f_y = yield stress of the shear reinforcement

μ = coefficient of friction and depends on surface conditions.

= 1.4λ for concrete placed monolithically

= 1.0λ for concrete placed against hardened concrete with surface intentionally roughened.

= 0.6λ for concrete placed against hardened concrete with surface not intentionally roughened.

= 0.7λ for concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars

λ = 1.0 for normal weight concrete

= 0.85 for sand-lightweight concrete

= 0.75 for all lightweight concrete

A_c = area of concrete engaged in shear transfer

ρ_v = ratio of the area of steel to the area of concrete.

The method provided in the ACI code is assuming an occurrence of crack along the shear plane considered. The ACI code allows the designer to use any other method whose results are in good agreement with comprehensive tests.

2.2.4. BS 8110-1-1997 [13] Approach for horizontal shear

According to code provisions at the interface of the precast and in situ components, the horizontal shear force due to design ultimate loads (V_h) is divided into two categories as given below.

a) Where the interface is in the tension zone: the total compression (or tension) calculated from the ultimate bending moment; or

b) Where the interface is in the compression zone: the compression from that part of the compression zone above the interface, calculated from the ultimate bending moment.

The horizontal shear force, V_h, (Figure 10) acting at the interface can be calculated from horizontal equilibrium of the cast in situ component

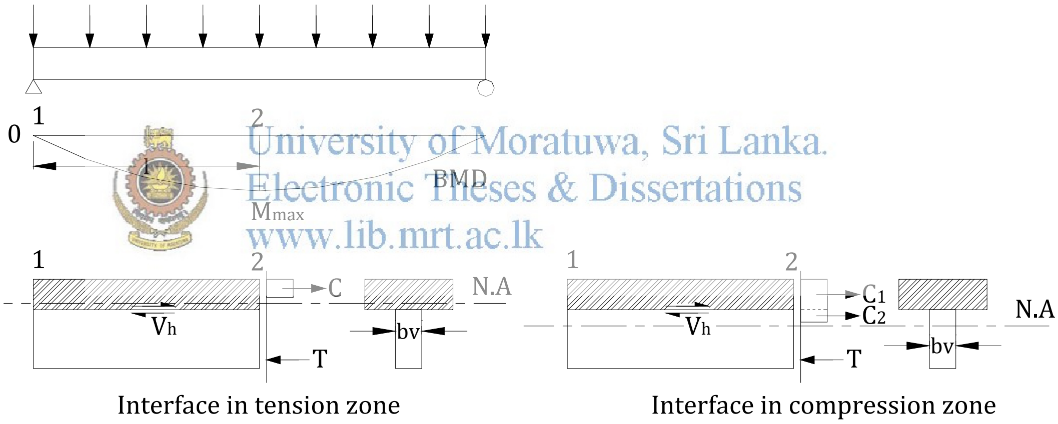


Figure 10: Horizontal shear force

When interface in the tension zone

$V_h = C$ or T

When interface in the compression zone

$V_h = C_1$

Where b_v is the width of contact surface and l is the distance between point of maximum moment and the point of zero moment

The average horizontal design shear stress is calculated by dividing the design horizontal shear force by the area obtained by multiplying the contact width by the beam length between the point of maximum positive or negative design moment and the point of zero moment.

The average horizontal shear stress is given as, $(v_h)_{av} = V_h / (b_v) l$

The average design shear stress should then be distributed in proportionate to the vertical design shear force diagram to give the horizontal shear stress at any point along the length of the member. The design shear stress (v_h) should be less than the appropriate value in Table 5.5 of the standard.

When Nominal links are provided, nominal links should be of cross-section at least 0.15 % of the contact area. Spacing should not be excessive. The spacing of links in T-beam ribs with composite flanges should neither exceed four times the minimum thickness of the in situ concrete nor 600 mm, whichever is the greater. Links should be adequately anchored on both sides of the interface.

When a higher resistance to horizontal shear is needed, the beam may be provided with nominal links that are projected through the interface and are anchored in the cast in situ concrete

Nominal links are defined as

$A = 0.15 (b_v) l / 100$

The spacing of shear links (S_v) should be less than the smaller value of $4h_f$ and 600 mm

h_f = minimum thickness of the in situ concrete

Links in excess of minimum where the horizontal shear stress exceeds the value given in Table 5.5 in the standard and reproduced as Table 2. All the horizontal shear force should be carried by reinforcement anchored either side of the interface. The amount of steel required A_h (in mm^2/m) should be calculated from the following equation:

$A_h = 1000\, b_v\, v_h / 0.87 f_{yv}$

Table 2: Design ultimate horizontal shear stresses at interface [13]

Precast unit	Surface type	Grade of in situ concrete		
		25 N/mm^2	30 N/mm^2	40 and over N/mm^2
Without links	As-cast or as-extruded	0.4	0.55	0.65
	Brushed, screeded or rough-tamped	0.6	0.65	0.75
	Washed to remove laitance or treated with retarder and cleaned	0.7	0.75	0.80
With nominal links projecting into in situ concrete	As-cast or as-extruded	1.2	1.8	2.0
	Brushed, screeded or rough-tamped	1.8	2.0	2.2
	Washed to remove laitance or treated with retarder and cleaned	2.1	2.2	2.5

NOTE 1 The description "as-cast" covers those cases where the concrete is placed and vibrated leaving a rough finish. The surface is rougher than would be required for finishes to be applied directly without a further finishing screed but not as rough as would be obtained if tamping, brushing or other artificial roughening had taken place.
NOTE 2 The description "as-extruded" covers those cases in which an open-textured surface is produced direct from an extruding machine.
NOTE 3 The description "brushed, screeded or rough-tamped" covers those cases where some form of deliberate surface roughening has taken place but not to the extent of exposing the aggregate.
NOTE 4 For structural assessment purposes, it may be assumed that the appropriate value of γ_m included in the table is 1.5.

2.2.5. Other proposed equations

Many studies have been performed by various researchers to develop an equation that provides satisfactory prediction of the interface horizontal shear strength. The following sections present different predicted equations that have been proposed by various researchers.

2.2.5.1. Linear shear friction equation

This equation was introduced by Mast [14] and later developed further by their co-workers:

$$v_n = \rho f_y \mu$$

Where v_n is the shear strength, ρ is the ratio of the reinforcement crossing the interface and; f_y is the yield stress of the reinforcement; and μ is the friction coefficient at the interface of the concrete.

2.2.5.2. Mattock's and Hawkins's equations

An important improvement was published by Mattock et al. [15] which is known today as the modified shear friction theory is given by following Equation

$v_n = 1.38 + 0.8(\rho f_y + \sigma_n)$

Where σ_n is the effect of the external normal force across the interface. The friction coefficient assumes the value 0.8; normal stresses at the interface, for the first time, cohesion of the interface is also considered including the aggregate interlocking, assuming the value being 1.38 MPa.

2.2.5.3. Loov's Equation

Loov [16] introduced a parabolic equation for horizontal shear strength which includes the effect of concrete strength. Loov expressed it as an equation, which gave an important contribution as follows.

$$(v_n / f_c) = k \sqrt{\{(\rho f_y + \sigma_n)/ f_c\}}$$

Where, the concrete compressive strength, f_c is taken into account for the first time, k is a constant equal to 0.5, for initially un-cracked interfaces:

2.2.5.4. Walraven's Equations

Walraven [17] has performed a statistical analysis of push-off test results and suggested an expression assuming perfectly spherical aggregate, this is given by following eqation.

$$v_n = C_1 (\rho f_y)^{C2}$$

where, $C_1 = 0.822 f_c (MP_a)^{0.406}$ and $C_2 = 0.159 f_c (MP_a)^{0.303}$

2.2.5.5. Randl's Equation

Randl [18] has proposed a design expression that separates and precisely incorporates the three influencing parameters (cohesion, friction and dowel action), as shown in following equation

$$v_n = \tau_{coh} + \mu \sigma_n + \alpha_{da} \rho \sqrt{(f_c f_y)}$$

Where τ_{coh} is the cohesion due to aggregate interlock, μ is friction coefficient and α_{da} is a coefficient relevant to the dowel action.

CHAPTER 3

FINITE ELEMENT MODELING AND ANALYSIS

CHAPTER 3 – FINITE ELEMENT MODELING AND ANALYSIS

3.1 Analysis of the pile cap by simplified 2D-model

3.1.1 Model Geometry

Three Dimensional configuration of dispersed precast concrete piles and piers over the existing pile cap (Figure 11) was simulated to a two dimensional configuration by projecting precast piles to a vertical plane taken along the centerline of the bridge piers (Figure 12). this model is prepared to observe the shear behavior along the longitudinal direction (X) of the pile cap.

In addition to the spanning of the pile cap along the X direction, the pile cap is spanning along the Y direction also, due to the fact that the precast piles are located away from the centerline of the bridge piers. To observe the shear behavior in the transverse direction, Model No. 2D-T (Figure 13) is prepared by projecting precast piles to a vertical plane taken along the centerline of the pier in the Y direction. Appropriate plane is selected by considering maximum pier load.

3.1.2 Applied Loads

Loading data considered in the original design of this particular pile cap was not available. The ultimate limit state loading was approximately calculated according to BS 5400 part 2-1978 for a critical load combination.

Concentrated Loads and Bending Moments indicated in the Model No 2D-L (Figure 12), Model No 2D-T (Figure 13) are ULS loads transferred to the pile cap from the superstructure through piers.

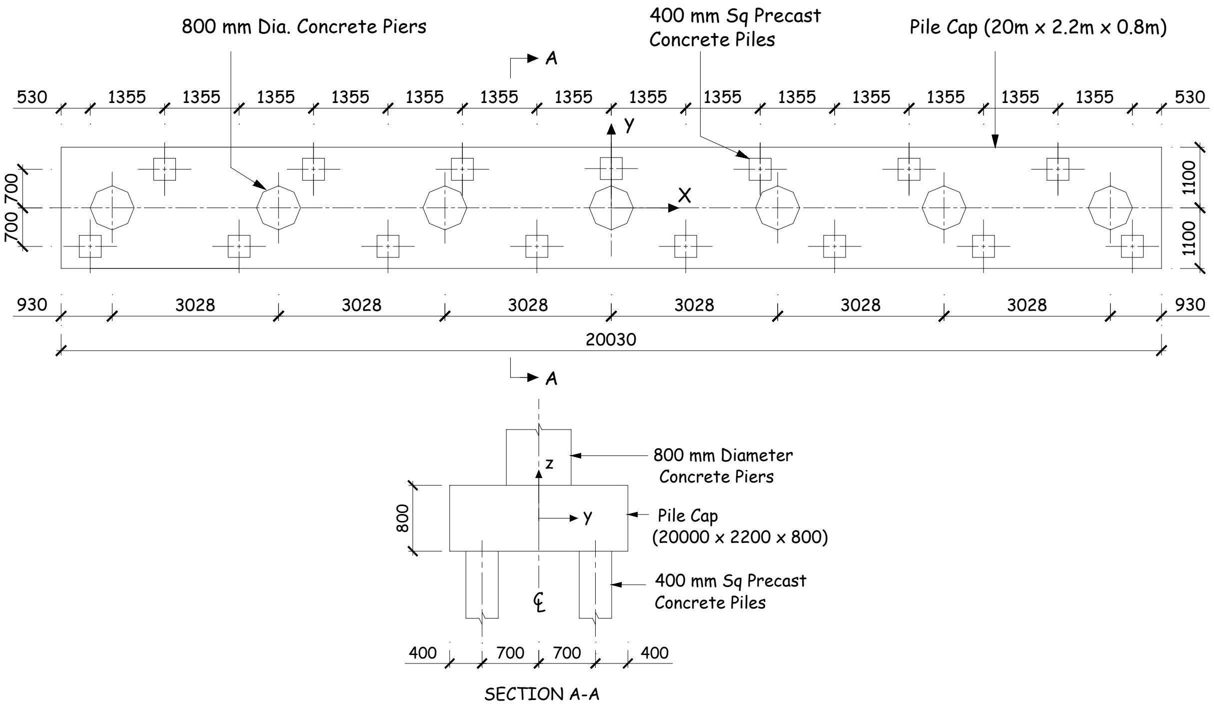


Figure 11: Details of the existing pile cap

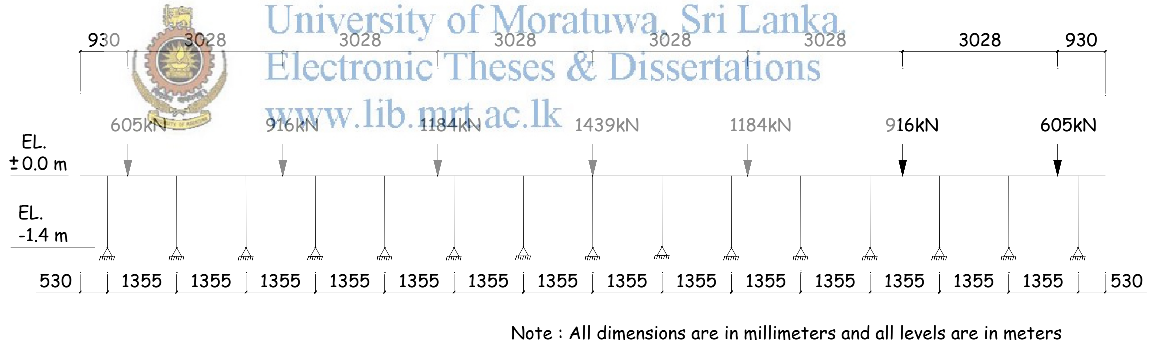


Figure 12: Model No: 2D-L

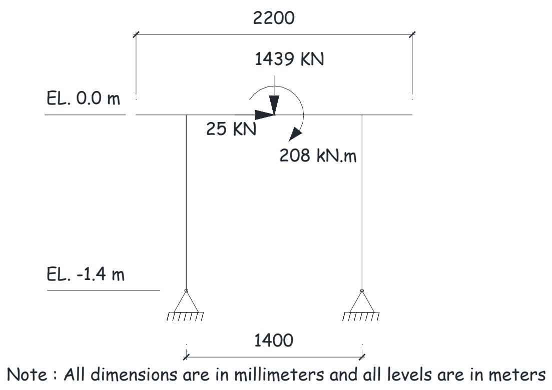


Figure 13: Model No: 2D-T

3.1.3 Finite Elements Used

Two-dimensional models (Model No: 2D-L, 2D-T) were prepared by using two node frame elements having six degrees of freedom per node. Pile-cap and precast piles were modeled by using frame elements. Frame elements having cross sectional dimensions of 800 mm x 2200 mm and 800 mm x 3028 mm is selected for these models respectively. Precast piles were modeled with a frame elements having cross sectional dimensions of 400 mm x 400 mm.

Grade of concrete and self-weight of the concrete were considered as 30 N/mm^2 and 24 kN/m^3 respectively. Modulus of elasticity was assigned as 26 kN/mm^2.

Total number of frame elements, nodes, stiffness degrees of freedom, and mass degree of freedoms pertaining to the models prepared for longitudinal and transverse directions are given in Table 3

Table 3: Characteristics of 2D-FE Models

	Model: 2D-L	Y-Direction
Total number of frame elements	37	6
Total number of nodes	38	7
Number of stiffness degree of freedom	183	36
Number of mass degree of freedom	69	15

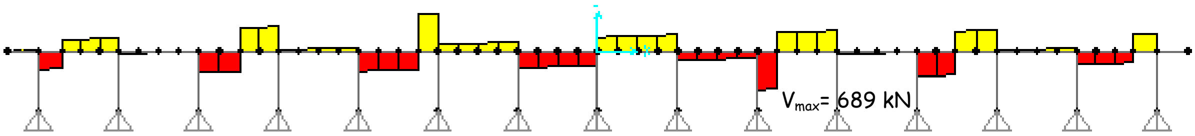
3.1.4 Boundary conditions

Simple pin supports were introduced at the bottom of the precast piles and length of the precast piles were limited to 1.4 m from the centerline of the pile cap. To reduce the computer time on modeling and analyzing of FE Models, this depth of precast pile is limited to 1.4m. At this depth, the point of contraflexure is observed in the precast piles by model output results of a 2D FEM analysis, which comprised 8m length precast piles with soil springs.

3.1.5 Output Results

Linear elastic analysis was performed for models no. 2D-L (Figure 12), 2D-T (Figure 13) and results are given in Figure 14 and Figure 15. Maximum shear forces obtained are 689 kN and 872 kN along longitudinal direction and transverse direction of the pile-cap respectively. In addition to that, distance between maximum moment and zero moment were obtained at the regions where overlay is subjected to tension and compression. (Figure 9), (Figure 10)

Previously mentioned maximum shear forces are required for interface shear stress calculation in the longitudinal direction and the transverse direction of the pile cap separately.



SFD

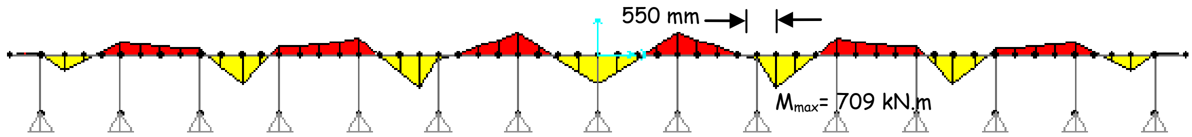


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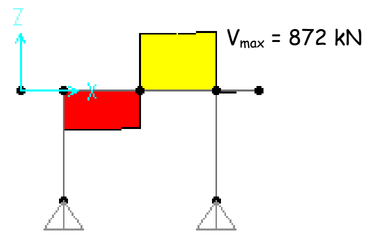
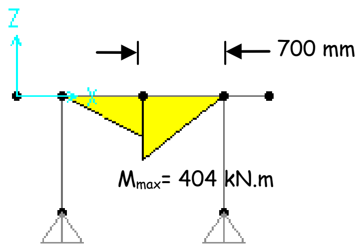
Figure 14: Shear force, Bending Moment variation along the X direction of the pile cap at ULS Loads



SFD



BMD

Figure 15: Shear force, Bending Moment variation along the Y direction of the pile cap at ULS Loads

3.2 Analysis of the pile cap by 3d-model

3.2.1 Model Geometry

Geometrical configuration of the pile cap with RC precast piles and RC piers generate the complex shear stress distribution within the body of the concrete. In addition to the geometry, the load distribution on the pile cap is also not symmetrical, which contributes to the complexity of the problem to some extent. Therefore, in order to observe the in-plane and out-of-plane shear stress distribution at the level of interface, three dimensional finite element model (FEM) was prepared by using 8 node solid elements in SAP 2000. Symmetry of geometry of the pile cap was also considered with an assumption of symmetrical loading in the solid model to minimize the time taken for modeling and analysis (Figure 16).

For the element surface load calculation, circular cross sectional piers were simplified as rectangular cross sectional piers and there by equivalent cross sectional areas were considered.

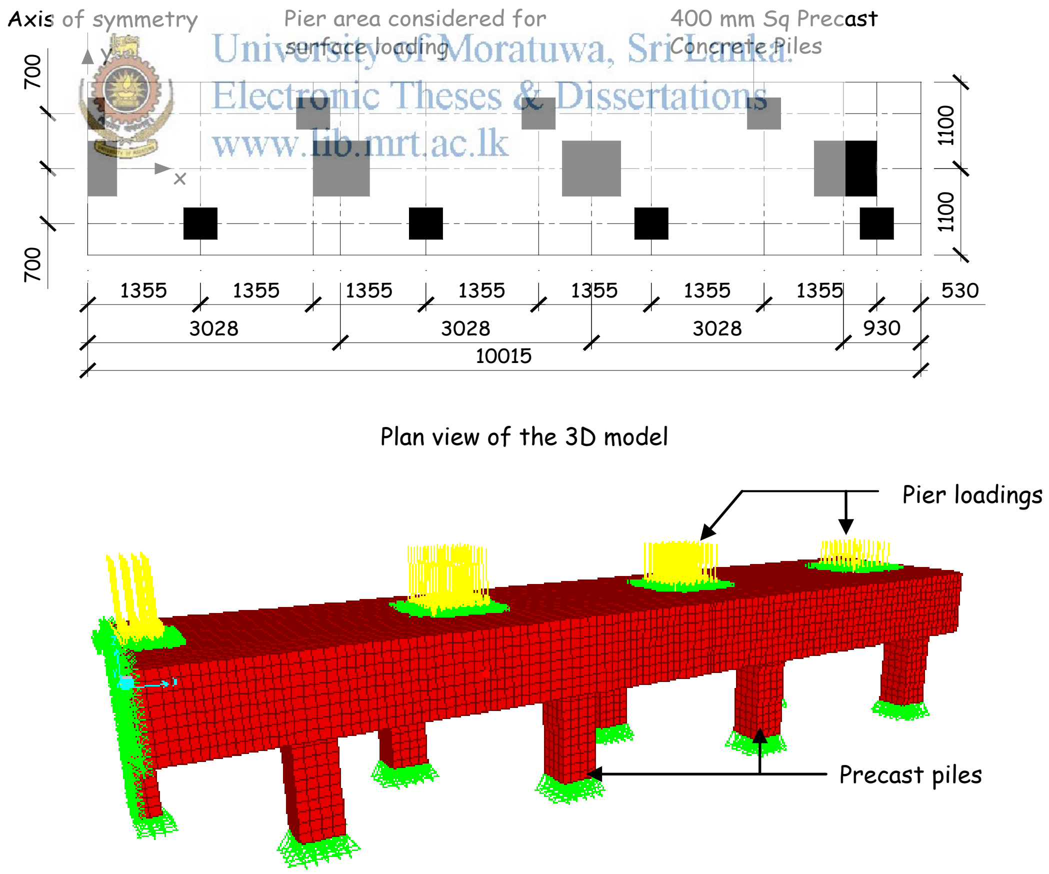


Figure 16: 3D Finite element solid model (Half of the pile cap)

3.2.2 Finite Elements Used

Linear elastic three dimensional finite element model was prepared by using eight node 3D solid elements having three degrees of freedom per node, The element facilitates eight integration points for its numerical integration. Material properties are assigned as same as the 2D finite element model.

Total number of solid elements, nodes, stiffness degrees of freedom and mass degree of freedoms pertaining to the model are shown in Table 4 separately.

Table 4: Characteristic of 3D-FE Model

Total number of solids	23088
Total number of nodes	27775
Number of stiffness degree of freedom	82495
Number of mass degree of freedom	82495

3.2.3 Applied Loads

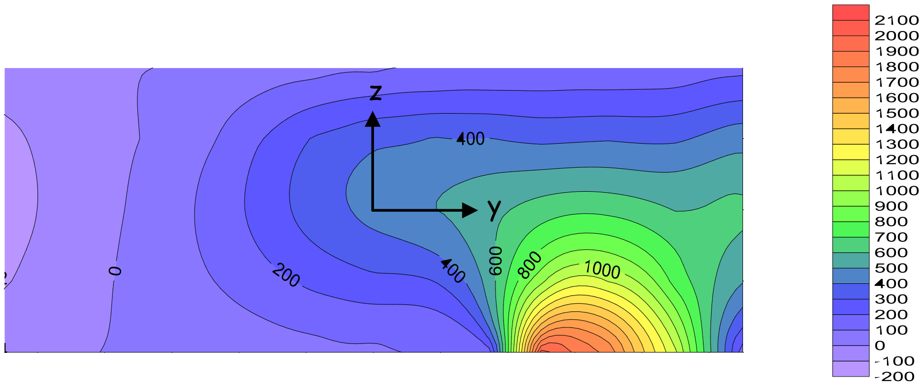
Ultimate limit state (ULS) loading applied to the model is as same as the values considered in 2D- FE models. However, loads coming from piers were applied to the pier areas (Figure 16) of the model as surface loadings.

3.2.4 Boundary conditions

Boundary conditions of the precast piles are exactly similar to those in the 2D-FE model. For nodes in the vertical plane section, where symmetry was considered, horizontal translations were fixed and vertical translations were allowed.

3.2.5 Output results

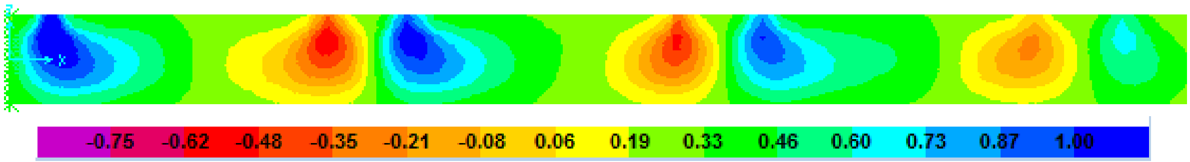
Linear elastic analysis was performed for the model and output results are shown in Figure 17 and Figure 18. The shear force of the 3D model acting at the location corresponding to the maximum shear force obtained from the 2D-finite element model (i.e. at x =2.71 m) were calculated by the volume bounded by the shear stress contours over the cross section of the pile cap at x =2.71 m (Figure 17) by using Simpson’s rule. The resulting shear force is 613 kN



Shear stress contours are in KN/m^2

Figure 17: Shear stress distribution over the depth of the pile cap

(Along the transverse direction at x = 2.71 m)



Shear stress contours are in N/mm^2

Figure 18: Shear stress distribution over the depth of the pile cap

(Along the longitudinal direction at y = 0)

3.3 Comparison of FE model analysis results

Two dimensional and three dimensional finite element model output results were compared based on the shear force observed in the 2D model and the shear force calculated from 3D model at a similar location (i.e. at x = 2.71 m). Shear force and stress variation in 2D and 3D models along the longitudinal direction were considered for this comparison and results obtained are shown in Table 5.

The intention of analyzing 3D-finite element model is to observe whether the complicated finite element model analysis is always essential for structures having complex geometrical shapes. Also, to see whether the simplified two dimensional model analysis results are significantly varied compared to 3D-finite element analysis results.

The output results show that the 2D- finite element analysis are not significantly varied compare to 3D-finite element analysis results and gives conservative value for design purpose . It is evident that the assumptions made during the simplified 2D model preparation are reasonable.

The shear force values of 689 kN and 872 kN obtained from the 2D FEM related to models 2D-L, 2D-T are used for design shear stress calculation in the Chapter 4.

Table 5: Shear force obtained from model analysis

Type of Model	Shear force/(kN)
2D-FE Model	689
3D-FE Model	613

CHAPTER 4

DESIGN OF DOWELS FOR INTERFACE SHEAR

CHAPTER 4 – DESIGN OF DOWELS FOR INTERFACE SHEAR

4.1 Design shear stress at the interface

The following design shear stress calculation was done based on the results obtained by 2D-FEM analysis as shown in Figure 14 and Figure 15.

4.1.1 EN-1992-1-1-2004 Approach

(a) Horizontal design shear stress (v_{Edi}) calculation in X direction

The shear stress for design at the interface is related to the maximum longitudinal shear stress at the interface between compression and tension zones given by $\beta.V_{Ed}/(z.b_i)$ considering equilibrium of the forces in either the tension or the compression zone. (Figure 19)

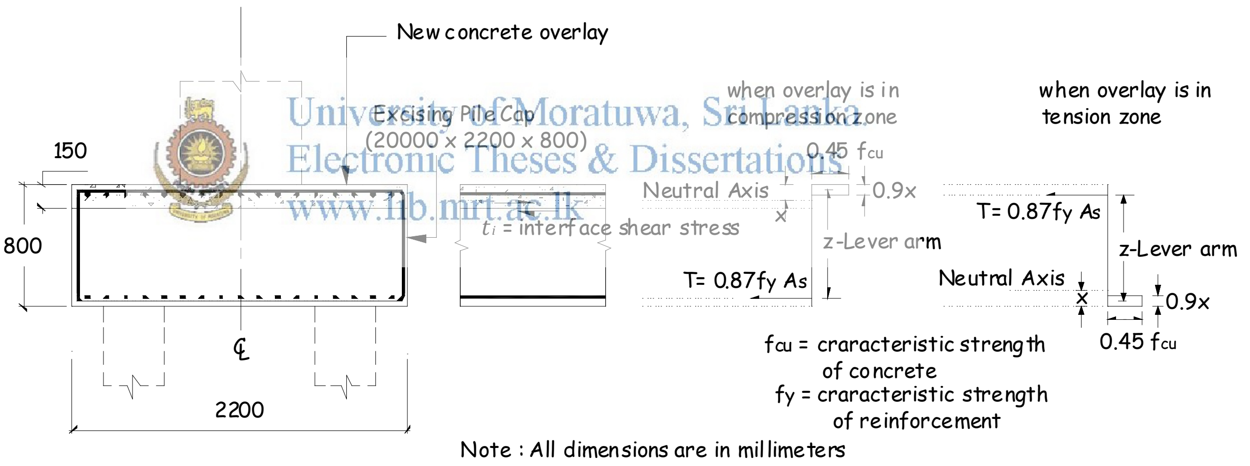


Figure 19: Stress distribution of cracked concrete section

Lever arm *(z)* can be calculated as follows based on the equation given in the *CL-3.4.4.4* in the *BS 8110*

Maximum Sagging Bending Moment *(M)* = 709 kN.m - (Figure 14)

$K = M/bd^2f_{cu}$ - *(CL-3.4.4.4, BS 8110-1:97)*

b = 2200 mm - (Figure 11)

Diameter of longitudinal bars = 16mm - (Figure 20)

Diameter of distribution bars = 16mm - (Figure 20)

Cover to reinforcement = 45 mm

Effective depth, *(d)* = 800-45-16-8 = 731 mm

Grade of concrete, (f_{cu}) = 30 N/mm^2

K = 709 x 10^6/(2200x731^{2}x30) = 0.02

z = *d {0.5+√ (0.25-K/0.9)} - (CL-3.4.4.4, BS 8110-1:97)*

z = 731{0.5+√ *(0.25-0.02/0.9)}*

z = 714.4 > 694.5 (0.95d)

z = 694.5 mm

$v_{Edi} = \beta V_{Ed} / (z\, b_i)$ - *(Eq. 6.24, CL-6.2.5)*

β = 1.0 - (Interface is in the tension zone, *CL-6.2.5)*

V_{Ed} = 689 kN - (Figure 14)

v_{Edi} = 689 x 10^3 / (694.5x 2200)

v_{Edi} = 0.45 N/mm^2

(b) Horizontal design shear stress *(v_{Edi})* calculation in Y direction

Maximum Sagging Bending Moment *(M)* = 404 kN.m - (Figure 15)

K = M/bd^2f_{cu} - *(CL-3.4.4.4, BS 8110-1:97)*

b = 3028 mm (Figure 11)

Diameter of longitudinal bars = 16mm - Appendix B

Diameter of distribution bars = 16mm - Appendix B

Cover to reinforcement = 45 mm

Effective depth, *(d)* = 800-45-8 = 747 mm

Grade of concrete, (f_{cu}) = 30 N/mm^2

K = 404 x 10^6/ (3028 x 747^2 x 30) = 0.008

z = *d {0.5+√ (0.25-K/0.9)} - (CL-3.4.4.4, BS 8110-1:97)*

z = 747{0.5+√ *(0.25-0.008/0.9)}*

z = 740 > 710 (=0.95d)

z = 710 mm

$v_{Edi} = \beta V_{Ed} / (z\, b_i)$ - *(Eq. 6.24, CL-6.2.5)*

β = 1.0 - (Interface is in the tension zone, *CL-6.2.5)*

V_{Ed} = 872 kN - (Figure 15)

v_{Edi} = 1.0 x 872 x 10^3 / (710 x 3028)

v_{Edi} = 0.41 N/mm^2

4.1.2 ACI 318M-2011 [12] Approach

(a) Horizontal design shear *(V_u)* calculation in X direction

Neutral axis depth *(x)* can be calculated as follows based on the equation given in the *CL-3.4.4.4* in the *BS 8110*

z = 694.5 mm - {Refer Chapter 4.1.1(a)}

d = 731 mm - {Refer Chapter 4.1.1(a)}

x = (d – z)/0.45 - *(CL-3.4.4.4, BS 8110-1:97)*

x = (731 – 694.5)/0.45 = 81.1 mm

V_u = $(0.45f_{cu} \times 0.9x \times b)$ - (Refer Figure 19)

b = 2200 mm - (Figure 11)

V_u = (0.45 x 30 x 0.9 x 81.1 x 2200) x 10^{-3} = 2168 kN

The average horizontal shear stress is given as,

v_u = $V_u / b\, l$

l = 550 mm - (Figure 14)

v_u = 2168 x 10^3/ (2200 x550) = 1.79 N/mm^2

(b) Horizontal design shear *(V_u)* calculation in Y direction

z = 710 mm - {Refer Chapter 4.1.1(b)}

d = 747 mm - {Refer Chapter 4.1.1(b)}

x = (d – z)/0.45 - *(CL-3.4.4.4, BS 8110-1:97)*

x = (747 – 710)/0.45 = 82.2 mm

V_u = $(0.45f_{cu} \times 0.9x \times b)$ - (Refer Figure 19)

b = 3028 mm - (Figure 11)

V_u = (0.45 x 30 x 0.9 x 82.2x 3028) x 10^{-3} = 3024.1 kN

l = 700 mm - (Figure 15)

v_u = 3024.1 x 10^3/ (3028 x700) = 1.43 N/mm^2

4.1.3 BS 8110-1-1997 [13] Approach

(a) Horizontal design shear stress *(v_h)* calculation in X direction

The horizontal shear force, V_h, acting at the interface can be calculated from horizontal equilibrium of the cast in situ component (Figure 19).

V_h $= (0.45f_{cu} \times 0.9x \times b)$ - (Refer Figure 19)

b = 2200 mm - (Figure 11)

V_h = $(0.45 \times 30 \times 0.9 \times 81.1 \times 2200) \times 10^{-3}$ - {Refer Chapter 4.1.2(a)}

= 2168 kN

The average horizontal shear stress is given as,

$(v_h)_{av}$ $= V_h / b\,l$

l = 550 mm - (Figure 14)

$(v_h)_{av}$ $= 2168 \times 10^3 / (2200 \times 550) = 1.79$ N/mm^2

Shear force diagram is almost uniform over a length of 550 mm (Figure 14); therefore, design shear stress (v_h) is very close to the value of average shear stress.

Hence,

v_h = 1.79 N/mm^2

(b) Horizontal design shear stress *(v_h)* calculation in Y direction

V_h $= (0.45f_{cu} \times 0.9x \times b)$ - (Refer Figure 19)

b = 3028 mm - (Figure 11)

V_h = $(0.45 \times 30 \times 0.9 \times 82.2 \times 3028) \times 10^{-3}$ - {Refer Chapter 4.1.2(b)}

= 3024.1 kN

l = 700 mm - (Figure 15)

$(v_h)_{av}$ $= 3024.1 \times 10^3 / (3028 \times 700) = 1.43$ N/mm^2

Shear force diagram is almost uniform over a length of 700 mm (Figure 15); therefore, design shear stress (v_h) is very close to the value of average shear stress.

Hence,

v_h = 1.43 N/mm^2

4.2 Design shear resistance at the interface

4.2.1 EN-1992-1-1-2004 Approach

The calculation of design shear resistance at the interface is based on the EN-1992-1-1-2004 [7]

a) Calculation of design shear resistance (v_{Rdi}) for a unit length of the pile cap along the longitudinal direction, X of the pile cap.

The clause 6.2.5.1 of EN 1992-1-1 gives the following equation to calculate v_{Rdi}

v_{Rdi} $= c\, f_{ctd} + \mu\sigma_n + \rho f_{yd}\, (\mu \sin\alpha + \cos\alpha) < 0.5\nu\, f_{cd}$ - *(Eq. 6.25, CL-6.2.5-1)*

α_{ct} = coefficient taking account of long term effects on the tensile strength and of unfavorable effects, resulting from the way the load is applied (α_{ct} =1.0)

γ_c = partial safety factor for concrete for design situations of persistent & transient (γ_c=1.5)

$f_{ctk,0.05}$ = 1.8 - *(f_{ck}=25 MP_a, Cylinder strength for C25/30, Table 3.1-EN 1992-1-1 -)*

f_{ctd} $= \alpha_{ct}\, (f_{ctk,0.05}) / \gamma_c$ - *(Eq. 3.16, CL-3.1.6-2P)*

f_{ctd} = 1.0 x 1.8/1.5 = 1.2

c and μ are factors, which depend on the roughness of the interface - (*CL-6.2.5-2*), and assumed as 0.45 and 0.7 respectively, which represent the characteristics of surface with at least 3 mm roughness at approximately 40 mm spacing, This can be achieved by raking, exposing of aggregate or other methods giving an equivalent behavior

Normal stress acting at the interface due to self-weight is very small and it is assumed as zero, ($\sigma_n = 0\ N/mm^2$)

ρ $= A_s / A_i$

A_s = Area of reinforcement crossing the interface, including ordinary shear reinforcement

b_i = 2200 mm (Figure 19)

A_i = 1000 x 2200 = 2.2 x 10^6 mm^2 (Area of the joint considering 1m length)

f_{yd} = 0.87 f_{yk} = 0.87x460 = 400 N/mm^2

α = 90° - *(Inclination of dowels to the interface)*

v = *Shear strength reduction factor*

= $0.6(1\text{-}f_{ck}/250)$

= 0.6(1-25/250) = 0.54 - *(Eq. 6.6N, CL-3.1.6)*

γ_c = partial factor for concrete

α_{cc} = coefficient taken into account of long term effects on the compressive strength

= 1.0 - *(Recommended value-CL-3.1.6)*

f_{cd} = $\alpha_{cc}\,(f_{ck}/\,\gamma_c)$

= 1.0 (25/ 1.5) =16.67 - *(Eq. 3.15, CL-3.1.6) and (CL-2.4.2.4)*

v_{Rdi} = $c\,f_{ctd} + \mu\sigma_n + \rho f_{yd}\,(\mu \sin\alpha + \cos\alpha) < 0.5\nu\, f_{cd}$

v_{Rdi} = $0.45 \times 1.2 + \rho \times 400 \times (0.7 \times \sin 90 + \cos 90) < 0.5 \times 0.54 \times 16.67$

v_{Rdi} = $0.54 + 280\rho < 4.5$ N/mm^2

A_s = 3090 mm^2 (i.e. area of r/f crossing the interface -2 x 6T16+ 6T12)

- (Figure 20, 21, and 23)

This area of reinforcement contribute to the shear capacity for mid-way between piers

ρ = 3090/ (2.2 x 10^6) = 0.0014

v_{Rdi} = 0.54 + 280 x0.0014 =0.54 + 0.39 = 0.93 N/mm^2 < 4.5 N/mm^2 - ok

v_{Edi} = 0.45 N/mm^2 - (Chapter 4,1,1, (a))

$v_{Edi} < v_{Rdi}$

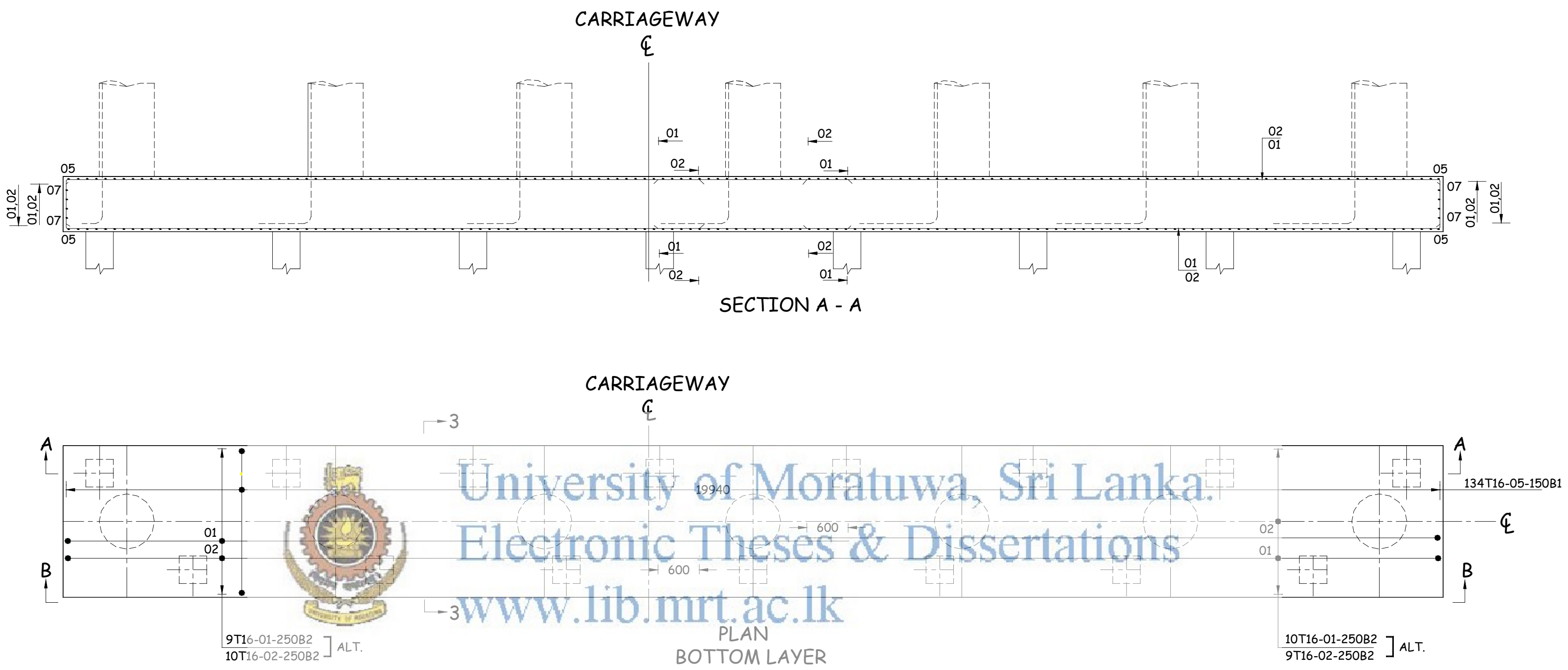
It is observed that the concrete and r/f contribution to the design shear resistance is 0.54 N/mm^2 and 0.39 N/mm^2 separately. Therefore, the concrete shear friction itself is enough to transfer horizontal shear without any additional interface shear reinforcements.

Nominal shear r/f ratio, $(\rho_{w,\,min})$ = $(0.08 \times \sqrt{f_{ck}})/f_{yk}$ - *(Eq. 9.5N, CL-9.2.2.5)*

= (0.08 x √25)/ 460

= 0.0009 < 0.0014 (i.e. ρ at between piers)

Area of nominal shear r/f = 0.0009 x 1000 x 2200 = 1980 mm^2/m



NOTES:

1. ALL DIMENSIONS ARE IN MILIMETRES.
2. CONCRETE FOR PILE CAPS TO BE OF GRADE 30
3. MINIMUM CLEAR COVER TO REINFORCEMENT TO BE 45mm FROM THE EXPOSED SURFACE.
4. TYPICAL BAR NOTATION

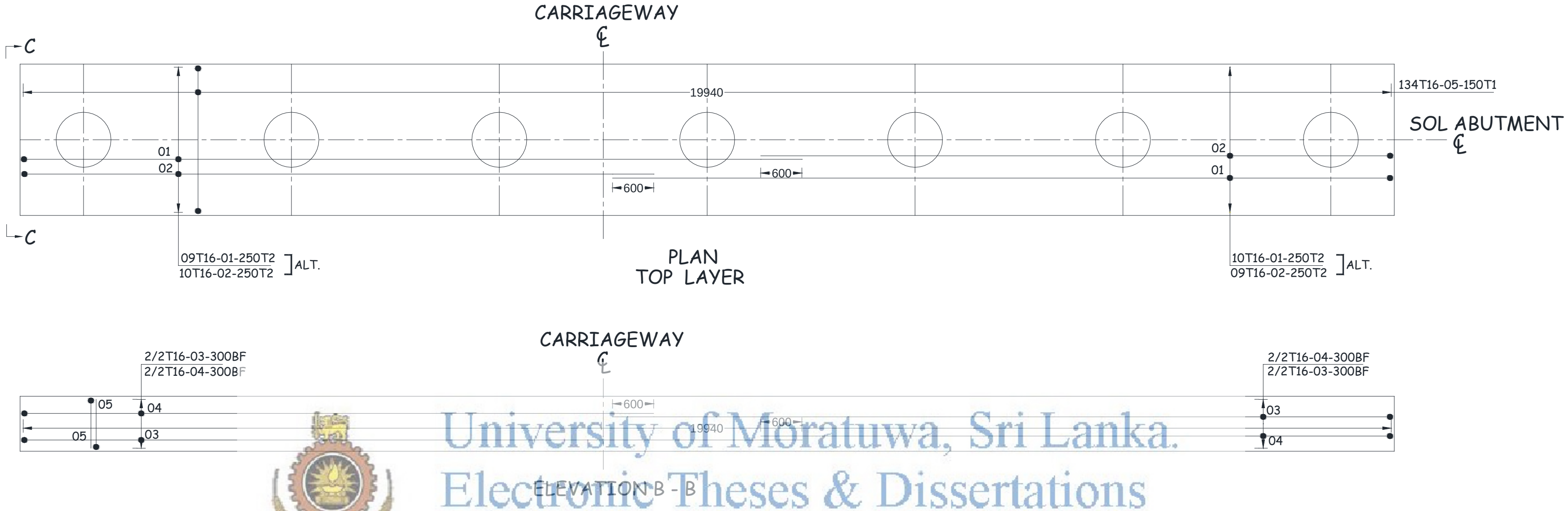
10 T 10 - 06 - 300 B1

- No OF BARS
- TYPE OF STEEL
- BAR DIAMETER IN mm
- BAR MARK
- BAR SPACING IN mm
- BAR LOCATION

R - MILD STEEL fy = 250 N/mm²
T - HIGH YIELD STEEL fy = 460 N/mm²

BAR LOCATION
T1 - TOP OUTER MOST LAYER
T2 - TOP INNER LAYER
B1 - BOTTOM OUTER MOST LAYER
B2 - BOTTOM INNER LAYER
BF - BOTH FACE

Figure 20: Details of reinforcement at the bottom of the existing pile-cap



NOTES:

1. ALL DIMENSIONS ARE IN MILIMETRES.
2. CONCRETE FOR PILE CAPS TO BE OF GRADE 30
3. MINIMUM CLEAR COVER TO REINFORCEMENT TO BE 45mm FROM THE EXPOSED SURFACE.
4. TYPICAL BAR NOTATION

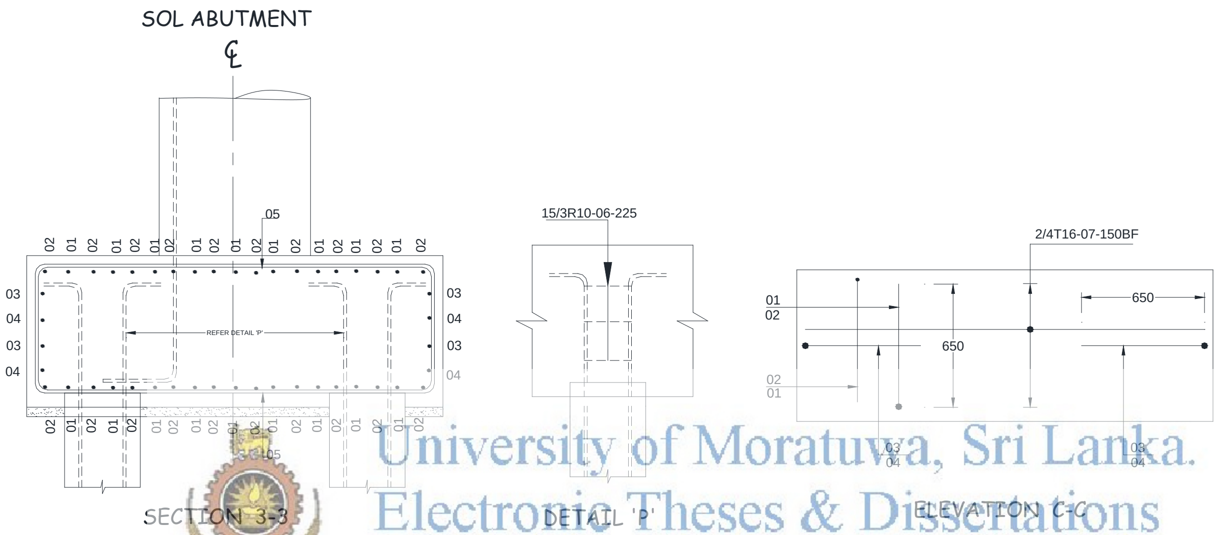
10 T 10 - 06 - 300 B1

No OF BARS
TYPE OF STEEL
BAR DIAMETER IN mm
BAR MARK
BAR SPACING IN mm
BAR LOCATION

T1 - TOP OUTER MOST LAYER
T2 - TOP INNER LAYER
B1 - BOTTOM OUTER MOST LAYER
B2 - BOTTOM INNER LAYER
BF - BOTH FACE

R - MILD STEEL fy = 250 N/mm²
T - HIGH YIELD STEEL fy = 460 N/mm²

Figure 21: Details of reinforcement at the top of the existing pile-cap



NOTES:

1. ALL DIMENSIONS ARE IN MILIMETRES.
2. CONCRETE FOR PILE CAPS TO BE OF GRADE 30
3. MINIMUM CLEAR COVER TO REINFORCEMENT TO BE 45mm FROM THE EXPOSED SURFACE.
4. TYPICAL BAR NOTATION

10 T 10 - 06 - 300 B1

No OF BARS
TYPE OF STEEL
BAR DIAMETER IN mm
BAR MARK
BAR SPACING IN mm

BAR LOCATION
T1 - TOP OUTER MOST LAYER
T2 - TOP INNER LAYER
B1 - BOTTOM OUTER MOST LAYER
B2 - BOTTOM INNER LAYER
BF - BOTH FACE

R - MILD STEEL fy = 250 N/mm²
T - HIGH YIELD STEEL fy = 460 N/mm²

Figure 22: Details of reinforcement at sections, elevations of the existing pile-cap

Hence, nominal area of shear reinforcement of 1980 mm^2 crossing the interface along the longitudinal direction, X of the pile-cap over an area of 2200 mm x 1000 mm is needed.

Maximum longitudinal spacing between links ($s_{l.max}$)

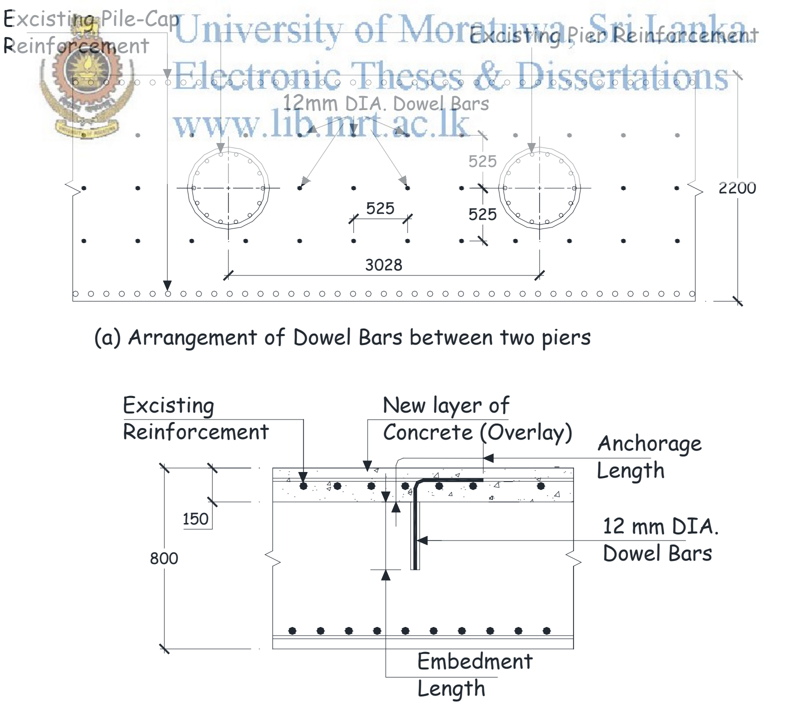
$$s_{l.max} = 0.75\, d \times (1 + \cot \alpha) \text{ - (Eq. 9.6N, CL-9.2.2.6)}$$

Effective depth, d = 800-45-16-8 = 731 mm

$$s_{l.max} = 0.75 \times 731 \times (1 + \cot 90)$$

$$= 548.25 \text{ mm}$$

Assuming that new Y12 dowel bars are introduced with a spacing of 525 mm along the X directions (Figure 23(a)) between two layers of existing perimeter reinforcement. The existing reinforcing bars satisfy the minimum area of r/f. However, it has not fulfilled the minimum spacing requirement. The following arrangement of shear dowels (Figure 23) gives reasonably conservative arrangement of dowels.



(a) Arrangement of Dowel Bars between two piers

(b) Anchorage and embedment details of Dowel Bars

Note : All dimensions are in millimeters

Figure 23: r/f bars which considered in calculating design shear resistance

b) Calculation of design shear resistance for full width of the pile cap along the Y Direction. (pier to pier distance of 3.028 m along X direction was considered)

b_i = 3028 mm (i.e. pier to pier distance)

A_i = 2200 x 3028 = 6.66 x 10^6 mm^2

v_{Rdi} = 0.54 + 280ρ - {Refer Chapter 4.2.1.(a)}

A_s =21104 mm^2 (i.e. existing transverse bars-2 x 20T16 + pier r/f of 14T32 + dowel bars of 16T12) (Figure 20, 21, and 23)

ρ = 21104/ (6.66 x 10^6) = 0.0032

v_{Rdi} = 0.54+ 280 x0.0032 =0.54 + 0.90= 1.44 N/mm^2 < 4.5 N/mm^2

v_{Edi} = 0.45 N/mm^2 - (Chapter 4,1,1, (b))

v_{Edi} < v_{Rdi}

Contribution of the concrete and r/f on the design shear resistance is 0.54 N/mm^2 and 0.90 N/mm^2 separately. The concrete shear friction itself is enough to transfer horizontal shear without any additional interface shear reinforcements.

Nominal shear r/f ratio, $(\rho_{w,min})$ = $(0.08 \times \sqrt{f_{ck}})/f_{yk}$ - (Eq. 9.5N, CL-9.2.2.5)

= (0.08 x √25)/ 460

= 0.0009 < 0.003 (i.e. at piers)

Area of nominal shear r/f = 0.0009 x 1000 x 3028 = 2725.2 mm^2/m

Hence, nominal area of shear reinforcement of 2725.2 mm^2 crossing the interface along the transverse direction, Y of the pile-cap over an area of 3028 mm x 1000 mm is needed.

Maximum transverse spacing between links $(s_{t.max})$

$s_{t.max}$ = $0.75\, d \leq 600$ mm - (Eq. 9.8N, CL-9.2.2.8)

$s_{t.max}$ = 0.75 x 731 = 548.25 mm <600 mm - ok

Y12 dowel bars are introduced with a spacing of 525 mm along the Y directions (Figure 23(a)) between two layers of existing perimeter reinforcement so that the minimum spacing requirement to be satisfied. Because the existing reinforcing bars have already satisfied the minimum area of r/f but it has not fulfilled the minimum spacing requirement. The above arrangement of shear dowels gives reasonably conservative arrangement of dowels.

4.2.2 ACI 318M-11 Approach

The following calculations for design shear resistance at the interface are based on the ACI 318M- 2011 [8]

$V_u \leq \phi V_{nh}$ *- (Eq. 17.1, CL-17.5.3)*

Where

V_u = factored horizontal shear stress

V_{nh} = nominal horizontal shear resistance

ϕ = 0.75 *- (CL-9.3.2.3-strength reduction factor for shear)*

When $V_u < \phi$ (3.5$b_v d$)

a) Calculation of design shear resistance for a unit length of the pile cap along the longitudinal direction, X of the pile cap.

$b_v d$ = Area of contact surface considered- *(CL-17.5.4)*

= 2200 x 1000 = 2.2x 10^6 mm^2- (Figure 14)

V_u = 2168 kN - (Chapter 4, 1, 2, (a))

V_u = 2168 < ϕ3.5($b_v d$) =0.75 x 3.5 x 2.2 x 10^6 = 5775 kN - ok- *(CL-17.5.4)*

V_{nh} = $(1.8+0.6\rho_v f_y)\,\lambda\, b_v d < 3.5\, b_v d$ - *(CL-17.5.3.3)*

Here it was assumed that the contact surfaces are clean, free of laitance and intentionally roughened to full amplitude of approximately 6mm and more than the minimum amount of shear reinforcement is provided. - *(CL-17.5.3.3)*

ρ_v = A_s / A_i

A_s = Area of reinforcement crossing the interface,

A_s = 3090 mm^2 (i.e. area of r/f crossing the interface -2 x 6T16+ 6T12)

- (Figure 20, 21, and 23)

This area of reinforcement contribute to the shear capacity for mid-way between piers

ρ_v = 3090/ (2.2 x 10^6) = 0.0014

b_i = 2200 mm - (Figure 14)

A_i = 1000 x 2200 = 2.2 x 10^6 mm²

The code specified value of specified yield strength of reinforcement (f_y) used for design of shear friction reinforcement shall not exceed 420 N/mm^2. (CL-11.6.6)

f_y = 420 N/mm^2- *(CL-11.6.6)*

λ = 1.0 for normal weight concrete - *(CL-11.6.4.3)*

V_{nh} = $(1.8+0.6\rho_v f_y)\lambda\, b_v d < 3.5\, b_v d$

V_{nh} = (1.8 + 0.6 x 0.0014x 420) x 1.0 x 2.2 x 10^6 < 3.5 x 2.2 x 10^6

$\phi\, V_{nh}$ = 0.75 x (1.8 + 0.588) x 1.0 x 2.2 x 10^6< 0.75 x (3.5 x 2.2 x 10^6)

= (1.35+0.44) x 2.2 x 10^6 = 3938 kN < 5775 kN - ok

Design shear resistance, ($\phi\, V_{nh}$) > V_u

Nominal area of shear r/f ($A_{v,min}$) = $0.062\sqrt{fc'}\,(b_w.s/f_{yt})$ - *(CL-11.4.6.3)*

= 0.062 x √30 x (2200 x 1000)/420

= 1778.8 mm^2/m

Check for ($0.35\, b_w.s/f_{yt}$) = ($0.35\, b_w.s/f_{yt}$)

= 0.35 x (2200 x 1000)/420

= 1833 mm^2/m > 1778.8 mm^2/m

Hence, it is needed to satisfy area of 1833 mm^2 as nominal reinforcement along the longitudinal direction, X of the pile-cap over an area of 2200 mm x 1000 mm

b) Calculation of design shear resistance for full width of the pile cap along the Y direction. (pier to pier distance of 3.028m along X direction was considered)

b_i = 3028 *mm* (i.e. pier to pier distance)

A_i = 2200x 3028 = 6.66 x 10^6 mm^2

A_s = 21104 mm^2 (i.e. existing transverse bars-2 x 20T16 + pier r/f of 14T32 + dowel bars of 16T12) (Figure 20, 21, and 23)

ρ = 21104/ (6.66 x 10^6) = 0.0032

$V_{nh} \quad = (1.8+0.6\rho_v f_y)\lambda\, b_v d < 3.5\, b_v d$

$V_{nh} \quad = (1.8 + 0.6 \times 0.0032 \times 420) \times 1.0 \times 6.66 \times 10^6 < 3.5 \times 6.66 \times 10^6$

$\phi V_{nh} = 0.75 \times (1.8 + 0.756) \times 1.0 \times 6.66 \times 10^6 < 0.75 \times (3.5 \times 6.66 \times 10^6)$

$= (1.35+0.57) \times 6.66 \times 10^6 = 12787.2$ kN < 17483 kN - ok

Design shear resistance, $(\phi\, V_{nh}) > V_u$

Nominal area of shear

reinforcement $(A_{v,min})$ $= 0.062\sqrt{fc'}\,(b_w.s/f_{yt})$ - (CL-11.4.6.3)

$= 0.062 \times \sqrt{30} \times (3028 \times 1000)/420$

$= 2448$ mm^2

Check for $(0.35\ b_w.s/f_{yt})$ $= (0.35 \times b_w.s/f_{yt})$

$= 0.035 \times (3028 \times 1000)/420$

$= 2649.5$ mm^2 > 2448 mm^2

Hence, it is needed to satisfy area of 2649.5 mm^2 as nominal reinforcement along the transverse direction, Y of the pile-cap over an area of 3028 mm x 1000 mm.

The above calculations for (a),(b) were done based on the assumption that the evenly distributed tie spacing does not exceed four times the least dimension of support element, nor exceed 600 mm and ties are adequately anchored on both sides of the interface.

4.2.3 BS 8110-1-1997 Approach

The calculation of design shear resistance at the interface is based on the BS 8110-1-1997 [9]

a) Calculation of design shear resistance for a unit length of the pile cap along the longitudinal direction, X of the pile cap.

According to the recommendation given in CL-5.4.7.2, the allowable design shear stress for precast unit with nominal links projecting in to in-situ concrete having surface type of brushed, screeded or rough tamped for grade 30 of in-situ concrete is 2.00 N/mm^2

According to the recommendation given in CL-5.4.7.2, Nominal cross sectional area of links (i.e. 0.15% (of the contact area, Ai)) should be provided connecting the existing concrete and the overlay for 1m length of pile cap.

A_i = 1000 x 2200

= 2.2 x 10^6 mm^2

Nominal cross sectional area of links required = 0.0015 x 2.2 x 10^6

= 3300 mm^2

Provided area of reinforcement crossing the interface = 3090 mm^2 (i.e. area of existing transverse bars-2 x 6T16 +6T12) - (Figure 23)

Hence, the balance area of 210 mm^2 is required to achieve interface design shear resistance of 2.00 N/mm^2 as specified in the standard.

b) Calculations of design shear resistance for unit width of the pile cap along the Y Direction. (pier to pier distance of 3.028m along X direction was considered)

b_i = 3028 mm (i.e. pier to pier distance)

A_i = 2200x 3028 = 6.66 x 10^6 mm^2

A_s = 21104 mm^2 (i.e. existing transverse bars-2 x 20T16 + pier r/f of 14T32 + dowel bars of 16T12) (Figure 20, 21, and 23)

Nominal cross sectional area of links = 0.15% (of the contact area, Ai)
= 0.0015 x 6.66 x 10^6
= 9990 mm^2

Area of nominal shear reinforcement,
for unit length of Y direction = 9990x 1000/2200 = 4541 mm^2

Hence, the value of design shear resistance can be expected higher value than the code specified value of 2.00 N/mm^2

The above calculation was done based on the assumption that the evenly distributed link spacing does not exceed four times the minimum thickness of the in-situ concrete nor 600 mm, whichever is the greater and links are adequately anchored on both sides of the interface.

4.3 Comparison and recommendations given in EN-1992-1-2004, ACI 318M-2011 and BS 8110-1-1997

The three different design codes provisions on design horizontal shear resistance, are different to each other and gives different values. These values are tabulated in following tables. (Table 6, 7)

Table 6: Design shear resistance, minimum area of r/f and dowel spacing at interface

Standard	Design shear stress (N/mm^2)		r/f requirement (mm^2/m)		Design shear resistance (N/mm^2)		Nominal area of r/f (mm^2/m)		Maximum Dowel spacing / (mm)	
	X direction	Y direction	X direction (bi=2200mm)	Y direction (bi=3028mm)	X direction	Y direction	X direction (bi=2200mm)	Y direction (bi=3028mm)	X direction	Y direction
EN-1992-1-1-2004	0.45	0.41	1980.00	2725.20	0.93	1.44	1980.00	2725.20	548.25	548.25
ACI 318M-11	1.79	1.43	1833.00	2649.50	1.79	1.92	1833.00	2649.50	600.00	600.00
BS 8110-1-1997	1.79	1.43	3300.00	4541.00	2.00	2.00	3300.00	4541.00	600.00	600.00

Table 7: Contribution of concrete and reinforcement on design interface shear resistance

Standard	Assumed code specified characteristics for a given surface preparation	Design shear resistance/ (N/mm^2)					
		X direction			Y direction		
		Concrete	R/F	Total	Concrete	R/F	Total
EN-1992-1-1-2004	Rough-(a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behavior)	0.54	0.39	0.93	0.54	0.90	1.44
ACI 318M-11	Contact surfaces are clean free of laitance, and intentionally roughened to a full amplitude of approximately 6mm and more than the minimum amount of shear reinforcement is provided	1.35	0.44	1.79	1.35	0.57	1.92
BS 8110-1-1997	Brushed, screeded or rough-tamped	-	-	2.00	-	-	2.00

4.3.1 Design shear resistance

The calculated design shear resistances at the interface according to the three different design code provisions are tabulated in the table 6 and 7. The BS 8110-1-1997 proposes the maximum design shear resistance while the minimum is provided by EN-1992-1-1-2004. The value obtained by ACI 318M-11 is in between these two standard and close to the value obtained by BS 8110-1-1997.

The ACI 318M-11 propose significant contribution of concrete surface friction on design interface shear resistance than the value propose by the EN-1992-1-1-2004. The BS 8110-1-1997 gives the value of design shear resistance as a total value in which the contribution of concrete and r/f is included.

EN-1992-1-1-2004 and ACI 318M-11 propose contribution of the concrete-to-concrete surface friction on the design shear resistance is significant compare to the resistance provide by the required interface reinforcement which is less than the minimum area of reinforcement. Therefore, the minimum area of reinforcement is indicated as an r/f requirement, which is indicated in the Table 6.

4.3.2 Minimum area of dowel bars

The calculated amount of minimum area of dowel bars passing through the interface are tabulated in the table 6 and 7. The BS 8110-1-1997 proposes the maximum area of dowel bars while the minimum is provided by ACI 318M-11. The value obtained by EN-1992-1-1-2004 is in between these two standard and close to the value obtained by ACI 318M-11

4.3.3 Maximum spacing of dowel bars

The maximum spacing of dowel bars proposed by the three design codes are calculated and tabulated in the table 6 and 7.

The values proposed by the ACI 318M-11 and the BS 8110-1-1997 are identical and higher than the value proposed by the EN-1992-1-1-2004

4.4 Recommendation for repairing the pile cap

- Since the top layer of concrete is defective, it is recommended to replace it with a new layer of same grade or higher grade of concrete compare to the grade of existing concrete.
- It is necessary to remove the defective concrete to a reasonable depth (about 50mm) below the reinforcing bars in order to achieve a good bond between reinforcement and the new layer of concrete.
- When removing defective concrete, heavy breakers shall not be used as it can introduce micro cracks in the sound concrete.
- The dowels shall be arranged as per the details given in Figure 23 throughout the pile cap.
- The dowels shall be anchored to the existing concrete with suitable adhesive material and appropriate embedment length recommended by adhesive material manufacture.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

CHAPTER 5 – CONCLUSIONS AND RECOMMENDATIONS

The results of this study have led to several significant observations about the design of dowels for shear transfer between concrete cast at different times. The following are the important observations.

- Complicated three dimensional finite element model analyses are not always essential for analysis of structures, which are having complex geometrical shapes. It is possible to transform three-dimensional problems to a simplified two-dimensional problem based on the level of accuracy required.
- For the selected surface characteristics and r/f percentage, the estimated design shear resistance based on recommendations of EN-1992-1-1-2004 was found be lower than the corresponding estimated value based on ACI 318M-11 recommendations. It was further observed that BS 8110-1-1997 recommendations gives the highest value for the design shear resistance independent of r/f percentage provided.
- EN-1992-1-1-2004 can be used to compare contribution of concrete interface roughness and interface reinforcement on design shear resistance without any limitation of design shear stress as specified in ACI 318M-11. Furthermore, the EN-1992-1-1-2004 recommends a conservative value for design shear resistance compared to other two standards.
- All three-design standards recommend that the maximum dowel spacing shall not to be more than 600 mm.

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