Column Footings Strengthened with External Prestressing and External Wrapping Systems

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Abstract

Footing enlargement method is widely applied to strengthen bridge footings with structural deficiencies, where dimensions of footings are enlarged by adding new concrete segments. In the traditional method, connections at the contact surfaces are achieved by installing a large number of steel dowels and splicing the flexural reinforcements, which are labor-intensive and time-consuming. To address this problem, five upgraded footing enlargement systems are proposed in this dissertation:

- Circular external prestressing system (CEP);
- Circular external regular reinforcement system (CERR);
- Circular external BFRP wrapping system (CEBW);
- Circular external CFRP wrapping system (CECW);
- Circular external steel jacketing system (CESJ).

In these systems, the connections at the contact surfaces are achieved by the confinement actions provided by different circular external strengthening materials (e.g., prestressing strands in the CEP system, and CFRP wraps in the CECW system). The CEP is an active system, in which the connections are activated during strengthening work. The other four are all passive systems, as the primary connections are activated after the external loads are applied.

By using ABAQUS, a series of finite element models were built to investigate the effectiveness of the five proposed strengthening systems, especially on their improvements in the punching shear capacity. 353 models were built in ABAQUS. For each system, a unique group of parameters were considered and investigated. As an active system, CEP significantly improved the punching shear capacity of RC footings, and the number of prestressing strands had the most significant influence. For the other four passive systems, the improvements in the punching shear
capacities were relatively lower. The parameters investigated such as the area of the regular reinforcements (in CERR), the thickness of the FRP wraps (in CEBW and CECW), and the thickness of the steel jackets (in CESJ) had only slight effects on the enhancements.

The analytical models for predicting the punching shear capacities of footings strengthened by the five proposed systems were all developed from the original model adopted by Eurocode 2, with the critical section located at $d/2$ to the edge of the column. Each analytical model is derived using linear aggression analysis, with the investigated parameters being considered.
Acknowledgments

First and foremost, I would like to express my sincere gratitude to my advisor Professor Riyad S. Aboutaha. “Ph.D. is a life.” He told me that when I started my research three years ago. I was enlightened when I heard that, and these words became more and more valuable with the progressing of my research. All the wealth I acquired from him: the professional knowledge and engineering experiences, the ideas and methods he developed, and the academic attitudes and enthusiasm he holds, will undoubtedly influence my whole life - my Ph.D. life.

I would also like to express my sincere gratitude to Professor Utpal Roy, Professor Baris Salman, Professor Dawit Negussey, Professor Eric M. Lui, and Professor Shobha K. Bhatia, for serving as my committee members. They are all brilliant professors with rich engineering knowledge and experience. I appreciate that I have learned a lot from them. Their insights and comments also greatly enriched my work. Also, I would like to express my gratitude to Department Chair Andria Costello Staniec, Mr. Nicholas A. Clarke, and Ms. Elizabeth Buchanan, for their continuing help and supports in the past five years.

I would also like to express my gratitude to all my friends at Syracuse University, for their help, supports, and encouragement. I will remember all the happy moments I had with them.

Last but not least, I would like to express my gratitude to my parents, my wife, and my daughter, for their love, trust, supports, and encouragement. Their words and smiles are the most potent weapons helping me overcome all the difficulties, in life and research.
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1 INTRODUCTION

1.1 Background

Bridges are known to benefit individuals on a personal level and to promote the economic growth of communities. Conditions of existing bridges in the U.S. are periodically reported by the Federal Highway Administration, for the purpose of monitoring their performance. According to the 2017 Infrastructure Report Card, the grade for bridges is C+ (Figure 1-1). Almost 40% of the total bridges in the U.S. are 50 years or older, despite the fact that most of them are only designed with a lifespan of 50 years. Even worse, nearly 9.1% of the bridges were identified as structurally deficient. This indicates that some critical structural members in those bridges have already deteriorated, and the bridges will be closed if no substantial improvements are made. In addition, for bridges built before the 1970s, they are vulnerable to seismic loads as they were designed based on elastic analysis (McLean et al. 1995; Xiao et al. 1996; Saiidi et al. 2001). Therefore, a significant strengthening of the structurally deficient bridges is urgently needed.

![2017 Infrastructure Grades](image)

Figure 1-1 2017 infrastructure grades (ASCE 2017)

Generally, strengthening an engineered structure includes accommodating the additional
loads, fixing the errors that occurred in the design or construction process, and repairing the
deficient capacity due to damage and/or deterioration. Major bridge strengthening initially took
place in California in the 1970s, driven by the 1971 San Fernando earthquake (Buckle et al. 2006).
In past decades, strengthening has been widely carried out on both superstructures (beams,
bearings, etc.) and substructures (columns, footings, etc.). However, compared to the increasing
attention on strengthening of bridge columns, only limited research has been carried out on bridge
footings. In addition, the strengthening methods for bridge footings are still under-developed.

A bridge footing may be structurally deficient due to insufficient load-carrying capacity,
inadequate flexural or shear capacity, or poor column/footing connection. In general, punching
shear failure is the dominant failure mode. To strengthen deficient footings, the most commonly
applied method is footing enlargement, where plan dimensions of a footing are enlarged by adding
new concrete segments, and in most cases, the depth of the footing is also increased.

1.2 Research Significance

In the traditional footing enlargement method, the connections at the interfaces between
the existing and the additional concrete segments are achieved by installing a large number of steel
dowels and splicing the reinforcements. This type of connection, although effective, is troublesome
to build. To address this problem, in this dissertation, five upgraded footing enlargement systems
are proposed:

- Circular external prestressing system (CEP);
- Circular external regular reinforcement system (CERR);
- Circular external BFRP wrapping system (CEBW);
- Circular external CFRP wrapping system (CECW);
- Circular external steel jacketing system (CESJ).
The CEP system utilizes a post-tensioning system where circular external prestressing strands are installed inside the additional concrete segment. The connection at the interface is achieved by the confinement action, provided by the external prestressing strands. CEP is an active system, as the connection at the interface is activated during retrofit work.

In CERR system, circular regular reinforcing steels are employed inside the additional concrete segment, at the same locations as the strands in CEP. In CEBW, CECW and CESJ systems, BFRP wraps, CFRP wraps, and steel jackets are applied on the exterior surface of the enlarged footing, respectively. These four systems are all passive systems, and each of them has two types of connections: a secondary connection and a primary connection. The secondary connection is achieved by applying strong construction adhesive or only a small number of steel dowels at the contact surfaces during retrofit work. The primary connection is achieved by the passive confinement actions, activated when the external loads are applied.

Compared to the traditional method, the connections in these proposed systems are easier to achieve. Additionally, improvements in the punching shear capacity are expected, as the shear transfer actions are enhanced.

1.3 Research Objectives

The main objectives of this research are as follows:

- Introduce five footing strengthening systems, including design details and construction processes;
- Investigate the effectiveness of the proposed retrofit systems on the improvement of punching shear capacity of reinforced concrete (RC) footings;
- Develop analytical models to predict the punching shear capacity of RC footings strengthened with the proposed systems.
1.4 Research Plan

Finite element analysis was applied in this research to investigate the five proposed strengthening systems, particularly on their effectiveness in improving the punching shear capacity of reinforced concrete (RC) footings. The commercial software ABAQUS was employed to build and analyze the finite element models. First and foremost, validations of the finite element modeling were accomplished by comparing the FEA results with the experimental results, and good agreements were achieved. Afterwards, a modeling plan was designed to direct the finite element analyses, in which both spread footings and pile caps were considered, and parameters to be investigated in each system were determined.

To understand the punching shear behavior of the original and strengthened footings, load-displacement curves generated from ABAQUS were studied. The curves of several representative models were displayed and discussed in this dissertation. The state of stresses on the contact surfaces was also studied to check the effectiveness of the connections. After that, for each system, a parametric study was carried out to investigate the influence of parameters on the punching shear capacity.

Finally, analytical models for calculating the punching shear capacities of footings strengthened with different systems were proposed, and all models were developed from the original analytical model adopted by Eurocode 2, with the critical section locating at \( d/2 \) to the edge of the column. For the CEP system, the factor \( k_i \) in the original model was replaced by factors \( \eta_{CEP, SF} \) and \( \eta_{CEP, PC} \), which considered the parameters including footing size, shear-span to depth ratio, flexural reinforcement ratio, eccentricity and number of prestressing strands. For the CERR system, a factor \( \eta_{CERR} \) was introduced to represent the improvement in punching shear capacity, considering the footing size and the regular reinforcement ratio. For CEBW, CECW and CESJ...
systems, a factor $\eta_{CEWJ}$ was introduced to represent the improvement, considering footing size, thickness and stiffness of strengthening material.

1.5 Outline

In Chapter 1, a brief introduction to this research was presented, and the existing issues in the traditional footing enlargement method were addressed. The five new strengthening systems were briefly introduced. The objectives, plan, and outline of this research were also stated.

In Chapter 2, the literature relevant to this research was reviewed. The structural deficiencies in existing RC footings, and the current retrofit strategies were introduced. The punching shear theory was discussed, with both mechanical models and empirical models being addressed. The experiments and the finite element analysis on RC footings were presented. The applications of external prestressing, FRP wraps, and steel jackets in structural strengthening were also reviewed.

In Chapter 3, for each strengthening system, details and construction process were elaborated. The advantages and limitations of each proposed system were also discussed.

In Chapter 4, material properties and element types employed in finite element models were identified. Boundary conditions and loading pattern were also introduced. Finite element modeling was validated by comparing the FEA results with available experimental data.

In Chapter 5, two matrixes were provided to present all finite element models investigated in this research. For the five proposed systems, different sets of parameters were considered. The influences of parameters on punching shear capacity of strengthened footings were discussed based on the parametric study conducted in each system. Punching shear behaviors of the strengthened footings and the states of Max. principal strains at the contact surfaces were also investigated.
In Chapter 6, the empirical models in ACI 318-14, AASHTO Code 2012, and Eurocode 2, and the mechanical model in Model Code 2010 were reviewed. The FEA results of the original spread footings were compared to those estimated by each model. The empirical model in Eurocode 2 with \( (a_{cr}=d/2) \) was determined to serve as the fundamental analytical model, due to the good agreements with FEA results.

In Chapter 7, developed analytical models for RC footings strengthened with the five systems were proposed. A factor \( \eta \) was introduced to each system to account for all parameters investigated in Chapter 5, and the formula to determine the factor \( \eta \) was derived by linear regression analysis.

In Chapter 8, the summary and conclusions drawn from this research were presented. Limitations and recommendations for future studies were also addressed.
2 LITERATURE REVIEW

2.1 Introduction

Bridge footing plays an essential role in transferring the load from the superstructure to the soil underneath. Footings with insufficient load-carrying capacity or structural deficiencies endanger the performance of the whole bridge. Thus, efficient strengthening is essential.

Similar to reinforced concrete (RC) slabs, punching shear failure is the dominant failure mode in RC footings. In most concrete design codes, such as ACI 318-14 (2014), AASHTO Code (2012), Eurocode 2 (2004) and Model Code 2010 (2012), the design of RC footings obeys the design provisions of RC slabs, with no distinctions being made between them (Hegger et al. 2009). However, compared to RC slabs, bridge footings typically have larger depth, smaller shear-span to depth ratio, and a relatively larger flexural reinforcement ratio. In terms of these differences in nature, it is questionable whether the punching shear provisions for RC slabs in the codes can be directly applied to bridge footings.

In this chapter, deficiencies of bridge footings are identified, and the traditional footing retrofit methods are reviewed. Given that punching shear failure is the dominant failure mode of RC footings, the relevant theories, experiments, and provisions in different codes are discussed. Afterwards, the experiments involving external prestressing, FRP wrapping, and steel jacketing are presented. Finally, finite element models of RC slabs and RC footings are reviewed.

2.2 Structural Deficiencies in Existing RC footings

Visual inspection of a structurally deficient bridge footing is not easy, as excavations above and around the footing are required. However, tilting of a pier, flexural cracking of the column, sloughing of the fill around the footing, or pulling away of the fill from the footing could be signs that the footing is deteriorated (Aboutaha et al. 2013).
The general structural deficiencies occurring in RC footings can be summarized as follows:

a) **Inadequate shear strength.** Punching shear failure is the dominant mode of failure of bridge footings, occurring when the punching shear capacity of the footing is inadequate;

b) **Inadequate flexural strength.** Footings are deficient in positive moment capacity when the bottom reinforcements are insufficient. Under seismic loads, footings without top reinforcements may also be deficient in negative moment capacity.

c) **Poor column-footing connection.** The connections between columns and footings are often vulnerable due to large shear force and moment, especially when spirals or hoops in the column were not extended into the footing. Local failure of connection usually results in the collapse of the entire bridge, especially under seismic loads.

d) **Inadequate axial load carrying capacity.** It happens when the bearing capacity of the soil underneath is insufficient. For spread footings, the primary types of bearing capacity failures include general shear failure, local shear failure, and punching shear failure.

Among them, punching shear failure is the dominating structural failure type.

### 2.3 Traditional Strengthening Methods for RC Footings

Compared to the numerous researches carried out on RC slabs, less attention has been focused on RC footings. The available researches about footing retrofit are also limited.

McLean *et al.* (1995) retrofitted one pile cap and one spread footing with different methods. The two retrofitted pile caps are shown in Figure 2-1. In Figure 2-1(a), the depth of footing was increased by a reinforced concrete overlay, and steel dowels were installed to connect the new
concrete segment to the existing footing. In addition to that, the plan dimensions of the pile cap in Figure 2-1(b) were enlarged, and the connection was achieved by splicing the reinforcing steels at the bottom. Test results indicated that the depth thickening strategy was effective in enhancing both bending and shear capacities of the pile cap, and the footing enlargement strategy could improve its overturning capacity.

![Figure 2-1 Footing retrofit methods investigated by McLean et al. (1995)](image)

(a) Increasing depth
(b) Increasing depth & plan dimensions

Similar retrofit methods were also investigated by Xiao et al. (1996), including increasing footing depth with a reinforced concrete overlay, and enlarging the plan dimensions. The connections at the horizontal interface and the vertical interfaces were achieved by installing “L” shaped dowels and inclined straight dowels, respectively (Figure 2-2). Moreover, they investigated the length of the “L” shaped dowels at the horizontal interface. Increasing the length of the steel dowels increased the flexural capacity.
Saiidi et al. (2001) employed similar methods to retrofit a spread footing. Both vertical and inclined straight dowels were used to connect the existing and additional concrete (Figure 2-3). Test results indicated that the maximum bearing pressure was reduced, and both bending capacity and shear capacity were improved.

Zumrawi & Aldaw (2018) also applied similar sectional enlargement methods to strengthen the footings of a building after the story extension. After excavating the soil around the original footing, small holes were drilled on its top and side surfaces, with epoxy being grouted. Then
dowel bars were inserted into the drilled holes, as shown in Figure 2-4.

![Figure 2-4 Drilled holes and inserted dowels bars (Zumrawi & Aldaw 2018)](image)

Additional reinforcements were placed surrounding the excavated footing. Both the depth and plan dimensions were increased by casting additional plain concrete (Figure 2-5).

![Figure 2-5 Placing additional reinforcements and casting additional concrete (Zumrawi & Aldaw 2018)](image)

In Caltrans’s “Example Seismic Retrofit Details” (2008), the example shown in Figure 2-6 illustrates the general retrofit strategies for a typical pile cap, including enlarging the footing depth...
and plan dimensions, adding additional piles, and adding a reinforced concrete overlay. Similarly, the connection between the existing and additional concrete is achieved by installing dowels.

FEMA 547 (2006) proposed three types of strategies to retrofit a spread footing, regarding different deficiencies. For a footing with deficient overturning capacity, its plan dimensions can be enlarged, and piles can also be added. Shear capacity can be enhanced by increasing the depth of the footing, or installing additional vertical bars to act as additional shear reinforcements. Insufficient bending capacity can be improved by adding a reinforced concrete overlay (Figure 2-7(a)). Similarly, the connections at the interfaces are built by installing vertical and inclined straight dowels (Figure 2-7(b)).

To sum up, general strengthening methods for a RC footing are as follows:
• **Enlarging plan dimensions.** It can significantly improve the axial load-carrying capacity of the footing;

• **Increasing depth.** It is usually achieved by adding a plain or reinforced concrete overlay, which is effective in improving both shear capacity and bending capacity of the footing;

• **Building up the connection.** Connections at the interface between the existing and additional concrete are built by installing a large number of steel dowels. The vertical dowels are applied at the horizontal interface, while the horizontal dowels are employed at the vertical interface. In most cases, the existing and additional flexural reinforcements are also spliced at the bottom of the footing.

### 2.4 Punching Shear Theory

Punching shear failure is a brittle failure, and it generally occurs prior to the yielding of the flexural reinforcements. The punching shear failure of a local member, such as the slab-column connection or the RC footing, may result in a progressive collapse of the whole structure. Figure 2-8 displays the punching shear failures of a RC slab (a) and a RC footing (b).

![Figure 2-8 Punching shear failures of (a) a RC slab and (b) a RC footing (Kueres et al. 2017)](image-url)
2.4.1 Shear Transfer Actions

For structural members without shear reinforcements, the shear transfer actions summarized by CEB-fib (2001) were distinguished by the cracked tension zone and the compression zone:

The shear transfer actions in the cracked tension zone include:

- Interface shear transfer, also called "aggregate interlock" or "crack friction".
- Dowel action of reinforcements across the cracks;
- Residue tensile stresses transmitted directly across the cracks;
- Cantilever action of concrete teeth.

The shear transfer actions in the compression zone include:

- Shear stresses in the uncracked concrete;
- Arch action.

Most of the shear transfer actions in the cracked tension zone are directly related to the concrete tensile strength, such as the cantilever action, the dowel action, and the residue tensile stresses across cracks.

2.4.2 Mechanical Models of Punching Shear

The first mechanical model of punching shear was proposed by Kinnunen & Nylander in 1960. Since then, different mechanical models have been proposed and developed. However, none of them have been universally accepted because neither could they accurately estimate the punching shear capacity, nor do they thoroughly consider all the relevant parameters. In most of the mechanical models, punching shear is simply defined as a behavior in which shear forces were transferred by concrete and steel on the inclined punching crack surfaces. Although this is a good strategy to simplify the complexity in punching shear, its accuracy is still questioned as the shear
forces transferred in adjacent regions are ignored.

Based on the differences in the flow of forces and the failure causes, CEB-fib (2001) classified the mechanical models into different types:

- **Flexural capacity approach**, such as Moe (1961). Some early models for punching shear are derived from the models for flexural capacity, because in those tests, the ultimate loads are close to the flexural capacities;

- **Plasticity Approach**, such as Bresnup et al. (1976). This approach is generally applied to determine the upper bound value for the ultimate load, and it does not consider the influence of the flexural reinforcements;

- **The Kinnunen & Nylander approach**. This group of models are developed from the original one proposed by Kinnunen & Nylander (1960), which is also the first mechanical model for punching shear.

- **Failure mechanism approaches with concrete tensile stresses in failure surface**, such as Georgopoulos (1988). This approach is based on fracture mechanics. It is similar to the Kinnunen & Nylander approach, except for that the tensile stresses across the cracks are utilized.

- **Truss models or strut-and-tie models**, such as Alexander and Simmonds (1991). In this approach, shear transfer actions are described by concrete ties, and concrete cracking is simulated using the smeared crack model.

- **Fracture mechanics**. These models are based on fracture mechanics, and numerical analyses are often involved.

The original and developed mechanical models about Kinnunen & Nylander approach, the mechanical model considering tensile stresses, and the truss model are further reviewed in this
section.

### 2.4.2.1 Kinnunen & Nylander approach

Among all mechanical models, the model provided by Kinnunen and Nylander (1960) was proved to yield good agreement with test results. Therefore, it attracted much attention, with some developments being further made by researchers such as Hallgren (1996) and Muttoni (2008). This group of models have the following similarities:

- Defining the load-rotation relationship;
- Defining the failure criterion;
- Predicting the punching shear capacity by iteration of the two formulas.

The original model proposed by Kinnunen and Nylander was developed based on 61 tests on circular slabs with circular column stubs. In each slab, flexural reinforcements were designed to be axisymmetrically disposed. The test results indicated that the punching shear capacity decreased with the increasing rotation of the slab, as displayed in Figure 2-9.

![Figure 2-9 Load-rotation curves for tests by Kinnunen and Nylander (Muttoni 2008)](image)

The rigid segments outside the punching cone are assumed to be supported by a
“compression conical shell”, which extends from the column face to the base of the punching shear crack (Figure 2-10). The punching shear failure is defined when the tangential compressive strain of concrete reaches the failure strain (-1.96‰).

![Figure 2-10 Mechanical model proposed by Kinnunen and Nylander (CEB-fib (2001))](image)

The predicted ultimate load is obtained by solving the equilibrium equation $V_{u,c} = V_{u,s}$, using iteration. Where $V_{u,c}$ depends on the ultimate concrete compressive strength $\sigma_{cu}$, as expressed by Equation 2-1. $V_{u,s}$ depends on the yield strength of the flexural reinforcements $f_y$, as expressed by Equation 2-2.

\[
V_{u,c} = \kappa \cdot \pi \cdot \eta \cdot d^2 \cdot k_x \cdot \frac{\kappa}{\eta^2} \cdot \sigma_{cu} \cdot f(\alpha) \quad (Equation\ 2-1)
\]

\[
V_{u,s} = \kappa \cdot 4\pi \cdot \rho \cdot f_y \cdot d \cdot r_f \cdot \left[1 + \ln \left(\frac{\delta \cdot d}{2r_f}\right)\right] \cdot \frac{1-k_x}{\delta-\eta} \quad (Equation\ 2-2)
\]

However, the failure criterion in this model is given as semi-empirical equations, which was later modified by Hallgren in 1996.
In Hallgren’s model, the failure criterion is built based on a simple mechanical model, considering both the brittleness of concrete and the size effect. According to the results from finite element analyses, Hallgren stated that close to failure, the concrete near the column base was under triaxial compression. While the concrete at a distance $y$ to the column face was under a state of biaxial compression in the two horizontal directions and tension in the vertical direction (Figure 2-12). Therefore, concrete in this area would crack horizontally. When this crack occurs, the confinement near the column base is lost, resulting in the punching shear failure.
Based on that, the failure criterion is expressed as:

$$\varepsilon_{ct\nu} = \frac{3.6 \cdot G_F^\infty}{x f_{ct}} \left(1 + \frac{13 \cdot d_a}{x}\right)^{-\frac{1}{2}} \quad (\text{Equation 2-3})$$

Where $x$ is the depth of the compression zone; $d_a$ is the maximum aggregate size; and $G_F^\infty$ is given by:

$$G_F^\infty = G_F \left(1 + \frac{13 \cdot d_a}{d_R}\right)^{-\frac{1}{2}} \quad (\text{Equation 2-4})$$

Based on the equilibrium of moments and forces, the applied load $P$ can be expressed by the following two equations:

$$P = \frac{(2\pi R_{st} + R_{sr})(d-x) + D \left(c_a - \frac{B}{2} - x\right) + \left[R_{st} + 2\pi (R_{st} - R_{ct})\right] x + 2\pi R_{ct} \cdot 2\pi}{0.5(c-B) - x} \quad (\text{Equation 2-5})$$

$$P = [R_{sr} + 2\pi (R_{st} - R_{ct})] \cdot \tan \alpha + D \quad (\text{Equation 2-6})$$

The ultimate applied load is obtained when the equilibrium of the two equations above is achieved. To solve the equilibrium equation, iteration is required, and the inclined angle $\alpha$ is used as the iteration variable.

Moreover, Muttoni and his co-workers (Muttoni 2008; Muttoni & Ruiz 2012; Ruiz et al. 2009) further developed this approach, and introduced the critical shear crack theory (CSCT). Their model is adopted by Model Code 2010, and it will be discussed in Chapter 6.

2.4.2.2 Georgopoulos model (1988)

The model proposed by Georgopoulos (1988) was developed to describe the punching shear behavior of flat slabs without shear reinforcements, in which the concrete tensile strength and the flexural reinforcement ratios are the two primary parameters.
In this model, 75% of the shear forces are assumed to be carried by the principal tensile stresses on the crack surface, and the rest is carried by the compressive stresses in the conical shell near the column (Figure 2-13). Therefore, the external load $P_u$ is expressed as:

$$P_u = \frac{Z_B \cos \theta}{0.75} \quad (Equation\ 2-7)$$

where, $Z_B$ is the tensile force on the crack surface; $\theta$ is the inclination of the crack, and it is related to the flexural reinforcement ratio $\omega$:

$$\tan \theta = \frac{0.056}{\omega} + 0.3 \quad (Equation\ 2-8)$$

The height of the compression zone is estimated as 1/5 of the effective depth, and a third-degree polynomial is assumed to describe the distribution of the tensile stress on the crack surface.

Figure 2-13 Load-bearing model proposed by Georgopoulos (CEB-fib (2001))

Figure 2-14 Distribution of concrete tensile stresses by Georgopoulos (CEB-fib (2001))
By integrating the tensile stress, with the equilibrium in the vertical direction, the final formula to determine the external load $P_u$ is:

$$P_u = 4.13 \cdot \sigma_1 \cdot h_e \cdot \cot \theta \cdot \left( \frac{2}{z} + 0.20 + 0.35 \cdot \cot \theta \right)$$  \hspace{1cm} (Equation 2-9)

The results predicted by this model was proved to yield a good agreement with the experimental data, as shown in Figure 2-15.

![Figure 2-15 Predicted results compared with the test results by Georgopoulos (CEB-fib (2001))](image)

2.4.2.3 Alexander and Simmonds model (1991) - truss model

In 1991, Alexander and Simmonds proposed a truss model, also known as the bond model, to describe the punching shear behavior. In their model, the flexural reinforcements are treated as tension ties, and the compression struts are assumed as curved, instead of straight and inclined in the traditional truss model (shown in Figure 2-16). Each radial strip extends from the column surface to the flexural reinforcements. The shear transfer actions are carried by the curved compression struts. The punching shear failure is defined when bond failure or yielding occurs in the flexural reinforcements.
The punching shear strength is obtained by summing the resistance of all radial strips. For a slab-column connection where four radial strips were assumed, the following equation was derived, and the flexural capacity of each radial strip is:

\[ P = \sum P_s = 4 \times P_s = 8\sqrt{M_s \times w} \]  

(Equation 2-10)

where, \( P_s \) and \( M_s \) are the shear capacity and flexural capacity of a radial strip, respectively; \( w \) is introduced as a loading term.

2.4.3 Empirical Models Adopted by Codes

In most design codes, such as ACI 318-14, AASHTO Code 2012, and Eurocode 2, the designs for punching shear of RC slabs or RC footings are based on empirical models, instead of the mechanical models discussed previously. The empirical models are developed based on the observations from experimental studies, and the formulas for punching shear are derived by evaluating the experimental data using linear/non-linear and single/multiple regression analyses. Compared to mechanical models, the empirical models have the following limitations:

- No constitutive laws or failure criteria are presented;
- Considering global equilibrium only;
- Not considering the plastic behavior of concrete and reinforcing steels.

However, the primary advantage of these models is that they are more practical and easier
to be carried out. Empirical models in different codes are not the same, but they have two features in common:

- The critical section (also called control surface) is defined by specifying a distance to the edge of the column (Figure 2-17), but the value of the distance may vary in different codes;

![Figure 2-17 Control surface for punching shear](image)

- The nominal shear stress on the control surface is defined by a formula involving a group of relevant parameters, and the parameters considered vary in different codes.

For example, the flexural reinforcement ratio is involved in the provisions in Eurocode 2, whereas it is not involved in ACI 318-14.

The empirical models in ACI 318-14, AASHTO Code 2012, and Eurocode 2 will be further discussed in Chapter 6.

2.4.4 Experimental Data Compared to Codes

Early in 2001, a technical report published by CEB-fib (2001) comprehensively compared the provisions for punching shear capacity without shear reinforcements specified by different codes, including ACI 318-95, Eurocode 2, and Model Code 90 (Figure 2-18). Based on this comparison, the punching shear capacity estimated by Eurocode 2 is the most conservative. Model Code 90 results in the highest maximum shear capacity when stud-rail (Figure 2-19) is considered.
ACI 138-95 determines a relatively higher estimated shear capacity when shear reinforcements are not considered.

![Figure 2-18 Maximum punching shear capacity versus concrete compressive strength for different codes (CEB-fib 2001)](image)

Figure 2-18 Maximum punching shear capacity versus concrete compressive strength for different codes (CEB-fib 2001)

![Figure 2-19 Stud rails (DECON® Studrails®)](image)

Figure 2-19 Stud rails (DECON® Studrails®)

For RC footings, Siburg et al. (2014) compared the test results from 32 column footings with the punching shear capacities predicted by Eurocode 2 and Model Code 2010 (CSCT). The comparisons indicated that the punching shear capacity predicted by Eurocode 2 is conservative, while the predictions from Model Code 2010 (LoA III) agreed well with the test results. Bonić &
Folić (2013) investigated the predictions made by Eurocode 2 and ACI 318-14, indicating the two codes both gave conservative results. However, the predictions from Eurocode 2 were more rational.

For RC slabs, relatively more comparisons have been made. Ricker and Siburg (2016) investigated the punching shear capacities specified by Model Code 2010 and Eurocode 2, based on the experimental results from several RC slabs with and without shear reinforcements. For the RC slabs without shear reinforcements, predictions from Eurocode 2 and Model Code 2010 (LoA II) both have good agreements with the test results. For the RC slabs with shear reinforcements, the predictions by Eurocode were significantly larger than those by Model Code 2010. Ferreira et al. (2014) carried out tests on RC slabs reinforced with double-headed studs, and they compared the results with the predictions from ACI 318-14, Eurocode 2, and the critical shear crack theory (CSCT). The comparisons indicated ACI 318-14 performed best in estimating the punching shear capacity, with only one unsafe prediction being obtained. Relatively more unsafe predictions were made by both Eurocode 2 and CSCT.

2.5 Effects of Parameters on Punching Shear Capacity of RC Footings

Relatively fewer tests were carried out on RC footings, compared with those on RC slabs. Among them, supporting footings on a soil surface or a sandbox is the most realistic test setup, and the redistribution of soil pressure underneath the footings can also be investigated. Table 2-1 summarizes the available experimental results about RC footings from the literature. Four types of test setups are distinguished:

- RC footings are supported on a soil surface (Figure 2-20 (a)) or a sandbox (Figure 2-20 (b)), with an axial load being applied at the column stub.
RC footings are supported by a rectangular or circular steel frame, with an axial load being applied at the column stub (Timm (2003), Hallgren et al. (1998));

- RC footings are supported on the column stub, with a battery of small hydraulic jackets being used to simulate a uniform surface load on the surface of the footing (Dieterle & Rostasy (1987), Kordina & Nölting (1981), Dieterle & Steinle (1981), Rivkin (1967));

- RC footings are supported by a bed of steel springs, with a hydraulic jack being used to apply an axial load at the column stub (Richart (1948), Talbot (1913)).

Table 2-1 Overview of tests on RC footings from literature

<table>
<thead>
<tr>
<th>Authors (Year)</th>
<th>Support type</th>
<th>No.</th>
<th>Geometry of footing</th>
<th>Shape</th>
<th>Dimension (mm)</th>
<th>Effective depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Folić et. al. (2013)</td>
<td>Gravel Surface</td>
<td>6</td>
<td></td>
<td>Square</td>
<td>850</td>
<td>100 to 175</td>
</tr>
<tr>
<td>Hegger et. al (2009)</td>
<td>I: Sandbox II: Column stub</td>
<td>17</td>
<td></td>
<td>Square</td>
<td>1200 to 1800</td>
<td>250 to 470</td>
</tr>
<tr>
<td>Hegger et. al (2006)</td>
<td>Sand box</td>
<td>5</td>
<td></td>
<td>Square</td>
<td>900</td>
<td>150 to 250</td>
</tr>
</tbody>
</table>
Based on the punching shear tests of RC footings, the following parameters were generally considered as related with the punching shear capacity:

- Shear-span to depth ratio $a/d$;
- Flexural reinforcement ratio $\rho_{flex}$;
- Shear reinforcements (amount and layout);
- Compressive strength of concrete $f_c$.

### 2.5.1 Shear-span to Depth Ratio

RC footings investigated by Hegger and co-workers (2009, 2006, 2014) accounted for different shear-span to depth ratios $a/d$, with a range from 1.2 to 2.5. The results indicated that the footing with a smaller shear-span to depth ratio would have a larger failure load (Figure 2-21).
Shear-span to depth ratio $a/d$ also has a significant influence on the inclination of critical shear crack. In tests completed by Hegger et al. (2009, 2006), RC footings with and without shear reinforcements were investigated (Figure 2-22).

For footings without shear reinforcements, the inclination angle was almost 45 degrees for...
the compact footings \((a/d = 1.25)\), and less than 35 degree for the slender footings \((a/d = 2.0)\). For footings with shear reinforcements, the inclinations of critical shear cracks were much steeper, with a range approximately from 50 to 60 degrees. Additionally, for the specimens without shear reinforcements, the effect of the shear-span to depth ratio on the inclination angle was more significant.

In tests conducted by Hallgren \textit{et al.} (1998) and Folić \textit{et al.} (2013), the results also showed that with the same \(f'_c\), decreasing \(a/d\) increased the failure load.

2.5.2 Flexural Reinforcement Ratio

Hallgren \textit{et al.} (1998) completed punching shear tests on 14 RC slabs with a small shear-span to depth ratio 1.2, and investigated three flexural reinforcement ratios 0.25, 0.40, and 0.65. According to the results displayed in Figure 2-23, the higher the flexural reinforcement ratio, the larger the punching shear capacity.

![Figure 2-23 Normalised punching shear strength \(V_{nt}\) as a function of the ratio of reinforcement \(\rho\)](image)

Also, in their tests, three types of end anchorage of the flexural reinforcements were investigated:

- Flexural reinforcements with hoops at the ends (Figure 2-24 (a));
- Flexural reinforcements with bent-up at the ends (Figure 2-24 (b)); and
- Flexural reinforcements with straight anchorages at the ends (Figure 2-24 (c)).

Figure 2-24 Flexural reinforcements with different types of anchorage (Hallgren et al. 1998)

The results indicated the effect of the anchorage types on the punching shear capacity was not significant.

Based on experimental results, Hegger et al. (2009, 2006) also came up with the same conclusion that increasing the flexural reinforcement ratio increased the ultimate load. In addition, they compared the ultimate loads obtained from tests with those estimated by ACI 318-02 (2002) (Figure 2-25(a)) and Eurocode 2 (Figure 2-25(b)), regarding different flexural reinforcement ratios.

Figure 2-25 Test results compared to ACI 318-02 and Eurocode 2, regarding different flexural reinforcement ratios
As flexural reinforcement ratio is considered in Eurocode 2, but not in ACI 318-02, the scattering of the data obtained from Eurocode 2 is less, and the coefficient of variation is smaller ($\nu = 0.22$).

### 2.5.3 Shear Reinforcements

Shear reinforcements can significantly improve the punching shear capacity of RC footings. In the tested conducted by Hallgren *et al.* (1998), three types of shear reinforcements were investigated:

- Vertical stirrups Figure 2-26(a);
- Shear reinforcements with an angle of 58° bent-up only Figure 2-26(b); and
- Shear reinforcements with both an angle of 58° bent-up and bent-down at the ends Figure 2-26(c).

The results indicated the punching shear capacity of footings with shear reinforcements were about 35% to 55% larger than the those without shear reinforcements. Shear reinforcements with inclined bent-ups (b and c) increased 15 percent more on punching shear capacity, compared with the vertical stirrups (a). However, the difference between the bent-down end (c) and the straight end (b) was not significant.

![Figure 2-26 Column footings with different types of shear reinforcements (Hallgren *et al.* 1998)](image)

Tests conducted by Hegger *et al.* (2009, 2006, 2014) also revealed the application of shear reinforcements improved the punching shear capacity of footings with different shear-span to
depth ratios, and the improvements increased with increasing $a/d$, as shown in Figure 2-27. However, the influence of shear reinforcements on the punching shear behavior was not obvious.

![Figure 2-27 Punching shear capacities of footings with shear reinforcements $V_{\text{Test}}$ compared with those without shear reinforcements $V_{\text{c,Test}}$ (Siburg & Hegger, 2014)](image)

**2.5.4 Concrete Compressive Strength**

Different concrete compressive strengths were studied in the tests carried out by Hegger *et al.* (2009), with a range from 19.0 MPa to 38.1 MPa. The results obtained from the footings with $\rho_{\text{flex}} = 0.87\%$ and different concrete compressive strengths are plotted in Figure 2-28.

![Figure 2-28 Influence of concrete compressive strength on failure loads (Hegger *et al.* 2009)](image)

For slender footings ($a/d = 1.5$ and $2.0$), the concrete compressive strength had a significant
influence on the failure load: the higher the concrete compressive strength, the larger the failure load. While for compact footings with $a/d = 1.25$, the influence was relatively slight.

In tests conducted by Folić et al. (2013), four concrete compressive strengths were investigated: 38.37 MPa, 21.25 MPa, 19.29 MPa, and 10.0 MPa. With the same shear-span to depth ratio, the higher the compressive strength, the higher the failure load.

2.6 Structural Strengthening by External Prestressing

Horizontal prestressing strands were suggested by FEMA 547 (2006) to strengthen bridge footings (Figure 2-29).

![Figure 2-29 Footing enhanced by horizontal prestressing (FEMA 2006)](image)

However, this strategy is not practical to be implemented, as drilling a horizontal hole all through the original footing is rather troublesome. The hole must be well-oriented and shaped, and attention must be paid to not damage the existing reinforcements.

Mostafaei et al. (2011) tested seven slabs strengthened by external prestressing (Figure 2-30). Four of the slabs were constructed using fiber reinforced concrete, and the remaining there were constructed using plain concrete. None of the slabs contained regular reinforcements. All specimens failed in punching shear, except Specimen P-3, which failed in punching-flexural. The results indicated that the axial compressive stress in the strengthened slabs significantly improved.
the punching shear capacity. The level of prestressing also influenced structural behavior, including cracking, deflection, stiffness, and failure mode. The increasing prestressing force would result in a larger ultimate load, but a smaller ductility.

Figure 2-30 Details of test specimens and post-tensioning system (Mostafaei et al. 2011)

**2.7 Structural Strengthening by FRP Wrapping or Steel Jacketing**

External confinement by using either FRP wrapping or steel jacketing has been widely applied in the strengthening of RC structural members, such as RC beams and columns, to improve their strength and ductility. For an RC column strengthened by the steel jacket, the confinement action can be illustrated by Figure 2-31. Under a combination of axial compression and bending moment, the column has a tendency of dilation, which is restrained by the radial stiffness of the external steel casing, resulting in a circumferential tension in the casing and a radial compression in the concrete. As compressive strength of concrete is enhanced, flexural strength and ductility of the column are improved.
2.7.1 Steel Jacketing

Strengthening structural members with steel jacketing was first introduced by Chai et al. (1991) on concrete columns, to strengthen the potential plastic region. The size of the cylindrical steel jacket was made slightly larger than the size of the column, and the gap was filled with a cement-based grout (Figure 2-32(a)). The result indicated that the steel jacket effectively enhanced the flexural capacity and ductility of the tested columns. Also, the stiffness of the column was increased by 10% to 15%.

Choi et al. (2009) pointed out that grouting and curing the concrete in the gap was not convenient, and put forward an upgraded method where steel jackets were attached to the column
surfaces by external lateral pressure (Figure 2-32(b)). Single-layered jacket and double-layered jacket with the same thickness were investigated. The result indicated that the upgraded method enhanced the displacement ductility of RC footings, and the double-layered jacket had a better performance.

2.7.2 FRP Wrapping

FRP wraps are easier to be installed, compared to steel jacketing. However, the price of FRP wraps is also higher. Pham et al. (2015) and Chastre & Silva (2010) tested circular columns wrapped with FRP under axial compression, by which considerable enhancements on strength and ductility of the column were observed. The more layers of FRP wraps were applied, the larger the improvement would be. In addition, columns with larger diameters would have a significant reduction on the enhanced compression strength, compared with smaller ones (Chastre & Silva 2010).

Figure 2-33 Details of the two groups of specimens tested by Chastre & Silva (2010)

Maaddawy (2009) investigated eccentrically loaded columns wrapped with FRP, showing that the strength improved using FRP wrapping would decrease as the eccentricity was increased. Pham et al. (2015) and Maaddawy (2009) studied the effect of wrapping schemes on compressive behavior of the columns (Figure 2-34). The results indicated that the columns with full wrappings would have a larger improvement on compression strength and ductility of the column.
Zeng et al. (2017) and Hadi et al. (2012) circularized rectangular or square columns before the FRP jacketing was installed (Figure 2-35), concluding that section circularization could significantly improve the effectiveness of the FRP confinement. Strength and ductility of the column were also increased.

2.8 FEA Models of RC slabs and Footings

Formulas for the design of RC structures are typically derived from numerous experiments,
which are generally time-consuming and labor-intensive. With the help of finite element analysis, experiments can be carried out virtually. It has been proved that the finite element analysis can make good predictions when the inputs are accurate. In 1967, Ngo and Scordelis published their work where the finite element method was applied to analyze several simply supported reinforced concrete beams (Ngo & Scordelis 1967). In their research, concrete, reinforcing steel, and the bond links between the concrete and steel were simulated by finite elements. The concrete cracks were also modeled, but the propagation of cracks was not involved. It was the first publication presenting the finite element analysis about RC structures. Ever since, the finite element method has enjoyed extensive interests, with numerous researches being carried out.

Hallgren & Bjerke (2002) completed the punching shear tests on two circular spread footings S12 (34.1 MPa) and S13 (24.7 MPa), and the influence of the compressive strength of concrete was investigated. The typical profile of the shear cracks is shown in Figure 2-37(a). A special-purpose computer program SBETAX 1.2 was then employed to carry out the 2D finite element analysis, and the results are displayed in Figure 2-37(b). The comparison between test results and numerical results showed good agreement.
In their research, the influence of the shear-span to depth ratio was also investigated, and the results are shown in Figure 2-38.

Based on the comparison, decreasing the shear-span to depth ratio increased the failure load, but decreased the ultimate displacement.

Most commercial finite element software originated in the 1970s. Among them, ABAQUS has been proved to be a powerful software on the simulation of the RC structures. Genikomsou & Polak (2015) employed ABAQUS to conduct finite element analysis on five RC slabs without shear reinforcements. The validation of finite element models was completed by comparing the FEA results with available test results, with good agreements being achieved. The cracking pattern
was visualized by the maximum principal plastic strains (Figure 2-39).

Figure 2-39 Cracking pattern at failure load shown by the maximum principal plastic strains (Genikomsou & Polak 2015)

Nana et al. (2017) carried out both experimental and numerical investigations of ten RC slabs, and all the finite element models were built using ABAQUS. The finite element modeling was first calibrated using experimental results. Afterwards, models with different parameters including slab depth, concrete aggregate size, longitudinal reinforcements, and transverse reinforcements were investigated. By carrying out finite element analyses, failure load of each slab was obtained, and the punching shear cracks were also identified by the maximum principal plastic strains (Figure 2-40).

Figure 2-40 Cracking pattern shown by maximum principal plastic strain (Nana et al. 2017)
3 PROPOSED STRENGTHENING SYSTEMS FOR RC FOOTINGS

3.1 Introduction

As reviewed previously, enlarging the plan dimensions of a RC footing is effective in improving its axial load-carrying capacity, especially when the bearing capacity of the soil underneath is inadequate. In the traditional footing enlargement method, connections at the interfaces between the existing and the additional concrete segments are built by installing considerable steel dowels and splicing reinforcing steels. However, as this type of connection is troublesome to build, the traditional method is not practical.

In this dissertation, five systems were proposed to upgrade the traditional footing enlargement method:

- Circular external prestressing (CEP) system;
- Circular external regular reinforcement (CERR) system;
- Circular external BFRP wrapping (CEBW) system;
- Circular external CFRP wrapping (CECW) system;
- Circular external steel jacketing (CESJ) system.

Compared to the traditional method, connections at the contact surfaces in these systems are primarily achieved by the confinement actions. It is worth mentioning that in this dissertation, only square footings are considered.

The compressive strength of the new concrete should be equal to or higher than that of the original concrete. Also, self-compacting concrete with small aggregates is suggested to be used to improve the workability.

3.2 Preliminary work

For each system, the following construction steps are required to be carried out before the
strengthening work starts:

- Live loads on the structure should be removed, to whatever extent is practical;
- Fills above and surrounding the existing footing need to be excavated.

3.3 The CEP System – Footings Strengthened with Circular External Prestressing

**Strands**

The CEP system utilizes an unbonded post-tensioning system, where circular external prestressing strands are installed inside the additional concrete segment (Figure 3-1). The connection at the contact surface is built by composite actions, consisting of:

- Tension force resisted by regular reinforcing steels at the top;
- Compressive force at the bottom provided by prestressing;
- Friction.

The CEP is an active system, as the connection has already been achieved during the construction process (after post-tensioning), before the external load is applied.

**3.3.1 Details and Construction Steps**

Details of the CEP system are shown in Figure 3-1. In order to reduce the friction between strands and duct, pre-greased prestressing strands are employed, as shown by the blue circle. Details of a pre-greased strand are displayed in Figure 3-2. The eccentricity and the amount of prestressing force (presented by the number of prestressing strands) will be investigated, as they are expected to have significant effects on the failure load and the structural behavior of the strengthened footings.
The near-surface regular reinforcing steels are embedded in the grooves on the top surface, as shown by the red straight lines, and the purpose of these reinforcements is to offset the tension force acting at the top of the contact surface, induced by the moment during the tensioning step. Therefore, the number of the grooves and the area of the reinforcing steels should be determined based on the prestressing force applied in the strengthening system.

The construction steps to implement the CEP system are as follows:

1. Cutting grooves on the top surface of the original footing for the installation of near-surface reinforcements;
2. Cutting notches at four corners;
3. Installing formwork, and placing pre-greased prestressing strands inside the notches;
4. Placing near-surface reinforcements inside the grooves;
5. Casting additional concrete segments;
6. Tensioning and anchoring the strands after concrete gains its strength, and;
7. Filling in the grooves with high strength concrete (HSC).

3.3.2 Anchorage System

The buttresses anchorage system suggested by VSL (1991) is employed to anchor the circular external prestressing strands. This anchorage system has been commonly applied in the storage tank, as shown in Figure 3-3. One circle of prestressing is composed of six different strands crossing at six buttresses, and the length of each strand is one-third of the circumference of the tank. By using this anchorage system, friction losses can be reduced effectively.

Figure 3-3 Typical buttresses in post-tensioning system (VSL, 1991)
3.3.3 Advantages and Limitations

As the area of the footing is enlarged, the CEP system is effective in improving the load-carrying capacity. Punching shear capacity is also expected to be improved, because shear transfer actions should be enhanced by the compressive force provided by the circular external prestressing strands.

Connections in this system are built by composite actions on the contact surfaces, instead of installing steel dowels or splicing the reinforcements. Therefore, the workload is reduced. Damages in existing reinforcing steels are avoided. The soil underneath the edge of the existing footing is not disturbed.

However, the costs of carrying out the CEP system may be higher, as the post-tensioning system is involved. Experienced engineers and skilled workers are required. Corrosion protection of the post-tensioning system needs to be well designed.

3.4 The CERR System – Footings Strengthened with Circular External Regular Reinforcements

In the CERR system, circular external regular reinforcements are employed (Figure 3-4), in place of the prestressing strands used in the CEP system.

During construction, strong construction adhesive or a small number of steel dowels are applied at the contact surfaces, serving as the secondary connection. After the construction is completed and the external load is applied, the primary connection is achieved by the confinement action provided by the circular external regular reinforcements. It is regarded as a passive system, since the primary connection is activated after the load is applied.

Under external loads, the connections at the contact surface consist of:

- Connections provided by construction adhesive or a small number of steel dowels
(secondary connection);

- Confinement action provided by circular external regular reinforcements (primary connection).

### 3.4.1 Details and Construction Steps

The CERR system is different from the CEP system in the following aspects:

- The near-surface regular reinforcements are not required. Instead, construction adhesive or a small number of steel dowels needs to be applied on the contact surface;
- Post-tensioning system is not required. Instead, regular reinforcing steels are placed at the same locations as the prestressing strands.

The area of the circular reinforcing steels will be investigated, as it is expected to affect the failure load and the structural behavior of the strengthened footings.

Figure 3-4 The CERR system– footings strengthened with circular external regular reinforcements
The construction steps to implement the CERR system are as follows:

1. Cutting notches at four corners;
2. Installing formwork, and placing circular external regular reinforcements inside the notches;
3. Cleaning contact surfaces;
4. Apply construction adhesive or installing steel dowels at contact surfaces;
5. Casting additional concrete segments.

3.4.2 Advantages and Limitations

As the area of the footing is enlarged, the CERR system is also effective in improving the axial load-carrying capacity of the footing. The enhancement of the punching shear capacity by using this system is also expected.

Compared to the traditional footing enlargement method, the workload is reduced. Only construction adhesive or a small number of steel dowels are applied at the contact surface.

Compared to the CEP system, the cost of the CERR system is less. However, as a passive system, the connection is weaker than that in the CEP system, and the integrity of the enlarged footing is relatively worse.

3.5 The CEBW / CECW System – Footings Strengthened with Circular External CFRP / BFRP wraps

In the CEBW / CECW system, circular external BFRP / CFRP wraps are installed on the exterior surface of the enlarged footing (Figure 3-5). Similar to the CERR system, they are also passive systems, as the primary connection is only activated when external loads are applied. There are two types of connections at the contact surfaces:

- During construction, a secondary connection is built by applying construction
adhesive or installing a small number of steel dowels;

- Under external loads, a primary connection is achieved by the confinement action provided by the external circular BFRP / CFRP wraps.

3.5.1 Details and Construction Steps

The CEBW / CECW system is different from the CERR system in the following aspects:

- Four corners are trimmed, instead of cutting notches in the CERR system, for the purpose of preventing the FRP wraps being damaged by the stress concentration;
- No reinforcements are placed inside the additional concrete segment. Instead, FRP wraps are installed on the exterior surface of the enlarged footing, in the direction that fibers orient along the circumference direction.

The thickness and the strengthening depth of FRP wraps will be investigated, as they are expected to affect the failure load and the structural behavior of the strengthened footings.

Figure 3-5 The CEBW / CECW system – footings strengthened with BFRP / CFRP wraps

The construction steps to implement the CEBW / CECW system are as follows:

1. Trimming four corners of the original square footing;
2. Installing formwork;
3. Cleaning contact surfaces;
4. Applying construction adhesive or installing steel dowels at the contact surfaces;
5. Casting additional concrete segments;
6. Installing FRP wraps.

3.5.2 Advantages and Limitations

As the area of the footing is enlarged, the CEBW / CECW system is also effective in improving the axial load-carrying capacity of the footing. The enhancement on the punching shear capacity by using this system is also expected.

Compared to the traditional footing enlargement method, the workload is reduced. Only construction adhesive or a small number of steel dowels are applied at the contact surface.

Compared to the CEP system, the cost of the CEBW / CECW system is less. However, as passive systems, the connection is weaker than that in the CEP system, and the integrity of the enlarged footing is relatively worse.

Compared to the CERR system, the cost of the CEBW / CECW system is higher. However, when the CFRP / BFRP wraps are applied at the full depth of the exterior surface. The connections are slightly stronger.

3.6 The CESJ System – Footings Strengthened with Circular External Steel Jackets

In the CESJ system, circular external steel jackets are installed on the exterior surface of the enlarged footing (Figure 3-6). Similar to the CERR system and the CEBW / CECW system, the CESJ is a passive system, as the primary connection is only activated when external loads are applied. There are two types of connections at the contact surfaces:

- During construction, a secondary connection is built by applying construction
adhesive or installing a small number of steel dowels;

- Under external loads, a primary connection is achieved by the confinement action provided by the circular external steel jackets.

### 3.6.1 Details and Construction Steps

The CESJ system is different from the CEBW / CECW system in the following aspects:

- Temporary formwork is not required. Instead, the steel jacket will serve as the permanent formwork;
- Instead of installing FRP wraps, steel jackets are applied on the exterior surface of the enlarged footing;
- Corrosion protection of the steel jackets must be considered.

The thickness and the strengthening depth of the steel jackets will be investigated, as they are expected to affect the failure load and the structural behavior of the strengthened footings.

![Figure 3-6 The CESJ system – footings strengthened with steel jackets](image)

The construction steps to implement the CESJ system are as follows:

1. Trimming four corners of the original square footing;
2. Placing the steel jackets;
3. Cleaning the contact surfaces;
4. Applying construction adhesive or installing steel dowels at the contact surfaces;
5. Casting additional concrete segments;
6. Applying corrosion protection.

### 3.6.2 Shear Studs

Shear studs are suggested to be welded on the steel jackets to enhance the stiffness and to improve the connection between steel jackets and concrete. The electric-arc stud welding is the most common process (Figure 3-7), in which the metal studs are joined into steel jackets by heating both parts with an electric arc. This process can provide a highly reliable connection and will not damage the steel jackets.

![Figure 3-7 Nelson stud welding process (Nelson® Stud Welding)](image)

### 3.6.3 Advantages and Limitations

As the area of the footing is enlarged, the CESJ system is also effective in improving the axial load-carrying capacity of the footing. The enhancement on the punching shear capacity by using this system is also expected.

Compared to the traditional footing enlargement method, the workload is reduced. Only construction adhesive or a small number of steel dowels are applied at the contact surface.

Compared to the CEP system, the cost of the CESJ system is less. However, as a passive
system, the connection is weaker than that in the CEP system, and the integrity of the enlarged footing is relatively worse.

Compared to the CERR system, the cost of the CESJ system is higher. However, when the steel jackets are installed at the full depth of the exterior surface, stronger connections at the contact surfaces can be expected.

Compared to the CEBW / CECW system, the cost of steel jackets is slightly lower. Besides, as the elastic stiffness of the steel jacket is higher, the confinement action is higher, and the connections should also be stronger.

However, in the CESJ system, corrosion protection of the steel jackets should also be considered, and that can be achieved by applying non-metallic or metallic coatings.

3.7 Summary and Conclusion

In this chapter, five upgraded footing enlargement systems (CEP, CERR, CEBW, CECW, and CESJ) were introduced. The CEP is an active system, and the other four are passive systems.

Traditional footing enlargement method is the most labor-intensive, as installing large amounts of steel dowels or splicing the existing reinforcements are troublesome.

The CEP system has distinct advantages and limitations. As an active system, a remarkable improvement in punching shear capacity can be expected, and the connections at the contact surfaces are the most stable. However, since the post-tensioning system and the buttresses anchorage system need to be built, this system is still somewhat labor-intensive. The cost is relatively high, and experienced labors are required.

The four passive systems (CERR, CEBW, CECW, and CESJ) have advantages in labor saving, lower cost, and ease of application. However, the confinement actions at contact surfaces are weaker, and less improvements in punching shear capacity can be expected.
As steel jacket can serve as permanent formwork, the workload of CESJ is relatively less, compared to the CEBW and CECW systems. Also, the cost of material in the CESJ system is relatively lower. However, corrosion protection for the steel jacket must be considered.

Compared to the traditional footing enlargement method, the primary advantage of these proposed systems is the improvement in building the connections at the contact surfaces between the existing and additional concrete segments. Additionally, by using the proposed systems, the punching shear capacity of the strengthened footing is expected to be improved, which will be further investigated in the following chapters.
4 FINITE ELEMENT MODELING

4.1 Introduction

In this research, finite element analysis was employed to investigate the effectiveness of the proposed systems on improving the punching shear capacity of RC footings, with FEA models being built using the commercial software ABAQUS. In this chapter, details of finite element modeling, such as material properties, element types and size, and boundary conditions, are introduced. Since the quality of inputs is critical to the accuracy of the simulation, the validation of finite element modeling was also accomplished.

4.2 ABAQUS\Explicit and ABAQUS\Standard

ABAQUS\Explicit and ABAQUS\Standard (implicit) are two commonly used analysis tools in ABAQUS. The ABAQUS\Standard (implicit) is unconditionally stable, while the ABAQUS\Explicit is conditionally stable. In both methods, incremental load (or displacement) steps need to be specified in advance, and the changing of the geometry or the material property is achieved by updating the stiffness matrix at the end of each increment.

4.2.1 ABAQUS\Standard (Implicit)

ABAQUS\Standard is based on Hilber-Hughes-Taylor time integration, which is an extension of the Newmark-\(\beta\) method with the advantage that the convergence can be improved by introducing the numerical damping without losing accuracy. It is an implicit analysis. At each increment, the operator matrix is inverted, and Newton-Raphson iteration is applied at the end to enforce equilibrium. Compared to ABAQUS\Explicit, the primary advantage of ABAQUS\Standard is that this method is unconditionally stable, so the size of increments does not influence the accuracy of the result.
4.2.2 ABAQUS\Explicit

In ABAQUS\Explicit, the central-difference time integration is employed to solve the equilibrium equations, and the recursive formulas are as follows:

\[
\hat{u}^N_{(i+1, \frac{1}{2})} = u^N_{(i-\frac{1}{2})} + \frac{\Delta t_{(i+\frac{1}{2})} + \Delta t_{(i)}}{2} \bar{u}^N_{(i)}
\]  
(Equation 4-1)

\[
u^N_{(i+1)} = u^N_{(i)} + \Delta t_{(i+1)} \bar{u}^N_{(i+\frac{1}{2})}
\]  
(Equation 4-2)

where \(u^N_{(i)}\) is the displacement (or the rotation) at the degree of freedom \(N\), and \(i\) is the label of the increment step. It is called explicit because the state of the new step \(u^N_{(i+1)}\) can be calculated using the known values \(\hat{u}^N_{(i-\frac{1}{2})}, \bar{u}^N_{(i)}\), and \(u^N_{(i)}\) obtained from the previous step. As it is conditionally stable, accurate results can only be obtained when the increments are small enough.

The stability limit for ABAQUS\Explicit follows:

\[
\Delta t \leq \frac{2}{\omega_{\text{max}}}
\]  
(Equation 4-3)

where, \(\omega_{\text{max}}\) is the highest frequency of the system. In ABAQUS, \(\Delta t\) is determined by:

\[
\Delta t \approx \frac{L_{\text{min}}}{C_d}
\]  
(Equation 4-4)

where, \(L_{\text{min}}\) is the dimension of the smallest element, and \(C_d\) is the dilatational wave speed.

Compared to the implicit analysis, the explicit analysis has remarkable advantages in solving problems with contact and material nonlinearities, and the convergence problems are avoided because the formation of the tangent stiffness matrix is not required. Therefore, it was employed in this research to investigate all models, where non-convergence would occur due to either the nonlinear contact (such as the contact between circular tendon and concrete), or the stiffness reduction during the punching shear failure.
4.3 Element Type and Material Property

4.3.1 Concrete

The first-order brick element C3D8 is commonly used to model solid members such as concrete. It has eight nodes at corners, with linear interpolation being applied in each direction. Therefore, it is also called a linear element. Compared to C3D20, where the quadratic interpolation is applied, the element C3D8 can significantly reduce computation time. However, it also has a notable drawback known as shear locking. Therefore, in this research, the 3D continuum element C3D8R with hourglass control was employed to simulate the concrete. It is an eight-node linear brick solid element with reduced integration (only one integration point in the center). Compared to C3D8, C3D8R can further save on computation time, and the shear locking effect can also be avoided.

![Figure 4-1 Element type adopted for concrete](image)

4.3.1.1 Stress-strain model for concrete in compression

For concrete in compression, the stress-strain relationship proposed by Yang et al. (2014) was adopted in this research (Figure 4-2).
The expression of the curve is:

\[
f_c = \left[ \frac{(\beta_1 + 1) (\frac{\varepsilon_0}{f'_c})}{(\frac{\varepsilon_0}{f'_c})^{\beta_1 + 1} + \beta_1} \right] f'_c
\]  
(Equation 4-5)

where, \( f'_c \) is the compressive strength of concrete, in this research, \( f'_c = 35 \) MPa; \( \varepsilon_0 \) is the strain corresponding to the compressive strength of concrete, and

\[
\varepsilon_0 = 0.0016 \cdot \exp \left[ 240 \left( \frac{f'_c}{f_0} \right) \right]
\]  
(Equation 4-6)

\( \beta_1 \) is the key parameter, and it is developed to determine the slope of both the ascending and descending branches, and

\[
\beta_1 = \begin{cases} 
0.2 \cdot \exp(0.73\xi), & \text{for } \varepsilon_c < \varepsilon_0 \\
0.41 \cdot \exp(0.77\xi), & \text{for } \varepsilon_c > \varepsilon_0
\end{cases}
\]  
(Equation 4-7)

The parameter \( \xi \) is introduced to simplify the \( \beta_1 \) equations, and

\[
\xi = \left( \frac{f_c}{f_0} \right)^{0.67} \left( \frac{w_0}{w_c} \right)^{1.17}
\]  
(Equation 4-8)

where, \( f_0 \) equals to 10 MPa, and \( w_0 \) equals to 2300 kg/m³.

4.3.1.2 Stress-strain model for concrete in tension

For concrete in tension, the stress-strain model proposed by Nayal & Rasheed (2006) was adopted (Figure 4-3). This model is upgraded from the initial model proposed by Gilbert and
Warner in 1978, and it considers the post cracking behaviours including tension softening, tension stiffening, and local bond-slip effects.

4.3.2 Prestressing Strands

4.3.2.1 Element type

The duct was simply modeled by removing a torus from the original footing (Figure 4-4(a)). The linear triangular prism element (C3D6) was employed to simulate the prestressing strands, which is a six-node wedge element with two integration points (Figure 4-4(b)). To model the strands, the cross-section is split into four quarters, and the mesh of the cross-section is shown in Figure 4-4(c).
4.3.2.2 Stress-strain model

All strands are Grade 270 seven-wire, low-relaxation steel strands. The stress-strain relationship followed the specifications in the PCI Design Handbook (2014), displayed in Figure 4-5. The effective prestressing stress $f_{pe}$ was considered as 1379 MPa (200 ksi), and the area of a single strand was 98.71 mm$^2$.

The curve can be described by the following equations:

$$ f_{ps}(ksi) = \begin{cases} 28800 \cdot \epsilon_{ps} & \epsilon_{ps} \leq 0.0085 \\ 270 - \frac{0.04}{\epsilon_{ps} - 0.007} & \epsilon_{ps} > 0.0085 \end{cases} $$  \hspace{1cm} (Equation 4-9)$$

$$ f_{ps}(MPa) = \begin{cases} 198569 \cdot \epsilon_{ps} & \epsilon_{ps} \leq 0.0085 \\ 1862 - \frac{0.276}{\epsilon_{ps} - 0.007} & \epsilon_{ps} > 0.0085 \end{cases} $$  \hspace{1cm} (Equation 4-10)
4.3.2.3 Prestressing modeling

Pre-greased strands in plastic sheathing were applied in this research, and a post-tensioning system was designed. Generally, two approaches are commonly applied to model the prestressing force in finite element analysis:

- Introducing the initial strain to the strands;
- Defining the initial temperature of the strands.

In this research, the latter approach was adopted, and the initial temperature was determined by the following equation:

\[ \Delta T = -\frac{P_e}{\alpha EA} = -\frac{f_{pe}}{\alpha E} \]  

*(Equation 4-11)*

where \( P_e \) is the effective prestressing force; \( f_{pe} \) is the effective prestressing stress; \( \alpha \) is the coefficient of linear expansion of steel and \( \alpha = 1.0 \times 10^{-5} \).

Therefore, \( \Delta T \) was determined to be \(-694 \, ^\circ C\).

4.3.3 Fiber Reinforced Polymer Composites (CFRP and BFRP)

CFRP and BFRP were modeled with continuum 2D shell elements (S4R), and they were modeled as linear elastic orthotropic materials (Figure 4-6). As FRP sheets were placed with their fibers oriented along the circumference direction, the elastic modulus in the circumference \( E_1 \) equals the modulus of FRP lamina, which can be obtained from a material test or calculated by using Equation 4-12, proposed by Agarwal & Broutman (1990). \( E_f \) and \( V_f \) are the elastic modulus and volume fraction of fibers, respectively. Similarly, \( E_m \) and \( V_m \) are the elastic modulus and the volume fraction of the epoxy matrix, respectively.

\[ E_1 = E_f V_f + E_m V_m = E_f \frac{A_f}{A} + E_m \frac{A_m}{A} \]  

*(Equation 4-12)*
The transverse elastic modulus and the shear modulus can also be calculated using:

\[
E_2 = E_3 = \frac{E_f E_m}{E_f \nu_m + E_m V_f} \tag{Equation 4-13}
\]

\[
G_{12} = G_{13} = \frac{G_f G_m}{G_f \nu_m + G_m V_f} \tag{Equation 4-14}
\]

And the Poisson’s ratios are calculated by:

\[
\nu_{12} = \nu_{13} = \nu_f V_f + \nu_m V_m \tag{Equation 4-15}
\]

where, \(G_f\) and \(\nu_f\) are the shear modulus and the Poisson’s ratio of the fibers, respectively. \(G_m\) and \(\nu_m\) are the shear modulus and the Poisson’s ratio of the epoxy matrix, respectively.

The material properties of BFRP applied in this research are the same as those tested by Mengal et al. (2014), and the material properties of CFRP followed those investigated by Obaidat et al. (2010), listed in Table 4-1.

<table>
<thead>
<tr>
<th></th>
<th>(E_1)</th>
<th>(E_2 = E_3)</th>
<th>(G_{12} = G_{13})</th>
<th>(G_{23})</th>
<th>(\nu_{12} = \nu_{13})</th>
<th>(\nu_{23})</th>
</tr>
</thead>
<tbody>
<tr>
<td>BFRP</td>
<td>37.7 GPa</td>
<td>5.237 GPa</td>
<td>2.05 GPa</td>
<td>3.63 GPa</td>
<td>0.2</td>
<td>0.21</td>
</tr>
<tr>
<td>CFRP</td>
<td>165 GPa</td>
<td>9.65 GPa</td>
<td>5.2 GPa</td>
<td>3.4 GPa</td>
<td>0.3</td>
<td>0.45</td>
</tr>
</tbody>
</table>

### 4.3.4 Regular Reinforcement and Steel Jacket

Regular reinforcements were modeled by the two-node linear 3D truss elements (T3D2).
The uniaxial stress-strain relationship was defined as bilinear elastic-plastic behavior, and the yield strength $f_y$ was defined as 420 MPa. The elastic modulus and the Poisson’s ratio of the steel were assumed to be 200 GPa and 0.3, respectively. The same material property was also applied to the steel jackets, with continuum 2D shell elements (S4R) being employed.

4.4 Concrete Damage Model

4.4.1 Modeling of Cracking and Fracture

Cracks in concrete are generally simulated by using either the discrete crack model or the smeared crack model. The discrete crack model was proposed by de Borst et al. (2004). In this model, cracks are treated as individual geometric entities, and the cracking of concrete is simulated by separating the edges of the elements in the cracking region. This model accords with the general understanding of cracks, and the propagating of a crack can be simply sketched in Figure 4-7. In 1985, Ingraffea and Saouma further developed this model by introducing the automatic remeshing. However, the obvious disadvantage of this method is that the cracks can only propagate along the predefined path.

![Figure 4-7 Propagation of a crack in discrete crack model (de Borst et al. 2004)](image)

The smeared crack model was first proposed by Rashid (1968), in which the cracked solid is still treated as a continuum. It does not track individual macro crack, instead, the cracks are considered as bands of micro cracks. Cracking of concrete is simulated by changing the stress-
strain relationship of the elements in the cracking region. This model is more widely used because it does not change the original mesh, and the path of crack propagation is not restricted.

In ABAQUS, different models are provided to simulate the cracking of concrete. Among them, the concrete damage plasticity model (CDP) and the brittle cracking model (BCM) are two models developed based on the smeared cracking model, which will be introduced and investigated in the following sections.

### 4.4.2 Concrete Damaged Plasticity Model (CDP)

The concrete damaged plasticity model is developed based on the model proposed by Lubliner et al. (1989), and the initial model is only applicable to concrete structures under monotonic loading. In 1998, Jeeho & Fenves further developed the model so that it could consider the cyclic loading. The uniaxial compressive and tensile behavior of concrete specified by this model are shown in Figure 4-8 (a) and (b), respectively. In the two figures, $E_0$ is the initial elastic stiffness of concrete; $\varepsilon_{c}^{pl}$ and $\varepsilon_{c}^{el}$ are plastic strain and elastic strain in compressive behavior, respectively; while $\varepsilon_{t}^{pl}$ and $\varepsilon_{t}^{el}$ are plastic strain and elastic strain in tensile behavior, respectively.

![Figure 4-8 Tensile and compressive behavior of concrete specified by CDP (ABAQUS Analysis User’s Manual, 2010)](image)

On the strain softening branch of each curve, the degradation of the stiffness is
characterized by the damage variables, $d_c$ and $d_t$, and the expressions are as follows:

\[
\sigma_c = (1 - d_c) \cdot E_0 \cdot (\epsilon_c - \epsilon_c^{pl}) 
\]

\[\text{(Equation 4-16)}\]

\[
\sigma_t = (1 - d_t) \cdot E_0 \cdot (\epsilon_t - \epsilon_t^{pl}) 
\]

\[\text{(Equation 4-17)}\]

In CDP, the yield surface is defined by five parameters: $\psi$ is the dilation angle measured in the $p-q$ plane as the inclination angle of the plastic potential function, for high confinement pressure. Generally, a higher value of the dilation angle results in a more ductile behavior of the concrete, and in this research, $\psi = 35^\circ$ was used. $\epsilon$ is the eccentricity of the plastic potential surface, and $\epsilon = 0.1$ was used. $\sigma_{b0}$ is the initial equibiaxial compressive yield stress, while $\sigma_{c0}$ is the initial uniaxial compressive yield stress. The default value of $\sigma_{b0}/\sigma_{c0}$ is 1.16. $K_c$ is the ratio of the second stress invariant on the tensile median to that on the compressive median at the initial yield, and the default value of $K_c$ is $2/3$. The viscosity parameter is used for the viscoplastic regularization of the concrete constitutive formulas. In ABAQUS/Standard, this parameter can help overcome the convergence difficulties caused by the stiffness degradation of concrete. Since ABAQUS/Explicit analysis was employed in this research, the default value of 0.0 was used, indicating that no viscoplastic regularizations were applied (ABAQUS Analysis User’s Manual, 2010).

4.4.3 Brittle Cracking Model (BCM)

Brittle cracking model is appropriate to model the structural members where the brittle behavior is dominant, such as the members with a low reinforcement ratio. Therefore, it was selected as another concrete cracking model to be investigated in this section.

At a node (material point), this model can consider different numbers of cracks in terms of different structural members, and the cracks are orthogonal. For a 3D, plane strain, and axisymmetric problem, a maximum of three cracks can be simulated. Two cracks can be simulated in a shell problem, while one crack can be simulated in a 2D beam or a truss problem. As punching
shear failure of three-dimensional RC footings is investigated in this research, three cracks (tri-directional) at each node is considered.

Cracks in the BCM are simply detected by the Rankine criterion, also called the maximum principal stress criterion. The crack forms when the maximum principal tensile stress of the node exceeds the tensile strength of the material, and the crack surface is normal to the direction of the maximum principal tensile stress. Once a crack forms at a node, it cannot recover in the rest of the analysis, but crack closing is achievable when the stress at the point becomes compressive.

Post-cracking behavior in the BCM can be defined by the post-cracking stress-strain relationship. For concrete material studied in this research, the tensile behaviors (including tension softening, tension stiffening, and local bond-slip effects) described by Figure 4-3 are applied. Brittle failure of a node is defined when one, two, or three strain components (depending on the structural type) at the node reach the failure strain which is predefined by the user, and then all stress components of the node are reduced to zero. The element is removed (visibly or virtually) when all nodes of the element are failed.

In ABAQUS, the primary purpose for introducing this model is to solve the problem where ignoring the elements unable to carry stresses in the model would cause the unexpected termination of the analysis due to excessive distortion.

4.5 Contacts

The contact between concrete and rebar was simply considered as “embedded”, because effects like bond slip or dowel action are already considered in the tensile behavior of concrete by introducing tension stiffening.

“Surface to surface contact” was employed to simulate the contact between prestressing strands and the concrete. The normal behavior was set as “hard contact”, so the penetration of the
strands into the concrete is prevented. As the prestressing strands applied in this research are pre-greased, the friction between the prestressing strands and the duct was ignored, and the tangential behavior was set as “frictionless”.

The contact between the additional and existing concrete was assumed as “fully bonded”, to reduce computation time and forbid the non-convergence caused by contact nonlinearity. However, the effectiveness of the connection in the strengthened models was investigated by checking the states of stress on the contact surfaces. Furthermore, on construction sites, an adhesive would be applied at the contact surface to ensure a good bond between additional and existing concrete.

With epoxy and high-quality construction, the contact between the concrete and the FRP wraps can be considered as “fully bonded”. Similarly, by installing shear studs on steel jackets, the contact between the concrete and the steel jackets are also considered as “fully bonded”.

4.6 Boundary Conditions

As the cross-sections of either the original footings or the strengthened footings are symmetric in both transverse directions, only a quarter of the footing was simulated in this research. The nodes on the top surface of the column are constrained in all degrees of freedom. All the nodes on the section perpendicular to Axial X are set as XSYMM, and all the nodes on the section perpendicular to Axial Y are set as YSYMM (Figure 4-9).
4.7 Validation of Experimental Data

A series of punching shear tests on spread footings with square sections were carried out by Hegger and co-workers ((Hegger et al. 2006, 2007, 2009), (Siburg et al. 2014; Siburg & Hegger 2014), (Wieneke et al. 2016), and (Kueres & Hegger 2018)). The details of a test specimen without shear reinforcements are shown in Figure 4-10 as an example. In their tests, the load was applied in increments of 100 kN to 200 kN, and the vertical displacement at the slab corner was measured by using the linear variable differential transformer (LVDT) gauges.
As shear reinforcements were not considered in our research, a total of six specimens without shear reinforcements were selected from their tests to carry out the validation of the finite element modeling, and the details of the selected specimens are listed in Table 4-2.

Table 4-2 Details of selected test specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>DF11</th>
<th>DF12</th>
<th>DF13</th>
<th>DF20</th>
<th>DF21</th>
<th>DF22</th>
</tr>
</thead>
<tbody>
<tr>
<td>d (mm)</td>
<td>395</td>
<td>395</td>
<td>395</td>
<td>395</td>
<td>395</td>
<td>395</td>
</tr>
<tr>
<td>c (mm)</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>b (mm)</td>
<td>1200</td>
<td>1400</td>
<td>1800</td>
<td>1200</td>
<td>1400</td>
<td>1800</td>
</tr>
<tr>
<td>a/d</td>
<td>1.27</td>
<td>1.52</td>
<td>2.03</td>
<td>1.27</td>
<td>1.52</td>
<td>2.03</td>
</tr>
<tr>
<td>$f'_{c}$ (MPa)</td>
<td>21.4</td>
<td>21.2</td>
<td>21.1</td>
<td>35.7</td>
<td>36.3</td>
<td>36.4</td>
</tr>
<tr>
<td>$E_{c}$ (MPa)</td>
<td>22,000</td>
<td>23,700</td>
<td>20,100</td>
<td>27,400</td>
<td>26,800</td>
<td>26,200</td>
</tr>
<tr>
<td>$\rho$ (%)</td>
<td>0.87</td>
<td>0.88</td>
<td>0.87</td>
<td>0.87</td>
<td>0.87</td>
<td>0.87</td>
</tr>
<tr>
<td>$V_{test}$ (kN)</td>
<td>2813</td>
<td>2208</td>
<td>1839</td>
<td>3037</td>
<td>2860</td>
<td>2405</td>
</tr>
</tbody>
</table>

The validation of the finite element model was conducted not only to ensure that the previously determined properties were accurate, but also to determine:

- Whether the CDP model or the BCM model is more appropriate to simulate the damage of concrete;
- Size of the element (mesh density); and
- Loading speed.

The finite element model of DF11 is shown in Figure 4-11. To generate the load-displacement curve, the vertical reaction forces of all the nodes on the top surface of the column were summed up as the total load, and the vertical displacement of the node at the corner of the footing was recorded as the displacement.

![Finite element model of DF11](image)

(a) (b)

Figure 4-11 Finite element model of DF11

### 4.7.1 Concrete Damage Model

Both CDP and BCM were employed to simulate the six selected specimens. For each footing, the load-displacement curves regarding different concrete models were compared with the experimental result. The investigations of the six specimens are shown in Figure 4-12.
Figure 4-12 Validating different concrete damage models with test results (Hegger et al. 2009).

Based on the comparisons, the load-displacement curves obtained using BCM apparently had better agreements with the experimental curves. Therefore, BCM was employed as the concrete damage model in finite element models in this research.

4.7.2 Element Size

In finite element analysis, the size of the element, or mesh density, is important for the accuracy of the simulation. It is widely recognized that the finite element model with a finer mesh...
can attain a result with higher accuracy, but computation time is longer. The five investigated element sizes were 10 mm, 15 mm, 20 mm, 25 mm, and 30 mm. The results of DF11 are shown in Figure 4-13.

![Figure 4-13 Investigations of different element sizes](image)

The differences between the curves regarding different element sizes were not significant. There was a tendency that the smaller the size of the element, the lower the failure load. For DF11, the curve with an element size of 25 mm yielded the best agreement with the experimental curve (Figure 4-13a). To study the convergence of key quantities, two curves were plotted in Figure 4-13b: Max. Stress vs. Element Size Curve (in orange dash line), and Ultimate Displacement vs. Element size Curve (in blue solid line). For each model, the maximum stress in reinforcing steels when the displacement reached 1.60 mm was tracked. For both Max. stress and ultimate displacement, good convergences were observed when the element size was decreasing.

### 4.8 Summary and Conclusion

In this chapter, details of finite element modeling were introduced. For different materials, including concrete, reinforcing steel, prestressing strand, BFRP and CFRP, their material properties and element types were identified. The contacts and boundary conditions were also presented.

ABAQUS\Explicit and ABAQUS\Implicit were introduced, and ABAQUS\Explicit was
determined to be employed in this research due to its remarkable advantages in solving problems with contact and material nonlinearities.

Validation of finite element modeling was completed in this chapter, with good agreements between the FEA results and the experimental results being achieved. Load-displacement curves obtained from models regarding the two different concrete damage models (BCM and CDP) were compared to the experimental curve, indicating that the BCM was more appropriate in this research. The element size was also calibrated, and it was determined to be 25 mm.
5 FEA MODELS AND PARAMETRIC STUDIES

5.1 Introduction

In this chapter, finite element analysis was applied to investigate the response of the five proposed strengthening systems (CEP, CERR, CEBW, CECW, and CESJ), particularly on their effectiveness in improving the punching shear capacity of RC footings. Totally 353 finite element models were built and analyzed using the commercial software ABAQUS. Details on finite element modeling (material properties, contacts, element size, etc.), and the validation of finite element modeling have already been addressed in the previous chapter.

The parameters investigated in the CEP system include:

- Footing type (spread footing and pile cap);
- Size of footing $B$ (3 m, 4 m, and 5 m);
- Shear-span to depth ratio $a/d$ (1.5, 2.0, and 2.5);
- Flexural reinforcement ratio $\rho_{\text{flex}}$ (0.75% and 1.50%);
- Notch length $N_L$ (300 mm, 150 mm, and 0);
- Number of prestressing strands $N_p$ (10, 20, 30, 40, and 50);
- Eccentricity of prestressing strands $e$ ($e_1$, $e_2$, $e_3$, and $e_4$), where $e_1$ and $e_4$ are the maximum eccentricity and minimum eccentricity, respectively.

The parameters investigated in the CERR system include:

- Size of footing $B$ (3 m and 5 m);
- Equivalent area of regular reinforcing steel $A_{rr}$ (two values were considered, which are equivalent to 10 and 30 strands, respectively).

The parameters investigated in CEBW, CECW, and CESJ:

- Size of footing $B$ (3 m and 5 m);
Figure 5-1 Modeling plan for spread footings

Spread Footing

Original \((h)\)

Thickened \((1.25h)\)

Preliminary

- \(N_p = 20\)
- \(e = \frac{2}{4} \cdot h\)
- \(\rho = 1.50\%\)

Determined

- \(a/d = 1.5\)
- \(N_L = 300 \text{ mm}\)
- \(h\) (original depth only)

CEP system (prestressing)

- \(\rho = 1.50\%\)
- \(e = \frac{1}{4} \cdot d\)
- \(N_p = 20\)

CFR system (regular reinforcement)

Determined

- \(a/d = 1.5\)
- \(N_L = 300 \text{ mm}\)
- \(\rho = 0.75\%\)
- \(e = \frac{1}{4} \cdot d\)
- \(N_p = 20\)

CFBW (BFRP wrapping)

Determined

- \(a/d = 1.5\)
- \(N_L = 300 \text{ mm}\)
- \(\rho = 0.75\%\)

CECW (CFRP wrapping)

CFSJ (Steel Jacketing)
- Thickness of FRP wrapping or steel jacket $t$ (8 mm, 16 mm, and 24 mm);
- Strengthening materials (BFRP, CFRP, and steel);
- Strengthening depth (full-depth, half-depth, and quarter-depth).

5.2 Plan of Modeling

The total number of finite element models investigated in this research is 353, including original and strengthened footings, and both spread footing and pile cap are considered.

Spread footings were designed to investigate all proposed strengthening systems, and a cogitative modeling plan is designed (Figure 5-1) to direct the investigations. As shown in Figure 5-1, the entire procedure of finite element modeling begins with the original spread footings, and the CEP system is the first to be investigated. Most of the finite element models (282/353) were built to investigate the CEP system, as it is an active system that can significantly improve the punching shear capacity of the RC footings.

Figure 5-2 Modeling plan for pile caps

Pile caps are only designed to investigate the CEP system, for the purpose of understanding
the differences in punching shear behaviors of RC footings under uniformly distributed load and concentrated load. The modeling plan of the pile cap is shown in Figure 5-2.

5.3 Evaluation Method of Proposed Systems

A term “Enhancement” was introduced to evaluate the effectiveness of each proposed system in improving the punching shear capacity of footings. The Enhancement is defined as:

\[
Enhancement = \frac{V_{ps, str} - V_{ps, ori}}{V_{ps, ori}} \times 100\%
\]

(Equation 5-1)

where, \(V_{ps, str}\) and \(V_{ps, ori}\) are punching shear capacities of strengthened and original footings, respectively. Punching shear capacity is calculated by the following equation:

\[
V_{ps} = P_{column} \cdot \frac{A_{total} - A_{critical}}{A_{total}}
\]

(Equation 5-2)

where, \(P_{column}\) is the total load in column when footing fails in punching shear; \(A_{total}\) is the total area of original or strengthened footings; and \(A_{critical}\) is the area enclosed by critical sections. The critical sections are determined to be located at a distance of \(d/2\) from the edge of the column, which is consistent with the provisions in ACI 318-14, AASHTO Code 2012 and Model Code 2010.

5.4 Original Footings

All original spread footings and pile caps were designed based on the AASHTO Code (2012), with only the square section being considered. For original spread footings, the parameters investigated are as follows:

- Size of footing \(B\) (3 m, 4 m, and 5 m)
- Flexural reinforcement ratio \(\rho_{flex}\) (0.75% and 1.50%)
- Shear-span to depth ratio \(a/d\) (1.5, 2.0, and 2.5).

In addition, the benefit of the depth thickening strategy on the punching shear capacity was
also studied. For spread footings with $\rho_{flex} = 1.50\%$, their depths were increased by one quarter of the original depth. Models of original and thickened spread footings regarding different sizes and different shear-span to depth ratios are listed in Figure 5-3.

![Figure 5-3 Original and thickened spread footings](image)

For original pile caps, the parameters investigated are as follows:

- Size of footing $B$ (3 m, 4 m and 5 m)
- Shear-span to depth ratio $a/d$ (1.5, 2.0 and 2.5).

The flexural reinforcement ratio $\rho_{flex}$ was only considered to be 1.50%. Also, depth thickening was considered by increasing the depth by one quarter of the original depth ($1.25h$). The original and thickened pile caps with $a/d = 1.5$ are shown in Figure 5-4 as an example. The diameter of each pile was considered as 300 mm, and the number of piles in each footing is equal to the square of the footing size ($B^2$). For example, in a 3 m pile cap, nine piles were designed.
5.4.1 Punching Shear Behavior

All original RC footings failed in punching shear. To understand the punching shear behavior, the load-displacement curve generated from each model was studied. The states of Max. principal strain on two surfaces (Figure 5-5) were also investigated to understand the internal causes of the behavior. One of them is the bottom surface, where flexural cracks initiate. The other one is the vertical surface, on which both shear and flexural cracks occur and propagate. As only one quarter of footing was simulated, the vertical surface is actually half of the middle cross-section of the original footing.
The original RC footings have similar punching shear behaviors. Thus, in this section, only three models were selected as examples to be discussed, including two spread footings and one pile cap, with different $a/d$ and $\rho_{\text{flex}}$ being considered:

- 3 m original spread footing, with $a/d = 1.5$, and $\rho_{\text{flex}} = 0.75\%$;
- 3 m original spread footing, with $a/d = 2.5$, and $\rho_{\text{flex}} = 1.50\%$;
- 3 m original pile cap, with $a/d = 1.5$, and $\rho_{\text{flex}} = 1.50\%$.

### 5.4.1.1 3 m original spread footing, with $a/d = 1.5$, and $\rho_{\text{flex}} = 0.75\%$

The load-displacement curve and the states of Max. principal strains are displayed in Figure 5-6. Three points are marked on the curve to describe the punching shear failure: FC means flexural cracking, marking the step when a primary flexural crack reaches to the edge of the bottom surface; FSC means flexural-shear cracking; and PSF means punching shear failure. Before FSC, the flexural cracking dominates. The first flexural crack occurs on the bottom surface, starting from the center, propagating towards the edge of the surface. At FC, a distinct decrease of the slope can be observed. Afterwards, the existing flexural cracks begin to propagate on the vertical cross-section, with more flexural cracks showing up on the bottom surface. After FSC, the flexural-shear cracking dominates, and a primary inclined shear crack gradually forms on the vertical section.
The shear crack generates from the bottom of the section, propagating to the edge of the column stub. At PSF, punching shear failure occurs when the primary shear crack reaches the upper edge of the section. The footing loses the load carrying capacity, and a negative slope can be observed from the curve.

![Load-displacement curve of 3 m original spread footing](image)

Until punching shear failure, the maximum stress in reinforcing steels is always below the yield strength (420 MPa).
5.4.1.2 3 m original spread footing, with $a/d = 2.5$, and $\rho_{\text{flex}} = 1.50\%$

To investigate if either the shear-span to depth ratio $a/d$ or the flexural reinforcement ratio $\rho_{\text{flex}}$ has an influence on the punching shear failure behavior, the load-displacement curve of the 3 m original spread footing with $\rho_{\text{flex}} = 1.50\%$ and $a/d = 2.5$ is shown in Figure 5-7.

![Figure 5-7 Load-displacement curve of 3 m original spread footing ($a/d = 2.5, \rho_{\text{flex}} = 1.50\%$) (Unit: MPa)](image)

A similar punching shear behavior can be observed, and two stages can be distinguished: a flexural cracking dominating stage (before FSC) and a flexural-shear cracking dominating stage.
(after FSC). Punching shear failure occurs at PSF when the primary shear crack on the vertical cross-section reaches the upper edge. The maximum stress in the reinforcing steels through the whole process is below the yield strength.

5.4.1.3 3 m original pile cap, with $a/d = 1.5$, and $\rho_{\text{flex}} = 1.50\%$

Figure 5-8 Load-displacement curve of 3 m original pile cap ($a/d = 1.5$, $\rho_{\text{flex}} = 1.50\%$) (Unit: MPa)

To investigate if the footing type has an influence on the punching shear behavior, the results of 3 m original pile cap with $a/d = 1.5$, and $\rho_{\text{flex}} = 1.50\%$ are displayed in Figure 5-8. The
punching shear behavior represented is slightly different from those of the spread footings. The second stage is dominated by shear cracking, instead of flexural-shear cracking, as the first shear crack occurs at the web of the footing. Punching shear failure still occurs when the primary shear crack reaches the upper edge of the vertical cross-section. The maximum stress in the reinforcing steels through the whole process is below the yield strength.

5.4.2 Effect of Flexural Reinforcement Ratio $\rho_{\text{flex}}$

Two different flexural reinforcement ratios $\rho_{\text{flex}}$ were investigated: 0.75%, and 1.50%. In Figure 5-9, results obtained from 3 m (a) and 5 m (b) original spread footings with $a/d = 1.5$ and different flexural reinforcement ratios are selected as examples to be discussed.

![Figure 5-9 Load-displacement curves of original spread footings with different sizes](image)

Investigations indicate the flexural reinforcement ratio has a significant influence on the punching shear capacity of the spread footings. The footing with a higher reinforcement ratio achieves a higher ultimate load, and it represents a much stiffer behavior in the second stage (after the flexural crack reaches the edge of the bottom surface). In Figure 5-9 (a) and (b), the two curves almost overlap at the early stage, indicating the reinforcement ratio only has a slight effect on the elastic behavior. Also, increasing the flexural reinforcement ratio postpones the beginning of the
second stage.

It is worth mentioning that in both ACI 318-14 and AASHTO Code 2012, flexural reinforcement ratio is not considered in the provisions for punching shear capacity. On the contrary, the formula for punching shear capacity in Eurocode 2 accounts for the influence of the flexural reinforcement ratio:

\[ v_{EC2} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho \cdot f'_{c})^{0.5} \cdot \frac{2d}{a_{cri}} \]  

(Equation 5-3)

where \( \rho \) is the flexural reinforcement ratio.

5.4.3 Effect of Shear-span to Depth Ratio \( a/d \)

Three different shear-span to depth ratios \( a/d \) were investigated: 1.5, 2.0, and 2.5. In Figure 5-10, results obtained from 3 m (a) and 5 m (b) original spread footings with \( \rho_{flex} = 1.50\% \) and different shear-span to depth ratios are discussed.

Figure 5-10 Load-displacement curves of original spread footings with different shear-span to depth ratios

Based on the comparisons, the shear-span to depth ratio has a significant influence on the punching shear capacity of spread footing. The smaller the parameter, the larger the ultimate load. A footing with a smaller shear-span to depth ratio presented a stiffer behavior, and the
displacement at failure was less.

5.4.4 Effect of Depth Thickening

In this research, depth thickening was applied by adding a plain concrete segment on the top of the original footing. The thickness of the additional segment was considered as one quarter of the original depth. The evaluation of the depth thickening strategy was carried out in footings with the larger flexural reinforcement ratio $\rho_{\text{flex}} = 1.50\%$. Other parameters including footing size and shear-span to depth ratio were taken as variables.

In Figure 5-11, results obtained from 3 m (a) and 5 m (b) spread footings with $\rho_{\text{flex}} = 1.50\%$ and the original $a/d = 1.5$, regarding different depths are discussed, to examine the effectiveness of the depth thickening strategy. The comparisons indicate the depth thickening strategy is capable of improving the punching shear capacity as expected. The thickened footing exhibits a stiffer behavior and results in a larger ultimate load, and the displacement at the punching shear failure is slightly smaller.

![Figure 5-11](image)

(a) 3 m  
(b) 5 m

Figure 5-11 Load-displacement curves of 3 m (a) and 5 m (b) original and thickened spread footings with $\rho_{\text{flex}} = 1.50\%$ and original $a/d = 1.5$

Figure 5-12 displays the enhancements in punching shear capacity after the depth thickening strategy is applied. It can be observed that the enhancements are between 33% and 45%.
When $a/d = 2.0$, for footings with different sizes, similar enhancements are obtained, and the enhancements are about 35%.

![Figure 5-12 Effective of the depth thickening strategy ($\rho_{\text{flex}} = 1.50\%$)](image)

5.4.5 Effect of Footing Type

Both spread footing and pile cap are considered in this research, to investigate the effects of the footing type, which is actually the loading type, on the punching shear behavior of RC footings. In Figure 5-13, results generated from 3 m (a) and 5 m (b) footings with $\rho_{\text{flex}} = 1.50\%$ and $a/d = 1.5$, regarding different footing types are displayed.

![Figure 5-13 Load-displacement curves of 3 m (a) and 5 m (b) footings with $\rho_{\text{flex}} = 1.50\%$ and $a/d = 1.5$, regarding different footing types](image)
For both sizes, ultimate failure loads of the spread footings are greater than those of the pile caps. At the early stage, load-displacement curves of spread footing and pile cap are almost overlapped, indicating that the load type only has a slight effect in the flexural cracking dominating stage. During the flexural-shear cracking dominating stage, shear cracks appear and propagate on the vertical cross-section, and distinct differences between the curves can be observed. The effect of the load type is less significant for the 5 m footings, as the load type in the pile cap is closer to that in spread footing when more piles participate.

5.5 Models for the CEP System

The investigation of the CEP system was accomplished in two steps: The Preliminary Step and the Determination Step (as shown in Figure 5-1). The purpose of the Preliminary Step is to select the parameters to be further investigated in the Determination Step.

5.5.1 The Preliminary Step

In the Preliminary Step, both spread footings and pile caps were investigated. The benefit of the depth thickening strategy was examined by adding a concrete overlay with the thickness equal to \( h/4 \). To make the investigations efficient, three parameters were set as constants in this step:

- Flexural reinforcement ratio \( \rho_{\text{flex}} = 1.50\% \);
- Number of prestressing strands \( N_p = 20 \);
- Eccentricity of the prestressing strands \( e = h/4 \).

The following parameters were set as variables to be investigated in the Preliminary Step:

- Notch length \( N_l \) (300 mm, 150 mm and 0);
- Shear-span to depth ratio \( a/d \) (1.5, 2.0 and 2.5);
- Footing size \( B \) (3 m, 4 m and 5 m).
5.5.1.1 Punching shear behavior

All strengthened footings investigated in this step failed in punching shear. To understand the punching shear behavior of the footings strengthened with the CEP system, load-displacement curves and state of Max. principal strains were studied. In this section, a strengthened spread footing and a strengthened pile cap were selected as examples to be discussed:

- 3 m spread footing, with \(a/d = 1.5\), \(\rho_{flex} = 1.50\%\), and \(N_L = 300\) mm, strengthened with 20 strands at \(e = h/4\);

- 3 m pile cap, with \(a/d = 1.5\), \(\rho_{flex} = 1.50\%\), and \(N_L = 300\) mm, strengthened with 20 strands at \(e = h/4\).

5.5.1.1.1 3 m spread footing, with \(a/d = 1.5\), \(\rho_{flex} = 1.50\%\), and \(N_L = 300\) mm, strengthened with 20 strands at \(e = h/4\)

The results of the strengthened spread footing are displayed in Figure 5-14. Similar to the original spread footing (Figure 5-6), the punching shear behavior can still be described by two stages: a flexural cracking dominating stage (before FSC) and a flexural-shear cracking dominating stage (after FSC). However, the following three differences can be detected:

- Most of the process is dominated by flexural-shear cracking, while in the original footing, most of the process is dominated by flexural cracking.

- Two primary shear cracks are observed on the vertical cross-section. One of them is the same as the one in the original footing, which starts from the bottom of the section, propagating and finally reaching the upper edge. The other one starts from the bottom of the column stub, propagating and finally reaching the location of prestressing strands.

- Punching shear failure occurs when the second primary shear crack reaches the
location of prestressing strands, while in the original footing, it occurs when the only primary shear crack reaches to the upper edge of the vertical section.

Figure 5-14 Load-displacement curve of 3 m strengthened spread footing \((a/d = 1.5, \rho_{\text{flex}} = 1.50\%, N_L = 300 \text{ mm}, \text{original depth}, e = h/4, N_p = 20)\) (Unit: MPa)

Until punching shear failure, the maximum stress in the prestressing strands is below the yield strength. Besides, the maximum stress in the reinforcing steels at PSF is only 285 MPa (< 420 MPa).
5.5.1.1.2 3 m pile cap, with \( a/d = 1.5 \), \( \rho_{\text{flex}} = 1.50\% \), and \( N_L = 300 \) mm, strengthened with 20 strands at \( e = h/4 \)

Different punching shear behavior is observed for the strengthened pile cap, and the behavior is completely dominated by shear cracking.

Figure 5-15 Load-displacement curve of 3 m strengthened pile cap (\( a/d = 1.5 \), \( \rho_{\text{flex}} = 1.50\% \), \( N_L = 300 \) mm, original depth, \( e = h/4 \), \( N_p = 20 \)) (Unit: MPa)

At SC, the first inclined shear crack occurs at the web of the footing, propagating towards
the closest pile. The second shear crack starts from the vicinity of the column stub, propagating towards the bottom edge of the vertical cross-section. Punching shear failure occurs at PSF, when the second shear crack reaches to the bottom edge.

At punching shear failure, the maximum stress in the reinforcing steels is 337 MPa ($< 420$ MPa). The maximum stress in the prestressing strands is 1685 MPa, which is close to the yield strength of 1687.8 MPa.

5.5.1.2 Effect of notch length $N_L$

As mentioned in Chapter 2, the first step to implement the CEP system is to cut notches at four corners of the original square footing, and the notches are prepared for prestressing duct and strands (Figure 5-16).

![Figure 5-16 Cutting notches at four corners](image)

Three notch lengths were considered: 0, 150 mm and 300 mm (Figure 5-17). As a footing enlargement system, the enlarged area is related to the notch length: the smaller the notch length, the larger the enlarged area.
Figure 5-17 Three different notch lengths considered in this research (Unit: mm)

Table 5-1 lists the enlarged area regarding different notch lengths for footings with different sizes. When $N_L = 300$ mm, the enlargement of the area is about 57%.

<table>
<thead>
<tr>
<th>Notch length (mm)</th>
<th>3 m Footing</th>
<th>4 m Footing</th>
<th>5 m Footing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Area (m²)</td>
<td>Enlargement</td>
<td>Area (m²)</td>
</tr>
<tr>
<td>300</td>
<td>14.15</td>
<td>57.2%</td>
<td>25.14</td>
</tr>
<tr>
<td>150</td>
<td>16.22</td>
<td>80.2%</td>
<td>27.88</td>
</tr>
<tr>
<td>0</td>
<td>18.43</td>
<td>104.8%</td>
<td>30.76</td>
</tr>
</tbody>
</table>

To investigate the effect of the notch length on the punching shear capacity, the flexural reinforcement ratio $\rho_{flex}$ was fixed to 1.50% to ensure that all the strengthened footings would fail in punching shear as expected, especially the footings with a larger shear-span to depth ratio ($a/d = 2.5$) and a smaller notch length ($N_L = 0$).

In Figure 5-18, load-displacement curves of 3 m and 5 m strengthened spread footings with
$a/d = 1.5$ and original depth regarding different notch lengths are compared. Since only small differences between the curves were observed, the effects of the notch length on both punching shear behavior and capacity are insignificant.

![Load-displacement curves](image)

**Figure 5-18** Load-displacement curves of 3 m (a) and 5 m (b) strengthened spread footings with original depth and $a/d = 1.5$, regarding different notch lengths

### 5.5.1.3 Effect of shear-span to depth ratio $a/d$

In the Preliminary Step, three values were selected to investigate the effect of the shear-span to depth on the punching shear capacity: 1.5, 2.0, and 2.5. In Figure 5-19, results obtained from 3 m and 5 m strengthened spread footings with original depth and $T_L = 300$ mm, regarding different shear-span to depth ratios are displayed.

![Load-displacement curves](image)

**Figure 5-19** Load-displacement curves of 3 m (a) and 5 m (b) strengthened spread footings with original depth and $N_L = 300$ mm, regarding different shear-span to depth ratios

Footings with a smaller shear-span to depth ratio presented a larger punching shear capacity.
and a smaller displacement at failure. With the same number of strands (20), the smaller the shear-span to depth ratio, the smaller the enhancement after retrofitting. For example, the enhancement of the 3 m footing with $a/d = 1.5$ is 99%, while the enhancement for the footing with $a/d = 2.5$ is 110%. With the same shear-span to depth ratio, the smaller the size, the larger the enhancement of punching shear capacity.

5.5.1.4 Effect of depth thickening

The benefit of the depth thickening strategy was also investigated in the Preliminary Step. Figure 5-20 displays the load-displacement curves generated for 3 m (a) and 5 m (b) spread footings with original $a/d = 1.5$, and $N_L = 300$ mm, regarding original depth and thickened depth. Based on the comparisons, footings with and without thickening represented similar punching shear behaviors. Thickening the depth significantly improved the punching shear capacity of footings. For 3 m spread footings (a), thickening the depth slightly increased the displacement at failure. The opposite conclusion was drawn from 5 m spread footings (b), where the displacement at failure decreased after the depth is thickened.

![Figure 5-20](image)

Figure 5-20 Load-displacement curves of 3 m (a) and 5 m (b) strengthened spread footings with original $a/d = 1.5$ and $N_L = 300$ mm, regarding original depth and thickened depth

5.5.1.5 Effect of footing type

For pile caps strengthened with the CEP system, additional piles are required as the size of
the footing is enlarged. The diameter of the additional pile in this research was determined to be 300 mm. Only four piles were added to each enlarged footing, with one at each side, regardless of the difference in the footing size. The notch length $N_l$ considered in pile caps were the same with those in the spread footings (Figure 5-17), and the locations of additional piles slightly varied regarding different notch lengths $N_l$. Figure 5-21 shows several pile caps strengthened with the CEP system as examples, and the shear-span to depth ratios $a/d$ is 1.5.

![Figure 5-21 Pile caps strengthened with the CEP system ($a/d = 1.5$)](image)

To investigate the effect of the footing type (also loading type) on the punching shear behavior, Figure 5-22 displays the results obtained from 3 m (a) and 5 m (b) footings strengthened with the CEP system ($\rho_{\text{flex}}=1.50\%, a/d=1.5, N_l = 300 \text{ mm, original depth}$), regarding different footing types. For 3 m footings, the enhancements in strengthened spread footing and strengthened pile cap are 99% and 100%, respectively, indicating that the improvements in punching shear capacities are almost the same. However, a significant difference is found in 5 m footings, and the enhancement in the spread footing (72%) is greater than that in the pile cap (26%).

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5.5.2 The Determination Step

As the number and the eccentricity of prestressing strands are two important parameters in the CEP system, they were further investigated in the Determination Step. The shear-span to depth ratio $a/d$ and the notch length $N_L$ were determined as 1.5 and 300 mm, respectively.

To sum up, the parameters considered in this step include:

- Footing size $B$ (3 m, 4 m and 5 m);
- Flexural reinforcement ratio $\rho_{\text{flex}}$ (0.75%, 1.50%);
- Number of prestressing strands $N_p$ (10, 20, 30, 40, 50);
- Eccentricity of prestressing strands $e$ ($e_1$, $e_2$, $e_3$, and $e_4$), where $e_1$ and $e_4$ are the maximum eccentricity and minimum eccentricity, respectively.

5.5.2.1 Punching shear behavior

All footings investigated in the Determination Step also failed in punching shear. To understand the influences of parameters $N_p$ and $e$ on the punching shear behavior, the load-displacement curves and the states of Max. principal strains were discussed. The results indicated that only the number of prestressing strands $N_p$ had a significant effect on the punching shear.
behavior. Therefore, 3 m spread footings \((a/d = 1.5, \rho_{\text{flex}} = 0.75\%, N_L = 300 \text{ mm}, \text{ original depth})\) strengthened with three different numbers of prestressing strands were selected as examples to be discussed in this section. The three models selected are as follows:

- 3 m spread footing \((a/d = 1.5, \rho_{\text{flex}} = 0.75\%, N_L = 300 \text{ mm}, \text{ original depth})\) strengthened with 10 strands at the maximum eccentricity \((e_1)\);

- The same 3 m spread footing strengthened with 30 strands at the maximum eccentricity \((e_1)\);

- The same 3 m spread footing strengthened with 50 strands at the maximum eccentricity \((e_1)\).

5.5.2.1.1 3 m spread footing \((a/d = 1.5, \rho_{\text{flex}} = 0.75\%, N_L = 300 \text{ mm}, \text{ original depth})\) strengthened with 10 strands at the maximum eccentricity

Since only 10 strands are applied, the punching shear behavior is similar to that of the original footing (Figure 5-23).

Two stages can be distinguished: a flexural cracking dominating stage (before FSC) and a flexural-shear cracking dominating stage (after FSC). Punching shear failure occurs at PSF when the primary shear crack on the vertical cross-section reaches the upper edge. At PY, the prestressing strands reach the yield strength. The maximum stress in the reinforcing steels through the whole process is below the yield strength, which also indicates the footing fails in punching shear.
5.5.2.1.2 The same 3 m spread footing strengthened with 30 strands at the maximum eccentricity

A different punching shear behavior is observed when the same 3 m spread footing is strengthened with 30 strands (Figure 5-24).
Figure 5-24 Load-displacement curve of 3 m strengthened spread footing ($a/d = 1.5$, $\rho_{\text{flex}} = 0.75\%$, $N_L = 300$ mm, original depth, $e_{\text{max}}$, $N_p = 30$) (Unit: MPa)

Two stages can be distinguished by the point SC (shear cracking). The first stage is still dominated by the flexural cracking, and a decrease of the slope is detected at FC, when the primary flexural cracks on the bottom surface reach to the edge. However, the second stage is dominated by the shear cracking, instead of the flexural-shear cracking detected in the footing strengthened with 10 strands. At SC, a primary shear crack occurs at the web of the vertical section. It propagates towards to two destinations: one is the bottom of the column stub, and the other one is the location of prestressing strands. Punching shear failure occurs at PSF, when the primary shear crack reaches
to the prestressing strands. Until punching shear failure, the maximum stresses in both the prestressing strands and the reinforcing steels are below their yield strengths.

**5.5.2.1.3 The same 3 m spread footing strengthened with 50 strands at the maximum eccentricity**

![Load-displacement curve](image)

Figure 5-25 Load-displacement curve of 3 m strengthened spread footing \((a/d = 1.5, \rho_{\text{flex}} = 0.75\%, N_L = 300 \text{ mm}, \text{original depth}, \varepsilon_{\text{max}}, N_P = 50)\) (Unit: MPa)

Punching shear behavior changes again when the same spread footing is strengthened with 50 strands, and the behavior is completed dominated by shear cracking (Figure 5-25). The primary shear crack occurs at an early stage, propagating towards to two destinations: the bottom of the
column stub and the location of prestressing strands. Unlike that in the footing strengthened with 30 strands, punching shear failure of this footing does not occur when the primary shear crack reaches the location of prestressing strands. Instead, it occurs when the reinforcing steels reach the yield strength. The maximum stress in the prestressing strands is still below the yield strength until the failure occurs.

5.5.2.2 Effect of eccentricity \( e \)

In the Determination Step, four eccentricities \((e_1, e_2, e_3, \text{ and } e_4)\) were considered, and they were determined by the following equations:

\[
e_1(\text{maximum}) = \begin{cases} \frac{h}{2} - 125\text{mm}, & N_p = 10, 20, 30 \\ \frac{h}{2} - 135\text{mm}, & N_p = 40, 50 \end{cases} \quad (\text{Equation 5-4})
\]

\[
e_4(\text{minimum}) = \frac{h}{4} \quad (\text{Equation 5-5})
\]

where \( h \) is the depth of the footing (Figure 5-26).

![Figure 5-26 Eccentricities investigated in the Determination Step](image)

In Figure 5-34, to discuss the influence of the eccentricity on the punching shear capacity, the results obtained from 3 m and 5 m spread footings \((a/d = 1.5, N_L = 300 \text{ mm}, \text{ and } \rho_{\text{flex}} = 1.50\%)\)
strengthened with 20 prestressing strands \((N_p = 20)\), regarding different eccentricities are displayed. Based on the comparisons, the larger the eccentricity, the larger the enhancement. The punching shear behaviors of footings with different eccentricities are similar. For 3 m strengthened spread footings, there is a tendency that the larger the eccentricity, the larger the displacement at failure.

![Figure 5-27 Load-displacement curves of 3 m (a) and 5 m (b) strengthened spread footings with \(a/d = 1.5\), \(N_L = 300 \text{ mm}\), \(\rho_{\text{flex}} =1.50\%\) and \(N_p = 20\), regarding different eccentricities](image)

**5.5.2.3 Effect of number of prestressing strands \(N_p\)**

Five values were considered to study the effect of the number of prestressing strands \(N_p\): 10, 20, 30, 40 and 50. The results obtained from 3 m (a) and 5 m (b) strengthened spread footings \((a/d = 1.5\), \(N_i = 300 \text{ mm}\), \(\rho_{\text{flex}} =0.75\%\) and \(e = e_i\)\), regarding different numbers of prestressing strands are displayed in Figure 5-28. The comparisons indicate the number of prestressing strands has a significant influence on the punching shear capacity. For 3 m spread footings strengthened with 10 and 50 strands, the enhancements range from 55% to 221%. For 5 m spread footings, the enhancements range from 30% to 171%. When the same number of prestressing strands is applied, the larger the size of footing, the smaller the enhancement. For example, for 5 m spread footing strengthened with 50 strands, the enhancement is 171%, which is smaller than that in 3 m spread footing (221%).
(a) 3 m  
(b) 5 m

Figure 5-28 Load-displacement curves of 3 m (a) and 5 m (b) strengthened spread footings \((a/d = 1.5, N_L = 300 \text{ mm}, \rho_{\text{flex}} = 0.75\% \text{ and } e = e_1)\), regarding different numbers of prestressing strands

5.5.2.4 Effect of flexural reinforcement ratio \(\rho_{\text{flex}}\)

Two flexural reinforcement ratios are investigated in the Determination Step: 0.75\% and 1.50\%. The results obtained from 3 m and 5 m spread footings \((a/d = 1.5, N_L = 300 \text{ mm})\) strengthened with \(N_p=10\), and \(e = e_1\), regarding different flexural reinforcement ratios are displayed in Figure 5-29.

(a) 3 m  
(b) 5 m

Figure 5-29 Load-displacement curves of 3 m (a) and 5 m (b) strengthened spread footings with \(a/d = 1.5, N_L = 300 \text{ mm} \text{ and } N_p=10\), regarding different flexural reinforcement ratios

The comparisons indicate that when the same number of prestressing strands are applied, the larger the reinforcement ratio, the larger the improvement in the punching shear capacity. Footings with different reinforcement ratios represent similar punching shear behavior. For 3 m
strengthened spread footings, the footing with $\rho_{\text{flex}}=1.50\%$ achieves a larger displacement at failure. Whereas for 5 m strengthened spread footings, the footing with $\rho_{\text{flex}}=0.75\%$ achieves a larger displacement at failure.

5.5.3 State of Stress at Contact Surface

To check the effectiveness of the connection at the contact surface between the existing and additional concrete segments, the state of stress at the contact surface was investigated. For spread footings strengthened with the CEP system, 3 m and 5 m spread footings strengthened with 10 strands are most critical. Therefore, they were selected to be investigated, and the surfaces to be examined are shown in Figure 5-30.

5.5.3.1 3 m spread footing ($a/d = 1.5, \rho_{\text{flex}} = 0.75\%, N_L = 300 \text{ mm, original depth}$) strengthened with 10 strands at the eccentricity $e_1$

The results for 3 m spread footing strengthened with 10 strands are displayed in Figure 5-31, and only tensile stresses ($\sigma > 0$) are displayed in colors.

At the beginning of loading, a small area at the top of the contact surface is in tension due to the prestressing force. At Point FC, the area under tension is decreased due to the increase in
the external loads. After that, the state of stress on the contact surface becomes disordered as flexural and shear cracks occur in the concrete. The maximum tensile stress 1.2 MPa is found at punching shear failure, which is smaller to the tensile strength of concrete. Along with applying the strong construction adhesive, the connection at the contact surface is reliable.

5.5.3.2 5 m spread footing \((a/d = 1.5, \rho_{flex} = 0.75\%, N_L = 300 \text{ mm}, \text{original depth})\) strengthened with 10 strands at the eccentricity \(e_1\)

The results for 5 m spread footing strengthened with 10 strands are displayed in Figure
5.32. The maximum stress on the contact surface is only 2.1 MPa, detected at the punching shear failure.

### 5.5.4 Comprehensive parametric study

To comprehensively study all the parameters considered in the Determinations Step, in this section, the enhancements on the punching shear capacity of spread footings regarding different parameters are displayed.

Figure 5-33 shows the enhancements on punching shear capacity of 3 m spread footings (a) $\rho_{\text{flex}}=0.75\%$ (b) $\rho_{\text{flex}}=1.50\%$ ($a/d = 1.5$, $N_L = 300$ mm), strengthened with the CEP system, regarding different numbers of prestressing strands and different eccentricities. Increasing the eccentricity and the number of prestressing strands increases the enhancement. The results for 4 m and 5 m strengthened spread footings can be found in Appendix A.

![Enhancements on punching shear capacity of 3 m spread footings](image)

Figure 5-33 Enhancements on punching shear capacity of 3 m spread footings (a) $\rho_{\text{flex}}=0.75\%$ (b) $\rho_{\text{flex}}=1.50\%$, strengthened with CEP, regarding different $N_p$ and $e$

### 5.6 Models for the CERR System

The investigations of the CERR system were only carried out on spread footings, with following parameters being determined as constants: $a/d = 1.5$, $N_L = 300$ mm, and $\rho_{\text{flex}}=0.75\%$. The depth thickening strategy was applied to all strengthened footings, and the depth of each
footing was increased by one quarter of the original depth. Other parameters studied in this system were listed as follows:

- Footing size $B$ (3 m and 5 m);
- Area of the circular regular reinforcing steel $A_{rr}$ ($EQ10$, $EQ30$).

### 5.6.1 Punching Shear Behavior

![Load-displacement curve of 3 m thickened spread footing strengthened with the CERR system ($A_{rr} = EQ10$) (Unit: MPa)](image)

Spread footings strengthened with the CERR system all failed in punching shear. The load-displacement curves obtained from strengthened footings regarding different parameters are
similar in shape. Therefore, in Figure 5-34, only the load-displacement curve and the states of Max. principal strains of the 3 m thickened spread footing strengthened with EQ10 are displayed as an example.

The punching shear behavior is similar to that of the original 3 m spread footing. Two stages can be distinguished: a flexural cracking dominating stage (before FSC) and a flexural-shear cracking dominating stage (after FSC). Punching shear failure occurs at PSF when the primary shear crack on the vertical cross-section reaches the upper edge. The maximum stress in the flexural reinforcements (352 MPa) and the maximum stress in the circular reinforcing steels (171.3 MPa) through the whole process are below the yield strength (420 MPa).

5.6.2 Effect of Area of Circular Reinforcing Steel

The area of the circular reinforcing steel $A_{rr}$ is an important parameter in this system. Two values were determined based on the force equivalent: the force causing the yielding of the circular reinforcing steel equals to the force causing the yielding of 10 (or 30) strands in the CEP system:

$$A_{rr} = \frac{f_{py}A_{p}N_{p}}{f_{sy}}$$

(Equation 5-6)

where, $f_{py}$ is the yield strength of prestressing strands; $A_{p}$ is the area of a single prestressing strand; $N_{p}$ is the number of prestressing strands, $f_{sy}$ is the yield strength of the regular reinforcing steel. As 10 and 30 strands were considered, the values of $A_{rr}$ were calculated as 3967 mm$^2$ (EQ10) and 11902 mm$^2$ (EQ30), respectively.

In Figure 5-35, the load-displacement curves obtained from 3 m and 5 m spread footings strengthened with different areas of the circular reinforcing steels are compared. The comparisons indicate the CERR system is effective in improving the punching shear capacity of the spread footings. However, as a passive system, the effect of the area of the circular reinforcing steel on punching shear capacity is not significant. Increasing the area slightly increases the punching shear
capacity. After being strengthened, the displacements at failure also slightly increased.

Figure 5-35 3 m (a) and 5 m (b) spread footings with thickened depth, strengthened with the CERR system

5.6.3 Effect of the Footing Size $B$

Based on the results shown in Figure 5-35, the effect of the footing size can also be discussed. For the 3 m spread footing, the enhancements in punching shear capacities by $EQ30$ and $EQ10$ are 76% and 57%, respectively. In the 5 m spread footing, the enhancements in punching shear capacities by $EQ30$ and $EQ10$ are 42% and 31%, respectively. Therefore, when the same area of circular reinforcing steel is applied, the enhancement on the punching shear capacity of the footing with a larger size is smaller.

5.6.4 State of Stress at Contact Surface

The effectiveness of the connection between the existing and additional concrete segments is investigated by checking the state of stress at the contact surface. For spread footings strengthened with the CERR system, 3 m and 5 m spread footings strengthened with $A_{re} = EQ10$ are most critical. Therefore, they were selected to be investigated, and the surfaces to be examined are shown in Figure 5-36.
5.6.4.1 3 m spread footing \((a/d = 1.5, \rho_{\text{flex}} = 0.75\%, N_L = 300 \text{ mm, thickened depth})\)

strengthened with \(A_{rr} = EQ10\)

![Diagram](image1)

Figure 5-37 State of stress on the contact surface for 3 m spread footing strengthened with \(EQ10\) (Unit: MPa)

The results for 3 m spread footing strengthened with \(EQ10\) are displayed in Figure 5-37, and only tensile stresses \((\sigma > 0)\) are displayed in colors. At the beginning of loading, concrete at the bottom of the contact surface is in tension. State of stress becomes disordered around Point FC, due to cracking in the concrete. The tensile stress during the whole loading process remains small,
and it is 1.9 MPa at punching shear failure. Along with applying the strong construction adhesive, the connection at the contact surface is reliable.

5.6.4.2 5 m spread footing \( (a/d = 1.5, \rho_{\text{flex}} = 0.75\%, \ N_L = 300 \text{ mm, thickened depth}) \) strengthened with \( A_{rr} = EQ10 \)

The results for 5 m spread footing strengthened with \( EQ10 \) are displayed in Figure 5-38. The tensile stress during the whole process also remains small, and it is 1.9 MPa at punching shear failure, which is smaller than the tensile strength of concrete. Therefore, the connection on the contact surface is also effective.

![Figure 5-38 State of stress on the contact surface for 5 m spread footing strengthened with \( EQ10 \) (Unit: MPa)](image)

5.7 Models for CEBW, CECW and CESJ Systems

CEBW, CECW and CESJ Systems were investigated simultaneously in this section. In these systems, BFRP wraps, CFRP wraps and steel jackets are applied on the external surface of the enlarged footings. Only spread footings were considered, and the following parameters are studied:

- Footing size \( B \) (3 m and 5 m);
- Thickness of FRP wrapping or steel jacket \( t \) (8 mm, 16 mm, and 24 mm);
- Three strengthening materials: BFRP wrap, CFRP wrap, and steel jacket;
- Strengthening depth $h_s$ (full depth, half depth, and quarter depth).

Other parameters were determined as constants: $a/d = 1.5$ (before strengthening), $N_L = 300$ mm, and $\rho_{fex}=0.75\%$. The benefit of the depth thickening strategy was also investigated.

5.7.1 Punching Shear Behavior

![Graph of Load-displacement curve](image)

Figure 5-39 Load-displacement curve of 3 m original spread footing strengthened with the 8 mm BFRP wraps (Unit: MPa)

All spread footings strengthened with either the FRP wraps or the steel jackets failed in punching shear. The load-displacement curves generated from strengthened footings regarding
different parameters are similar in shape. Therefore, in Figure 5-39, only the results obtained from the 3 m original spread footing strengthened with the 8 mm BFRP wraps are displayed as examples.

The punching shear behavior is similar to that of the original 3 m spread footing. Two stages can be distinguished: a flexural cracking dominating stage (before FSC) and a flexural-shear cracking dominating stage (after FSC). Punching shear failure occurs at PSF when the primary shear crack on the vertical cross-section reaches the upper edge. The maximum stress in the reinforcing steels through the whole process is below the yield strength, which also proves the footing fails in punching shear. The maximum stress in the BFRP wrapping is 92 MPa, which is also smaller than the yield strength of the BFRP wrapping.

5.7.2 Effect of Strengthening Material

Figure 5-40 displays the results generated from 3 m and 5 m footings strengthened with the three systems, and the thickness of strengthening material is 24 mm (largest). The investigations indicate all three systems are effective in improving the punching shear capacity, and the displacements at failure increase in the strengthened footings. For both sizes, the strengthening material has a significant effect on the Enhancement. The greater the stiffness of the strengthening material, the higher the Enhancement. For 3 m footings strengthened with BFRP wraps, CFRP wraps, and steel jackets, the improvements are 25%, 34%, and 47%, which are greater than those in 5 m footings. Therefore, when the same thickness of strengthening material is applied, the improvements in footings with smaller sizes are greater. Similar conclusions were drawn from footings strengthened with 8 mm and 24 mm strengthening materials, and the results are presented in Appendix A.
5.7.3 Effect of Thickness $t$

Three thicknesses were examined: 8 mm, 16 mm, and 24 mm. The results obtained from 3 m spread footing with original depth, strengthened with CFRP wraps and steel jackets, regarding different thicknesses are selected as examples to be discussed in Figure 5-41.

As passive systems, the effect of thickness on Enhancement is not significant. Increasing the thickness slightly increases the Enhancement on the punching shear capacity.

5.7.4 Effect of Strengthening Depth

To examine the effect of strengthening depth on the improvement of punching shear capacity, half-depth strengthening and quarter-depth strengthening were compared with the full-
depth strengthening (when thickness is 8 mm), and the total cross-sectional area was kept as the same. In the CFRP wrapping system, half-depth was studied, and the thickness of CFRP wrap was 16 mm. In the steel jacketing system, both half-depth and quarter-depth were studied, and the thickness of steel jackets were 16 mm and 32 mm, respectively. Comparisons in both systems indicate the effect of strengthening depth is not significant. In the CFRP wrappings system (Figure 5-42), footings strengthened with 16 mm half-depth wraps perform slightly greater punching shear capacities, compared with those strengthened with 8 mm full-depth wraps.

In the steel jacketing system (Figure 5-43), for 3 m footings strengthened with 8 mm full-depth and 16 mm half-depth steel jackets, the improvements are almost the same. The improvement is slightly greater when the footing is strengthened with 32 mm quarter-depth steel jackets. For 5 m footings, the closer the total area of steel jacketing to the bottom, the larger the improvement. The footing strengthened with 32 mm quarter-depth steel jacketing presents the largest improvement.
5.7.5 Effect of Depth Thickening

In these three strengthening systems, the benefit of the depth thickening strategy was also investigated. In Figure 5-44, results obtained from 3 m spread footings with the original depth and the thickened depth, strengthened with 8 mm BFRP wraps and 8 mm steel jackets were compared.

Based on the comparisons, the benefit of the depth thickening strategy in improving the punching shear capacity of spread footings is significant. For footings strengthened with 8 mm BFRP wraps, the enhancement of punching shear capacity rises from 12% to 43% after the depth thickening.
is thickened. For footings strengthened with 8 mm steel jackets, the benefit is even higher, and the enhancement of punching shear capacity rises from 27% to 76% after the depth is thickened.

5.7.6 State of Stress at Contact Surface

In order to check the effectiveness of the connection at the contact surface between the existing and additional concrete segments, the state of stress at the contact surface was investigated. For spread footings strengthened with these three passive systems, 3 m and 5 m spread footings strengthened with 8 mm BFRP wraps were most critical. Therefore, they were selected to be discussed, and the surfaces to be examined are shown in Figure 5-45.

5.7.6.1 3 m spread footing (original depth) strengthened with 8 mm BFRP wraps

The results for 3 m spread footing strengthened with 8 mm BFRP wraps are displayed in Figure 5-46, and only tensile stresses ($\sigma > 0$) are displayed in colors. At the beginning of loading, concrete at the bottom of the contact surface is in tension. State of stress becomes disordered around Point FC, due to cracking in the concrete. The connection at the contact surface is effective as the tensile stress is relatively small during the whole loading process.
5.7.6.2 5 m spread footing (original depth) strengthened with 8 mm BFRP wraps

The results for 5 m spread footing strengthened with 8 mm BFRP wraps are displayed in Figure 5-47. At punching shear failure, the tensile stress is only 1.9 MPa.

5.7.7 Comprehensive Parametric study

To comprehensively study all the parameters considered in these three systems, in this section, the enhancements on the punching shear capacity of spread footings regarding different parameters are displayed.
The results for 3 m and 5 m strengthened spread footings regarding different parameters are shown in Figure 5-48.

![Enhancements on punching shear capacity of (a) 3 m and (b) 5 m spread footings strengthened with the CEBW, CECW and CESJ, regarding different thicknesses](image)

Based on the comparisons in the two figures, the following conclusions can be drawn:

- All the three passive systems are effective in improving the punching shear capacity of the spread footings;
- The footing strengthened with a stiffer material presents a greater improvement on the punching shear capacity, and the differences are significant;
- As passive systems, the effect of thickness $t$ on improving the punching shear capacity is not significant;
- The closer the strengthening materials to the bottom, the slightly greater the improvement;
- When the same material and the same thickness are applied, the larger the footing size, the smaller the improvement.

### 5.8 Summary and Conclusion

In this chapter, the plan of modelling for investigating the five proposed strengthening systems was introduced, with the parameters investigated being clarified. Furthermore, the results
obtained from the finite element analyses using ABAQUS were discussed in detail.

All footings, including original and strengthened footings, failed in punching shear. To understand the punching shear behavior, the load-displacement curves and the states of Max. principal strain were investigated.

- For original spread footings, two stages can be distinguished: a flexural dominating stage and a flexural-shear cracking dominating stage. In the first stage, flexural cracks occur and propagate on the bottom surface, and a distinct decrease of the slope can be observed when a primary flexural crack reaches to the edge of the surface. In the second stage, a primary inclined shear crack gradually forms on the vertical section. It generates from the bottom of the section, propagating to the edge of the column stub. Punching shear failure occurs when the primary shear crack reaches to the upper edge of the section.

- For spread footings strengthened with the CEP system, the punching shear behavior is complicated, and it is significantly affected by the number of prestressing strands. For 3 m spread footing with $\rho_{\text{flex}} = 0.75\%$, when 10 strands are applied, the behavior is similar to that of the original footing, and the strands yield before the punching shear failure. When 30 strands are applied, the stage in which flexural cracking dominates is shorter. Shear cracking, instead of flexural-shear cracking dominates in the second stage. The punching shear failure occurs when a primary shear crack propagates to the location of strands. When 50 strands are applied, the whole process is dominated by shear cracking. Although the primary shear crack is still propagating towards the location of strands, the punching shear failure does not occur when it reaches the location. Instead, it occurs until the flexural reinforcing
steels reach the yield strength.

- For spread footings strengthened with the four passive systems, the punching shear behaviors are all similar to those of the original footings.
- Compared to spread footings, the shear cracking dominating in pile caps is more significant.

After the punching shear behaviors were discussed, for each strengthening system, a parametric study was completed. In summary, the following conclusions were drawn.

- All systems are effective in improving the punching shear capacity of RC footings. However, as an active system, the CEP system is more effective, compared to the four passive systems: CERR, CEBW, CECW and CESJ.
- In the CEP system, the number of prestressing strands has the most significant influence on the improvement of the punching shear capacity. The higher the number of strands, the larger the improvement. Other parameters such as the notch length and the eccentricity have relatively slighter influence.
- In the four passive systems, the investigated parameters, such as the area of the regular reinforcing steels and the thickness of either FRP wrapping or steel jacket, have only slight influences on the improvement of the punching shear capacity.
- The investigation of the shear-span to depth ratio indicates that the larger this parameter, the smaller the failure load or the punching shear capacity.
- The investigation of the footing size indicates that when the same strengthening material is applied, the larger the footing size, the smaller the improvement on the punching shear capacity.
- The investigation of the type of footing indicates that the load type has a significant
influence on the punching shear capacity. The CEP system is more effective in improving the punching shear capacity of spread footings.
6 ANALYTICAL MODEL FOR ORIGINAL RC FOOTINGS

6.1 Introduction

To develop an analytical model (semi-empirical model equation) which can predict the punching shear capacity of the strengthened RC footings, two steps were carried out:

1. The analytical model for original spread footings was determined by validating the models adopted by ACI 318-14, Eurocode 2 and Model Code 2010 with FEA results of original footings;

2. The analytical models for strengthened RC footings were developed from the model determined in Step 1, with considering the parameters investigated in each system.

In Chapter 2, mechanical and empirical models of punching shear were reviewed. The mechanical models are developed based on equilibrium equations, and they are more easily accepted because of their theoretical bases. However, since more efforts are required to carry out the analysis (e.g., the determinations of parameters often require iterations), applications of these mechanical models are relatively impractical. Therefore, in many codes, such as ACI 318-14, AASHTO Code 2012, and Eurocode 2, designs of punching shear are based on empirical models.

In this chapter, the empirical models for punching shear adopted by ACI 318-14, Eurocode 2, and the mechanical model adopted by Model Code 2010 were reviewed. As the model in the AASHTO Code (2012) is identical to that in ACI 318-14, it is not discussed herein. Finite element models of the original spread footings with a shear-span to depth ratio $a/d$ of 1.5, regarding different sizes $B$ (3 m, 4 m, and 5 m) and different reinforcement ratios $\rho_{fex}$ (0.75% and 1.50%) are built and analyzed by using ABAQUS. The validations of the models are accomplished by comparing the results obtained from the finite element analysis with the ones estimated by different empirical models.
As shear reinforcements are not considered in this research, only the analytical models without shear reinforcements are reviewed and discussed in this section. Besides, it is noteworthy that the analytical models considering the prestressing effect are not addressed in this section, and they will be further discussed in Chapter 7.

6.2 ACI 318-14 (2014)

6.2.1 Analytical Model

The prototype of the formula in ACI 318-14 was proposed by Moe (1961), shown as:

\[ v_n = \frac{v_u}{bd} = \left[ 15 \left( 1 - 0.075 \frac{b_c}{d} \right) - 5.25\phi_0 \right] \sqrt{f'_c} \]  

(Equation 6-1)

where \( d \) is the effective depth; \( b \) is the perimeter of the column; \( b_c \) is the side length of the column; \( \phi_0 = \frac{v_u}{V_{flex}} \); \( V_u \) is the shear capacity; \( V_{flex} \) is the ultimate shear force for the flexure failure.

The equation was calibrated with the test results, shown in Figure 6-1.

![Figure 6-1 Design equations compared with test data (ACI-ASCE Committee 326, 1962)](image)

In 1962, the ACI-ASCE Committee 326 submitted a report about shear and diagonal tension. In that report, they reviewed the work completed by Moe (1961). They addressed that \( \phi_0 \)
was not an important variable in a practical design. Also, it was not economical to control the shear capacity by the flexural capacity. Accordingly, they recommend $\phi_0 = 1.0$, and the original equation became:

$$v_n = \frac{v_u}{bd} = \left(9.75 - 1.125 \frac{b_c}{d}\right) \sqrt{f_c'}$$  \hspace{1cm} (Equation 6-2)

Furthermore, based on the test results shown in Figure 6-1, it can be observed that the shear resistance $v_n$ approaches $4\sqrt{f_c'}$ when $b_c/d$ approaches infinity. To ensure a lower bond estimation, the equation was further modified as:

$$v_n = \frac{v_u}{bd} = 4\left(\frac{d}{b_c} + 1\right) \sqrt{f_c'}$$  \hspace{1cm} (Equation 6-3)

Also, in that report, the critical sections for punching shear were firstly determined to be at a distance $d/2$ to the edges of the column.

For a square column, the perimeter of the critical sections $b_0 = 4(b_c + d)$. Therefore, the final form of the empirical equation was proposed as:

$$v_n = \frac{v_u}{b_0d} = 4\sqrt{f_c'}$$  \hspace{1cm} (psi)  \hspace{1cm} (Equation 6-4)
In 1972, Vanderbilt presented that increasing $b_0/d$ could result in a decrease of the punching shear resistance. To consider this effect, the following expression was developed:

$$v_n = \frac{v_u}{b_0d} = 0.332\sqrt{f'_c} \quad \text{(MPa)} \quad \text{(Equation 6-5)}$$

$$v_n = \frac{v_u}{b_0d} = (2 + \frac{\alpha_s\cdot d}{b_o})\sqrt{f'_c} \quad \text{(psi)} \quad \text{(Equation 6-6)}$$

$$v_n = \frac{v_u}{b_0d} = 0.083(2 + \alpha_s\cdot d)\sqrt{f'_c} \quad \text{(MPa)} \quad \text{(Equation 6-7)}$$

where $\alpha_s$ is 40 for interior columns, 30 for edge columns, and 20 for corner columns. For the interior columns, Equation 6-6 (or 6-7) governs the punching shear resistance when $b_0/d=20$ (or $b_c/d=4$), meaning that the side length of the column exceeds 4 times the effective depth.

In 1974, the ACI-ASCE Committee 426 suggested that the effect of the geometry should be considered, as test results indicated that the Equation 6-4 (or 6-5) became unconservative when the ratio of long side to short side of the column was larger than 2. Therefore, a factor $\beta$ was introduced, with the following equation being developed:

$$v_n = \frac{v_u}{b_0d} = (2 + \frac{4}{\beta})\sqrt{f'_c} \quad \text{(psi)} \quad \text{(Equation 6-8)}$$

$$v_n = \frac{v_u}{b_0d} = 0.083(2 + \frac{4}{\beta})\sqrt{f'_c} \quad \text{(MPa)} \quad \text{(Equation 6-9)}$$

In summary, for RC footings without shear reinforcements, the punching shear resistance defined by ACI 318-14 is:

$$v_{ACI} = \min\left\{ \begin{array}{l}
4\sqrt{f'_c} \\
(2 + \frac{4}{\beta})\sqrt{f'_c} \\
(2 + \frac{\alpha_s\cdot d}{b_o})\sqrt{f'_c}
\end{array} \right\} \quad \text{(psi)} \quad \text{(Equation 6-10)}$$

$$v_{ACI} = \min\left\{ \begin{array}{l}
0.332\sqrt{f'_c} \\
0.083(2 + \frac{4}{\beta})\sqrt{f'_c} \\
0.083(2 + \frac{\alpha_s\cdot d}{b_o})\sqrt{f'_c}
\end{array} \right\} \quad \text{(MPa)} \quad \text{(Equation 6-11)}$$
6.2.2 Validation with FEA Results

To validate the analytical model in ACI 318-14, the ultimate loads estimated from this model were compared with those obtained from the FEA results, as displayed in Figure 6-3.

![Figure 6-3 Validate the analytical model in ACI 318-14 with FEA results](image)

Since the flexural reinforcement ratio is not considered in ACI 318-14, the analytical model was found conservative for the footings with $\rho_{\text{flex}}=1.50\%$, although good estimations were achieved for the footings with $\rho_{\text{flex}}=0.75\%$.


6.3.1 Analytical Model

The analytical model adopted by Model Code 2010 was simplified from the mechanical model proposed by Muttoni (2008, 2012), based on the critical shear crack theory (CSCT). In the CSCT, the punching shear capacity is related to the slab rotation $\psi$. Therefore, this model can be regarded as an upgraded model of the mechanical model proposed by Kinnunen & Nylander (1960).

When a critical shear crack occurs and propagates along the theoretically inclined compression strut (Figure 6-4), the shear transfer action is weakened, resulting in the decrease of the punching shear capacity.
The formulas for punching shear capacity consist of a failure criterion and a load-rotation relationship, and the punching shear capacity is defined as the intersection of the curves of the two formulas (Figure 6-5).

To develop the failure criteria, the width of the critical crack is assumed to be proportional to the term $\psi d$ (Figure 6-4). The ability of shear transfer across the critical crack is considered to be related to the roughness of the crack, and it can be expressed by the maximum aggregate size $d_g$. The failure criterion is:

$$\frac{V_R}{b_0d^2\sqrt{f_c}} = \frac{3/4}{1+15\frac{\psi d}{d_{g0}+d_g}} \quad (N, mm)$$  \hspace{2cm} (Equation 6-12)

where $d_{g0}$ is a reference size, and $d_{g0} = 16$ mm; $d_g$ is the maximum aggregate size.

The failure criterion was compared with test results, and good agreements were achieved.
Moreover, the comparison with the formula in ACI 318-05 indicated that for most of the test results, the estimations made by ACI 318-05 were conservative.

For axisymmetric cases, the load-rotation relationship can be developed by the numerical integration of the moment-curvature relationship, based on the following simplifications:

- The radial moment inside the critical shear crack ($r_y < r_0$) is assumed to be constant;
- The radial curvature/moment outside the critical crack ($r_y > r_0$) decreases rapidly;
- The rotation $\psi$ of the slab portion outside the critical crack is assumed to be constant;
- The force in the reinforcement is assumed to be constant.
Considering a quadrilinear relationship between the moment and curvature (shown by the solid line in Figure 6-8), the fully developed formula for the load-rotation relationship can be expressed as:

\[
V = \frac{2\pi}{r_q-r_c} \left\{ -m_r r_0 + m_R (r_y - r_0) + EI_1 \psi \left[ \ln(r_1) - \ln(r_y) \right] + EI_1 X_{TS} (r_1 - r_y) + m_{cr} (r_{cr} - r_1) + EI_0 \psi \left[ n(r_s) - \ln(r_{cr}) \right] \right\}
\]

(Equation 6-13)

where \( m_r \) is the radial moment per unit length at \( r_y = r_0 \); \( \langle x \rangle \) is the operator, and

\[
\langle x \rangle = \begin{cases} 
  x, & x \geq 0 \\
  0, & x < 0 
\end{cases}
\]

(Equation 6-14)
If both tensile strength of concrete and tension stiffening are neglected, the quadrilinear moment-curvature relationship can be simplified to a bilinear relationship (shown by the dotted line in Figure 6-8), and the previous formula could be simplified as:

\[
V = \frac{2\pi}{r_q - r_c} E I_1 \psi \left( 1 + \ln \left( \frac{r_s}{r_0} \right) \right) \quad \text{for} \quad r_y \leq r_0 \quad \text{(elastic phrase)}
\]

(Equation 6-15)

\[
V = \frac{2\pi}{r_q - r_c} E I_1 \psi \left( 1 + \ln \left( \frac{r_s}{r_y} \right) \right) \quad \text{for} \quad r_0 \leq r_y \leq r_s \quad \text{(inelastic phrase)}
\]

(Equation 6-16)

For design purposes, in Model Code 2010, both the failure criteria and the load-rotation relationship are further simplified, and the critical sections are also defined at a distance \(d/2\) to the edges of the column (Figure 6-9).

![Figure 6-9 Critical sections specified by Model Code 2010](image)

The failure criteria in Model Code 2010 is:

\[
\frac{V_{MC}}{b_0 d \sqrt{f_c}} \leq \frac{1}{1.5 + 0.6 \psi d k_{dg}} \leq 0.6 \quad \text{(Equation 6-17)}
\]
where $\gamma_c$ is the partial factor for concrete, and $\gamma_c$=1.5; $k_{dg}$ is a factor considering the maximum aggregate size $d_g$, and $k_{dg}=48/(16+d_g)$ (mm).

Particularly, the load-rotation relationship is determined by a levels-of-approximation (LoA) approach (Figure 6-10). Lower levels offer a preliminary estimation of punching shear capacity, with less time but less accuracy. Higher levels are time-consuming, but they provide a better estimation with higher accuracy.

![Levels of approximation](image)

**Figure 6-10 Levels-of-approximation approach**

The load-rotation relationships regarding different LoAs are:

**LoA I:** both $m_s$ and $m_R$ are simplified, and

$$\psi = 1.5 \cdot \frac{r_s}{d} \cdot \frac{f_{yd}}{E_s} \cdot \left(1 + \frac{m_s}{m_R}\right)^{1.5} \quad (Equation \ 6-18)$$

**LoA II:** only $m_s$ is simplified, and

$$\psi = 1.5 \cdot \frac{r_s}{d} \cdot \frac{f_y}{E_s} \cdot \left(\frac{m_s}{m_R}\right)^{1.5} \quad (Equation \ 6-19)$$

$$m_s = V_e \left(\frac{1}{b} + \frac{e_u}{2b_s}\right) \quad (Equation \ 6-20)$$

**LoA III:** linear-elastic analysis is performed; both $m_s$ and $m_R$ need to be calculated by integration:

$$\psi = 1.2 \cdot \frac{r_s}{d} \cdot \frac{f_y}{E_s} \cdot \left(\frac{m_s}{m_R}\right)^{1.5} \quad (Equation \ 6-21)$$
LoA IV: the load-rotation behavior of the slab is investigated by the integration of the moment-curvature curve.

In the above equations, \( r_s \) is the distance from the column axis to the edge of the footing; \( m_s \) is the average moment per unit length; \( m_R \) is the average flexural strength per unit length.

### 6.3.2 Validation with FEA Results

In this research, only the LoA II in Model Code 2010 is validated with the FEA results. As stated by Clément et al. (2013), for footings without unbalanced moments, \( m_{sd} \) can be estimated as \( V/8 \). Accordingly, the load-rotation relationship in LoA II becomes:

\[
\psi = 1.5 \cdot \frac{r_s}{d} \cdot \frac{f_y}{E_s} \cdot \left( \frac{V_{MC}}{0.6 m_R} \right)^{1.5}
\]  

(Equation 6-22)

And the failure criteria is:

\[
\frac{V_{MC}}{b_0 d - \frac{\sqrt{r_t}}{\gamma_c}} \leq 0.6
\]  

(Equation 6-23)

For 3 m spread footings with \( a/d = 1.5 \), regarding two flexural reinforcement ratios 0.75% and 1.50%, the process to obtain the ultimate load is shown in Figure 6-11.

![Figure 6-11](image_url)

Figure 6-11 Validate the analytical model in Model Code 2010 with FEA results from 3 m original spread footings.
According to Figure 6-11(b), a good estimation is achieved for the 3 m spread footing with $\rho=0.75\%$. However, the estimation is obviously conservative for the 3 m spread footing with $\rho=1.50\%$.

For 4 m and 5 m spread footings, the results are shown in Figure 6-12 and Figure 6-13, respectively. The estimations from the analytical model for both flexural reinforcement ratios are obviously lower than the FEA results.

Figure 6-12 Validate the analytical model in Model Code 2010 with FEA results from 4 m original spread footings

Figure 6-13 Validate the analytical model in Model Code 2010 with FEA results from 5 m original spread footings

6.4.1 Analytical Model

The analytical model adopted by Eurocode 2 (2004) was derived from the Model Code (1993), and two different control sections for punching shear were specified (Figure 6-14).

![Diagram showing control sections for punching shear](image)

Figure 6-14 Control sections for punching shear specified by Eurocode 2

The first control section is defined at the face of the column (shown as Control section I), for the purpose of checking the maximum punching shear resistance:

\[ v_{EC2,\text{cot}} = \frac{V_{EC2,\text{cot}}}{b_0,\text{cot}a} \leq v_{Rd,max} \quad (Equation \ 6-24) \]

\[ v_{Rd,max} = 0.4f_{cd} \quad (Equation \ 6-25) \]

where, \( v_{Rd,max} \) is the maximum punching shear resistance; \( v \) is a reduction factor of the compressive strength of cracked concrete, and

\[ v = 0.6(1 - f_c'/250) \quad (Equation \ 6-26) \]

The second control section is defined at a critical distance \( a_{cri} \) to the edge of the column (shown as Control section II), and \( a_{cri} < 2.0d \). According to Eurocode 2, \( a_{cri} \) should be determined
iteratively. The design formula for punching shear resistance on the critical sections is:

\[ v_{EC2} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho \cdot f'_c)^{1/3} \cdot \frac{2d}{a_{cri}} \]  

(Equation 6-27)

where, \( f'_c \) is the concrete compressive strength; \( C_{Rd,c} \) is an empirical factor, recommended as 0.18/\( \gamma_c \); \( \gamma_c \) is the partial safety factor for concrete, and \( \gamma_c = 1.5 \); \( k \) is the factor considering the size effect of the effective depth \( d \), and:

\[ k = 1 + \left( \frac{200}{d} \right)^{1/2} \leq 2.0 \]  

(Equation 6-28)

6.4.2 Validation with FEA Results

To validate the analytical model in Eurocode 2, two critical distances are investigated: \( a_{cri} = d \) and \( a_{cri} = d/2 \). The results of 3 m, 4 m, 5 m original footings with different reinforcement ratios, regarding different critical distances are displayed in Figure 6-15, Figure 6-16, Figure 6-17, respectively.

![Figure 6-15 Results of 3 m original footings regarding different critical distances](image)

(a) \( a_{cri} = d \)  

(b) \( a_{cri} = d/2 \)

Figure 6-15 Results of 3 m original footings regarding different critical distances
Unlike the analytical model discussed in ACI 318-14, the analytical model in Eurocode 2 accounts for the effect of the flexural reinforcement ratio. Based on the comparisons, it is apparent that the analytical models with $a_{cri}=d/2$ are more appropriate for the original footings with different sizes. When $a_{cri}=d/2$, the ultimate loads obtained from finite element analysis is slightly larger than the estimations from the analytical model. Whereas the analytical model with $a_{cri}=d$ is conservative, and the ultimate loads obtained from finite element analysis are obviously larger than the estimations from the analytical model.
6.5 Summary and Conclusion

In this chapter, the empirical models in ACI 318-14, AASHTO Code 2012, and Eurocode 2, and the mechanical model based on CSCT in Model Code 2010 were reviewed. For each analytical model, the validation was accomplished by comparing the estimations from the analytical model with the results from the finite element analysis. Based on the investigations, the following conclusions were drawn:

- Both Eurocode 2 and Model Code 2010 accounted for the effect of the flexural reinforcement ratio. Therefore, the estimations from these two models were more reasonable, compared to the ones made by ACI 318-14 and AASHTO Code 2012, where the flexural reinforcement ratio was not considered.

- In Model Code 2010, the ultimate load was determined as the intersection of two curves: the failure criteria and the load-rotation relationship, which was relatively difficult to be applied. Besides, the validations indicated that this analytical model was conservative, especially for the footings with a higher reinforcement ratio ($\rho=1.50\%$).

- In Eurocode 2, when the critical distance $a_{cr}=d/2$, the estimations were only slightly higher than the FEA results. The validations also indicated that this analytical model can accurately represented the effects of both the footing size and the flexural reinforcement ratio. Therefore, the analytical model in Eurocode 2 is selected, and it will be further developed in Chapter 7, to consider the improvement made by different strengthening systems.
7 ANALYTICAL MODELS FOR STRENGTHENED RC FOOTINGS

7.1 Introduction

In Chapter 6, analytical models without considering the prestressing effect in ACI 318-14, AASHTO Code 2012, Eurocode 2, and Model Code 2010 were reviewed. Based on the investigations, the empirical model in Eurocode 2 was determined to be utilized as the fundamental analytical model in this research, as the estimations obtained yielded the best agreements with the FEA results of the original spread footings. The fundamental model is expressed by the following equation:

\[ v_{EC2} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho \cdot f'_{c})^{\frac{1}{2}} \cdot \frac{2d}{a_{cri}} \]  
(Equation 7-1)

where, \( f'_{c} \) is the concrete compressive strength; \( C_{Rd,c} \) is an empirical factor, recommended as \( 0.18/\gamma_{c} \); \( \gamma_{c} \) is the partial safety factor for concrete, and \( \gamma_{c} = 1.5 \); \( k \) is the factor considering the size effect of the effective depth \( d \), and:

\[ k = 1 + \left( \frac{200}{d} \right)^{1/2} \leq 2.0 \]  
(Equation 7-2)

The critical distance \( a_{cri} \) is determined as \( d/2 \), according to the conclusion drawn in Chapter 6.

7.2 Developed Analytical Model for the CEP System

7.2.1 Prestressing Effect Considered by Eurocode 2

Specified by Eurocode 2, the empirical formula considering the prestressing effect for a RC slab is:

\[ v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho \cdot f'_{c})^{\frac{1}{2}} + k_{1} \sigma_{cp} \]  
(Equation 7-3)

where, \( \sigma_{cp} \) is the normal concrete stress due to prestressing at the critical section; \( k_{1} \) is specified as a constant, and \( k_{1}=0.1 \).
However, Equation 7-3 was proposed only for RC slabs, and the critical section for this equation is specified as the basic control section, which is 2\(d\) from the edge of the column. Considering the difference between RC slabs and RC footings, the critical distance \(a_{cri}\) for RC footings was determined as \(d/2\), and Equation 7-3 was modified to:

\[
v_{Rd,c} = \left[C_{Rd,c} \cdot k \cdot (100 \cdot \rho \cdot f'_{c})^{\frac{1}{3}} + k_{1}\sigma_{cp}\right] \cdot \frac{2d}{a_{cri}} \quad \text{(Equation 7-4)}
\]

In this dissertation, \(\sigma_{cp}\) is calculated by using the following equation:

\[
\sigma_{cp} = \frac{A_{strand}^2 \cdot N_{p} \cdot \sigma_{pe} \cdot \sin(\theta)}{L_{cri} \cdot d_{e}} \quad \text{(Equation 7-5)}
\]

Taking 3 m strengthened spread footing as an example (Figure 7-1), \(A_{strand}\) is the area of a single strand, and \(A_{strand} = 98.71 \text{ mm}^2\); \(N_{p}\) is the number of prestressing strands; \(\sigma_{pe}\) is the effective prestressing stress, and \(\sigma_{pe} = 1379 \text{ MPa}\); \(L_{cri}\) is the length of the critical cross-section.

Figure 7-1 Determine the prestressing stress \(\sigma_{cp}\)

For spread footings strengthened with the CEP system, the punching shear capacities estimated by the original analytical model (Equation 7-4, where \(k_{1}=0.1\)) were compared with the FEA results, shown in Figure 7-2. It indicates the original model is conservative in estimating the
punching shear capacity of spread footings, and $P_{FEA}/P_{SEC2}$ ranges from 1.0 to 2.5.

For pile caps strengthened with the CEP system, the comparison between the results estimated by the original analytical model (Equation 7-4, where $k_1=0.1$) and the FEA results is displayed in Figure 7-3. It indicates the original Eurocode 2 model is mostly conservative in estimating the punching shear capacity of pile caps. The range of $P_{FEA}/P_{EC2}$ is from 0.8 to 1.8.

Both comparisons indicate the original Eurocode 2 model is not competent in estimating the ultimate load of the RC footings strengthened with the CEP system. Besides, the factor $k_1$ in
the original model has not accounted for the parameters investigated in this research, such as footing size, and flexural reinforcement ratio. Therefore, further modifications of the original Eurocode 2 model are required, typically through modifying the factor $k_1$.

### 7.2.2 Developed Analytical Model for Spread Footings Strengthened with the CEP System

For spread footings strengthened with the CEP system, a factor $\eta_{CEP,SF}$ was introduced, replacing the factor $k_1$ in the original Eurocode 2 model. Therefore, the modified analytical model is:

$$v_{CEP,SF} = \left[ C_{Rd,c} \cdot k \cdot \left( 100 \cdot \rho \cdot f'_c \right)^{\frac{1}{3}} + \eta_{CEP,SF} \cdot \sigma_{cp} \right] \cdot \frac{2d}{a_{cri}} \quad (Equation \ 7-6)$$

The proposed factor $\eta_{CEP,SF}$ should be able to account for all the parameters investigated in Chapter 6, including:

- Size of footing $B$ (3 m, 4 m, and 5 m);
- Shear-span to depth ratio $a/d$ (1.5, 2.0, and 2.5);
- Flexural reinforcement ratio $\rho_{flex}$ (0.75%, and 1.5%);
- Eccentricity of prestressing strands $e$ ($e_1$, $e_2$, $e_3$, and $e_4$);
- Number of prestressing strands $N_p$ (10, 20, 30, 40, and 50);
- Notch length $N_L$ (0, 150 mm, and 300 mm).

To consider the effect of the number of prestressing strands, a parameter $\rho_p$ was introduced, instead of the parameter $N_p$, and it was calculated by:

$$\rho_p = \frac{A_{stran}^e \cdot N_p}{D_{enlarged} \cdot d_e} \times 100\% \quad (Equation \ 7-7)$$

where $A_{stran}$ is the area of a single strand; $D_{enlarged}$ is the diameter of the enlarged footing.

The linear regression analysis was employed to study the relationship between the factor $\eta_{CEP,SF}$ and the investigated parameters, with the following formula being derived:
\[
\eta_{\text{CEP, SF}} = 0.1 \cdot \frac{B}{10^3} - 0.2 \cdot \rho_p + 0.2 \cdot \rho_{\text{flex}} + 0.5 \cdot \frac{e}{h} - 0.2 \cdot \frac{a}{d} - 0.1 \cdot \frac{N_L}{10^3} + 0.05
\]

(Equation 7-8)

The results estimated by the formula above were compared with those obtained from the finite element analysis, shown in Figure 7-4. It can be observed that the proposed linear formula is competent in estimating the factor \(\eta_{\text{CEP, SF}}\). Compared to the FEA results, 89% of the estimated results were on the safe side.

![Figure 7-4 Estimated results compared with FEA results for factor \(\eta_{\text{CEP, SF}}\)](image)

Based on the parametric studies completed in Chapter 5, for spread footings strengthened by the CEP system, the relationships between the punching shear capacity and different parameters are as the followings: by setting the other parameters as constants, increasing the footing size \(B\), prestressing ratio \(\rho_p\), flexural reinforcement ratio \(\rho_{\text{flex}}\), eccentricity \(e\), or notch length \(N_L\) will increase the punching shear capacity. However, increasing the shear span to depth ratio \(a/d\) will decrease the punching shear capacity.

### 7.2.3 Developed Analytical Model for Pile Caps Strengthened with the CEP System

For pile caps strengthened with the CEP system, a factor \(\eta_{\text{CEP, PC}}\) was introduced, replacing
the factor $k_1$ in the original Eurocode 2 model. The modified analytical model is:

$$v_{\text{CEP,PC}} = \left[ C_{Rd,e} \cdot k \cdot (100 \cdot \rho \cdot f'_c)^{1/3} + \eta_{\text{CEP,PC}} \cdot \sigma_{cp} \right] \cdot \frac{2d}{a_{cri}} \quad \text{(Equation 7-9)}$$

The proposed factor $\eta_{\text{CEP,PC}}$ should also be able to account for all the parameters investigated in Chapter 6, including:

- Size of footing $B$ (3 m, 4 m, 5 m);
- Shear-span to depth ratio $a/d$ (1.5, 2.0, 2.5);
- Eccentricity of prestressing strands $e$ ($e_1$, $e_2$, $e_3$, $e_4$);
- Number of prestressing strands $N_p$ (10, 20, 30, 40, 50);
- Notch length $N_L$ (0, 150 mm, and 300 mm).

Similarly, the parameter $\rho_p$ was applied to consider the effect of the number of prestressing strands, instead of $N_p$, and it was calculated by Equation 7-7.

The formula for $\eta_{\text{CEP,PC}}$ was also derived by the linear regression analysis, and:

$$\eta_{\text{CEP,PC}} = -0.08 \cdot \frac{B}{10^3} - 0.2 \cdot \rho_p - 0.15 \cdot \rho_{fle x} + 1.16 \cdot \frac{e}{h} - 0.1 \cdot \frac{a}{d} - 0.16 \cdot \frac{N_L}{10^3} + 0.63$$

\begin{align*}
\text{(mm)} & \quad \text{%} & \quad \text{%} & \quad \text{(mm)} \\
\text{(Equation 7-10)}
\end{align*}

The results estimated by the formula above were compared with the ones obtained from the finite element analysis, shown in Figure 7-5. It can be observed that the proposed linear formula is able to predict the factor $\eta_{\text{CEP,PC}}$. Compared to the FEA results, 79.3% of the estimated results are on the safe side.

Based on the parametric studies completed in Chapter 5, for pile caps strengthened by the CEP system, the relationships between the punching shear capacity and different parameters are the followings: by setting the other parameters as constants, increasing the footing size $B$, prestressing ratio $\rho_p$, flexural reinforcement ratio $\rho_{fle x}$, eccentricity $e$, or notch length $N_L$ will
increase the punching shear capacity. However, increasing the shear span to depth ratio $a/d$ will decrease the punching shear capacity.

![Figure 7-5 Estimated results compared with FEA results for factor $\eta_{CEP, PC}$](image)

Please note that the two equations proposed in this section are applicable for square RC column footings with: (a) a minimum concrete compressive strength 35 MPa; (b) all strands are the Grade 270 seven-wire, low relaxation steel strands (PCI Design Handbook, 2014); and (c) the yield strength of the flexural reinforcements is 420 MPa.

### 7.3 Developed Analytical Model for the Four Passive Systems: CERR, CEBW, CECW and CESJ

For spread footings strengthened with the four passive systems (CERR, CEBW, CECW, and CESJ), no relevant provisions can be found in Eurocode 2. The punching shear capacities estimated by the original analytical model were compared with the finite element results, shown in Figure 7-6. The investigation indicates the original analytical model cannot accurately predict the punching shear capacities of the strengthened footings.
7.3.1 Developed Analytical Model for RC Footings Strengthened with the CERR System

For spread footings strengthened with the CERR system, a factor $\eta_{CERR}$ was introduced, and the modified analytical model is expressed as:

$$v_{CERR} = \eta_{CERR} \cdot v_{EC2} = \eta_{CERR} \cdot C_{Rd,cr} \cdot k \cdot (100 \cdot \rho \cdot f_y)^{1/3} \cdot \frac{2d}{a_{cri}}$$  \hspace{1cm} (Equation 7-11)

The factor $\eta_{CERR}$ should be able to account for the two parameters investigated in the CERR system, including:

- Footing size $B$ (3 m and 5 m);
- Area of the circular regular reinforcing steel $A_{rr}$ (EQ10 and EQ30)

The effect of the area of the circular regular reinforcing steel was considered by using the regular reinforcement ratio $\rho_{rr}$, instead of $A_{rr}$, and it was calculated by:

$$\rho_{rr} = \frac{A_{rr}^2}{d_{enlarged}d_c} \times 100\%$$  \hspace{1cm} (Equation 7-12)

The formula for $\eta_{CERR}$ was derived by the linear regression analysis, and:

$$\eta_{CERR} = -0.02 \cdot \frac{B}{10^3} + 0.4 \cdot \rho_{rr} + 1.24$$  \hspace{1cm} (Equation 7-13)

$(\text{mm})$ $(\%)$
The results estimated by the formula above were compared with the FEA results, shown in Figure 7-7. It can be observed that the proposed linear formula is able to predict the factor $\eta_{CERR}$. Compared to the FEA results, all the estimated results are on the safe side.

![Figure 7-7 Estimated results compared with FEA results for factor $\eta_{CERR}$](image)

Based on the parametric studies completed in Chapter 5, for spread footings strengthened by the CERR system, by setting the other parameter as a constant, increasing the footing size $B$ or the area of the regular reinforcements ratio $\rho_{rr}$ will increase the punching shear capacity.

### 7.3.2 Developed Analytical Model for RC Footings Strengthened with CEBW, CECW, and CESJ

For spread footings strengthened with CEBW, CECW, and CESJ, a factor $\eta_{Tri}$ was introduced to present the improvement on the punching shear resistance, and the modified analytical model is expressed as:

$$v_{Tri} = \eta_{Tri} \cdot v_{EC2} = \eta_{Tri} \cdot C_{Rd,E} \cdot k \cdot (100 \cdot \rho \cdot f'c)^{\frac{1}{3}} \cdot \frac{2d}{a_{cri}} \quad (Equation \ 7-14)$$

The factor $\eta_{Tri}$ should be able to account for the parameters investigated in the three systems, including:
- Footing size $B$ (3 m and 5 m);
- Strengthening materials (BFRP, CFRP, steel)
- Thickness of FRP wrap or steel jacket $t$ (8 mm, 16 mm, 24 mm);
- Strengthening depth (full-depth, half-depth, and quarter-depth).

The effect of the strengthening material was considered by its elastic modulus $E_{ela}$. For the BFRP and CFRP, $E_{ela} = E_1$, and $E_1$ is the elastic modulus in the circumference direction where the fibers orient. $E_{ela} = 37.7$ GPa and 165 GPa, for BFRP and CFRP, respectively. The elastic modulus of the steel jacket is 200 GPa. The strengthening depth $h_s$ is represented by the ratio $r_s = h_s/h$ (1, 0.5, and 0.25).

In addition, as depth thickening was also investigated in three systems, shear-span to depth ratio $a/d$ was involved in the formula to consider this effect.

Based on linear regression analysis, the formula for $\eta_{Tri}$ was derived:

$$
\eta_{Tri} = -0.05 \cdot \frac{B}{10^3} - 0.3 \cdot \frac{a}{d} + 1.21 \cdot \frac{E_{ela}}{10^6} + 0.01 \cdot t + 0.16 \cdot r_s + 1.4 \quad (Equation \ 7-15)
$$

The results estimated by the formula above were compared with the FEA results, shown in Figure 7-8. It can be observed that the proposed linear formula is competent in estimating the factor $\eta_{Tri}$. Compared to the FEA results, 80.6% of the estimated results are on the safe side.

Based on the parametric studies completed in Chapter 5, for spread footings strengthened by CEBW, CECW and CESJ systems, by setting the other parameters as constants, increasing the footing size $B$, the elastic modulus $E_{ela}$ or the thickness $t$ of the strengthening material, or the strengthening depth $r_s$ will increase the punching shear capacity. However, increasing the shear span to depth ratio $a/d$ will decrease the punching shear capacity.
Summary and Conclusion

In this chapter, for each strengthening system, an analytical model was proposed. All analytical models were developed from the original analytical model suggested by Eurocode 2.

For RC footings strengthened with the CEP system, different analytical models were proposed for spread footings and pile caps. The formula considering the prestressing effect in Eurocode 2 was modified by replacing the factor $k_1$ with $\eta_{CEP, SF}$ and $\eta_{CEP, PC}$, respectively. Based on the linear regression analysis, the formulas to determine these two factors are:

The $\eta_{CEP, SF}$ for spread footings:

$$\eta_{CEP, SF} = 0.1 \cdot \frac{B}{10^3} - 0.2 \cdot \rho_p + 0.2 \cdot \rho_{flex} + 0.5 \cdot \frac{e}{h} - 0.2 \cdot \frac{a}{d} - 0.1 \cdot \frac{L_T}{10^3} + 0.05$$

\((mm) \quad (%) \quad (%) \quad (mm)\)

\((Equation 7-16)\)

The $\eta_{CEP, PC}$ for pile caps:

$$\eta_{CEP, PC} = -0.08 \cdot \frac{B}{10^3} - 0.2 \cdot \rho_p - 0.15 \cdot \rho_{flex} + 1.16 \cdot \frac{e}{h} - 0.1 \cdot \frac{a}{d} - 0.16 \cdot \frac{L_T}{10^3} + 0.63$$

\((mm) \quad (%) \quad (%) \quad (mm)\)

\((Equation 7-17)\)
Only spread footings were strengthened with the four passive systems: CERR, CEBW, CECW, and CESJ. For them, the analytical models were proposed by simply introducing the factor $\eta_{CERR}$ and $\eta_{Tri}$, respectively. Based on the linear regression analysis, the formulas to determine these two factors are:

$$
\eta_{CERR} = -0.02 \cdot \frac{B}{10^3} + 0.4 \cdot \rho_{rr} + 1.24 \quad (Equation \ 7-18)
$$

$$(mm) \quad (\%)$$

and,

$$
\eta_{Tri} = -0.05 \cdot \frac{B}{10^3} - 0.3 \cdot \frac{a}{d} + 1.21 \cdot \frac{E_{ela}}{10^6} + 0.01 \cdot t + 0.16 \cdot r_s + 1.4 \quad (Equation \ 7-19)
$$

$$(mm) \quad (MPa)$$
8 SUMMARY AND CONCLUSIONS

8.1 Summary

The primary objective of this research is to develop several cost-effective solutions than the traditional footing enlargement method, which is impractical as the dowel connections and rebar splicing at the contact surfaces are difficult to be built. Five new footing strengthening systems are proposed, and they are:

- Circular external prestressing system (CEP);
- Circular external regular reinforcement system (CERR);
- Circular external BFRP wrapping system (CEBW);
- Circular external CFRP wrapping system (CECW);
- Circular external steel jacketing system (CESJ).

In the above systems, connections at the contact surfaces are achieved by the composite actions provided by the circular external prestressing (CEP), the circular external regular reinforcements (CERR), the circular external CFRP/ BFRP wraps (CECW/ CEBW), or the circular external steel jackets (CESJ). These connections are reliable and more practical, compared to those in the traditional method. The CEP is an active system, where the connection is activated during the construction process. Whereas, the other four systems are passive systems, where the connections are activated after the external loads are applied.

Finite element analysis is a powerful tool to simulate and analyze the RC structures. Since the accuracy of the simulation highly depends on the quality of inputs, validation of finite element modelling was accomplished by comparing the FEA results with the experimental results, and good agreements were achieved. Afterwards, a total of 353 models were built, with different sets of parameters being considered in the five systems.
The parameters investigated in the CEP system include:

- Footing type (spread footing and pile cap);
- Size of footing $B$ (3 m, 4 m, and 5 m);
- Shear-span to depth ratio $a/d$ (1.5, 2.0, and 2.5);
- Flexural reinforcement ratio $\rho_{\text{flex}}$ (0.75% and 1.50%);
- Notch length $N_L$ (300 mm, 150 mm, and 0);
- Number of prestressing strands $N_p$ (10, 20, 30, 40, and 50);
- Eccentricity of prestressing strands $e$ ($e_1$, $e_2$, $e_3$, and $e_4$), where $e_1$ and $e_4$ are the maximum eccentricity and minimum eccentricity, respectively.

The parameters investigated in the CERR system include:

- Size of footing $B$ (3 m and 5 m);
- Equivalent area of regular reinforcing steel $A_{rr}$ (two values were considered, which are equivalent to 10 and 30 strands, respectively).

The parameters investigated in CEBW, CECW and CESJ:

- Size of footing $B$ (3 m and 5 m);
- Thickness of FRP wrapping or steel jacket $t$ (8 mm, 16 mm, and 24 mm);
- Strengthening materials (BFRP, CFRP, and steel);
- Strengthening depth (full-depth, half-depth, and quarter-depth).

All models, including original and strengthened RC footings, failed in punching shear. For each strengthening system, load-displacement curves of several models are selected as examples for discussion, for the purpose of understanding the punching shear behavior. Afterwards, parametric studies were carried out to investigate the influences of the parameters on punching shear capacity.
Analytical models in ACI 318-14, AASHTO Code 2012, Eurocode 2, and Model Code 2010 were reviewed. The empirical model in Eurocode 2 with the critical distance $a_{cri} = d/2$ was determined as the basic analytical model, as the results calculated using this model for the original spread footings yielded the best agreements with the FEA results. Afterwards, for each strengthening system, an analytical model was proposed, based on linear regression analysis, with the primary parameters investigated in each system being considered. The analytical models for the five proposed systems are as follows:

For the CEP system, the shear capacity at the critical section ($a_{cri} = d/2$) is expressed as:

$$v_{CEP} = \left[ C_{Rd,c} \cdot k \cdot (100 \cdot \rho \cdot f_c')^{\frac{1}{2}} + \eta_{CEP} \cdot \sigma_{cp} \right] \cdot \frac{2d}{a_{cri}} \quad \text{(Equation 8-1)}$$

where, for spread footings:

$$\eta_{CEP, SF} = 0.1 \cdot \frac{B}{10^3} - 0.2 \cdot \rho_p + 0.2 \cdot \rho_{flext} + 0.5 \cdot \frac{c}{h} - 0.2 \cdot \frac{a}{d} - 0.1 \cdot \frac{N_t}{10^3} + 0.05$$

(mm)   (%)    (%)    (mm)   (Equation 8-2)

And, for pile caps:

$$\eta_{CEP, PC} = -0.08 \cdot \frac{B}{10^3} - 0.2 \cdot \rho_p - 0.15 \cdot \rho_{flext} + 1.16 \cdot \frac{c}{h} - 0.1 \cdot \frac{a}{d} - 0.16 \cdot \frac{N_t}{10^3} + 0.63$$

(mm)   (%)    (%)    (mm)   (Equation 8-3)

For the CERR system, the shear capacity at the critical section ($a_{cri} = d/2$) is expressed as:

$$v_{CERR} = \eta_{CERR} \cdot C_{Rd,c} \cdot k \cdot (100 \cdot \rho \cdot f_c')^{\frac{1}{2}} \cdot \frac{2d}{a_{cri}} \quad \text{(Equation 8-4)}$$

$$\eta_{CERR} = -0.02 \cdot \frac{B}{10^3} + 0.4 \cdot \rho_{tr} + 1.24 \quad \text{(Equation 8-5)}$$

(mm)   (%)    

For CEBW, CECW and CESJ, the shear capacity at the critical section ($a_{cri} = d/2$) is
expressed as:

\[ v_{Tri} = \eta_{Tri} \cdot C_{Ra,c} \cdot k \cdot (100 \cdot \rho \cdot f_{c}^{d})^{1/3} \cdot \frac{2d}{a_{cri}} \]  \hspace{1cm} (Equation 8-6)

\[ \eta_{Tri} = -0.05 \cdot \frac{B}{10^3} - 0.3 \cdot \frac{a}{d} + 1.21 \cdot \frac{E_{ela}}{10^6} + 0.01 \cdot t + 0.16 \cdot r_s + 1.4 \]  \hspace{1cm} (Equation 8-7)

8.2 Conclusions

According to the finite element analyses on the five proposed strengthening systems, the following conclusions were drawn:

8.2.1 Punching Shear Behavior

All models investigated in this dissertation failed in punching shear.

- For original spread footings and pile caps, two stages can be distinguished: a flexural dominating stage and a flexural-shear cracking dominating stage. In the first stage, flexural cracks occur and propagate on the bottom surface, and a distinct decrease of the slope can be observed when a primary flexural crack reaches to the edge of the surface. In the second stage, a primary inclined shear crack gradually forms on the vertical section. It generates from the bottom of the section, propagating to the edge of the column stub. Punching shear failure occurs when the primary shear crack reaches the upper edge of the section.

- For spread footings strengthened with the CEP system, the punching shear behavior is complicated, and it is significantly affected by the number of prestressing strands. For 3 m spread footing with \( \rho_{flex} = 0.75\% \), when 10 strands are applied, the behavior is similar to that of the original footing, and the strands yield before the punching shear failure. When 30 strands are applied, the stage in which flexural cracking dominates is shorter. Shear cracking, instead of flexural-shear cracking dominates.
in the second stage. The punching shear failure occurs when a primary shear crack propagates to the location of strands. When 50 strands are applied, the whole process is dominated by shear cracking. Although the primary shear crack is still propagating towards the location of strands, the punching shear failure does not occur when it reaches the location. Instead, it occurs until the flexural reinforcing steels reach the yield strength.

- For spread footings strengthened with the four passive systems, the punching shear behaviors are similar to those of the original footings.
- Compared to spread footings, the shear cracking dominating in pile caps is more significant.

### 8.2.2 Parametric Studies

The following conclusions were drawn based on a series of parametric studies:

- Compared to the other four passive systems, the CEP active system is more effective in improving the punching shear capacity of RC footings.
- In the CEP system, the number of prestressing strands has the most significant influence on the improvement of the punching shear capacity. The higher the number of strands, the higher the improvement. Other parameters such as the notch length and the eccentricity have relatively slighter influence.
- In the four passive systems, CERR, CEBW, CECW, and CESJ, the parameters investigated, such as the area of the regular reinforcing steels and the thickness of the strengthening material, have relatively slighter influences on the improvement of the punching shear capacity, compared to those using the CEP system.
- The investigation of the shear-span to depth ratio indicates that the larger the ratio,
the smaller the failure load or the punching shear capacity.

- The investigation of the footing size indicates that for the same strengthening system used, the larger the footing size, the smaller the improvement of the punching shear capacity.

- The investigation of the type of footing indicates that the load type has a significant influence on the punching shear capacity. The CEP system is more effective in improving the punching shear capacity of spread footings.

8.3 Limitations and Recommendations for Future Study

Finite element analysis is the primary analytical method used in this research. Although it has been proved to be powerful in simulating RC structures, the accuracy of the results highly depends on the quality of the inputs and the experience of the user. As the proposed strengthening systems are innovative, tests for validating the finite element models are not available. Therefore, experimental investigations on the five proposed strengthening systems are strongly suggested.

In this research, the soil pressure underneath original or enlarged RC footing was assumed to be uniform. However, researches conducted by Hegger et al. (2006, 2009) revealed that close to failure, a concentration of soil pressure under the column stub was detected. Therefore, for future work, using a non-uniformly distributed load is suggested, in which the distribution should be expressed by a pre-defined equation.

Since the CEP system can significantly improve the punching shear capacity of RC footings, prestressed FRP sheets or steel jackets should be investigated, as it will probably show a greater improvement.
APPENDIX

Figure A-1 Load-displacement curves of 4 m strengthened spread footings \((a/d = 1.5, N_L = 300\ mm, \text{ and } \rho_{\text{flex}}=0.75\%)\), regarding different \(N_p\) and \(e\)

Figure A-2 Load-displacement curves of 4 m strengthened spread footings \((a/d = 1.5, N_L = 300\ mm, \text{ and } \rho_{\text{flex}}=1.50\%)\), regarding different \(N_p\) and \(e\)

Figure A-3 Load-displacement curves of 5 m strengthened spread footings \((a/d = 1.5, N_L = 300\ mm, \text{ and } \rho_{\text{flex}}=0.75\%)\), regarding different \(N_p\) and \(e\)
Figure A-4 Load-displacement curves of 5 m strengthened **spread footings** \((a/d = 1.5, N_L = 300 \text{ mm}, \text{and } \rho_{\text{flex}} = 1.50\%\)) regarding different \(N_p\) and \(e\)

Figure A-5 Load-displacement curves of 3 m strengthened **pile caps** \((a/d = 1.5, N_L = 300 \text{ mm}, \text{and } \rho_{\text{flex}} = 1.50\%\)) regarding different \(N_p\) and \(e\)

Figure A-6 Load-displacement curves of 4 m strengthened **pile caps** \((a/d = 1.5, N_L = 300 \text{ mm}, \text{and } \rho_{\text{flex}} = 1.50\%\)) regarding different \(N_p\) and \(e\)
Figure A-7 Load-displacement curves of 5 m strengthened pile caps \( (a/d = 1.5, N_L = 300 \text{ mm, and } \rho_{flex} = 1.50\%) \), regarding different \( N_p \) and \( e \)
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Major Area: Civil Engineering
Area Focused: Bridge Engineering
Thesis Topic: Monitoring and control in cantilever construction of a three-span continuous rigid frame railroad bridge on Yarlung Zangbo River
Advisor: Richen Ji, Professor, Ph.D.

WORK EXPERIENCE

Jan. 2020 - May 2020 Teaching assistant in Rehab of Civil Infrastructure (CIE 478/678), Syracuse University
Jan. 2020 - May 2020 Teaching assistant in Finite Element Analysis (CIE 633), Syracuse University
Sep. 2019 - Dec. 2019 Teaching assistant in Transportation Engineering (CIE 443/643), Syracuse University
Jan. 2019 - May 2019 Teaching assistant in Design of Concrete Structures (CIE 332), Syracuse University
Sep. 2018 - Dec. 2018 Course assistant in Statics (ECS 221), Syracuse University
Apr. 2014 - Apr. 2015 Visiting Scholar, Department of Civil and Environmental Engineering, University of Nevada, Las Vegas
Jun. 2013 - Sep. 2013 Internship, National Natural Science Foundation of China (NSFC), Beijing
JOURNAL PUBLICATIONS


CONFERENCE PUBLICATIONS

Aug. 5 - 7, 2021 8th International Conference on Advanced Composite Materials in Bridges and Structures (ACMBS-VIII), Sherbrooke, Quebec, Canada (rescheduled)
Paper title: *Bridge footing strengthening using various FRP systems* (Lu, X., & Aboutaha, R. S.)

Dec. 3 - 6, 2019 16th East Asia-Pacific Conference on Structural Engineering & Construction (EASEC16), Brisbane, Australia
Paper title: *Structural strengthening of concrete footings using external prestressing* (Lu, X., & Aboutaha, R. S.)

Jun. 4 – 7, 2019 14th International Symposium on Fiber Reinforced Polymers for Reinforced Concrete Structures (FRPRCS-14), Queen’s University, Belfast, Northern Ireland, United Kingdom
Paper title: *Structural Strengthening of Concrete Footings Using FRP and External Prestressing* (Lu, X., & Aboutaha, R. S.)

ADDITIONAL RESEARCH PROJECTS

Jun. 2013 - Jun. 2015 The Research on Anti-Seismic Property and Risk Assessment of Long Span Prestressed Concrete Continuous Beam Bridge on the High-Speed Railway, funded by both China Railway Corporation and National Natural Science Foundation of China (NSFC)

Jan. 2013- June 2015 The Research on Key Technology of Retrofitting the Overloaded Freight Passage, founded by Ministry of Transport of the People's Republic of China

Sep. 2012 - Oct. 2012 Load testing of three highway bridges, Inner Mongolia, China

Sep. 2011 - Jun. 2012 Monitoring and control in cantilever construction of a three-span continuous rigid frame railroad bridge on Yarlung Zangbo River

HONORS and AWARDS

- Invited as a visiting scholar to University of Nevada, Las Vegas (Apr. 2014)
- Recommended to graduate study in Beijing Jiaotong University without exam (Sep. 2012)
- College Excellent Master Thesis in Beijing Jiaotong University (May 2015)
- Scholarships of Beijing Jiaotong University (2013, 2014)
- Excellent Graduate of Lanzhou Jiaotong University (2012)
- Member of American Concrete Institute (ACI)
- Member of American Society of Civil Engineers (ASCE)
- Member of Precast/Prestressed Concrete Institute (PCI)
- Member of Concrete Reinforcing Steel Institute (CRSI)

RESEARCH INTERESTS

1. Structural rehabilitation
2. Finite element analysis
3. Earthquake engineering