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Improvements on Design and Analysis of Pile Caps Based on the Strut-and-Tie Method

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by

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Summary:

The strut-and-tie method is an alternative design and analysis method for discontinuity regions like pile caps and deep beams. In the following paper, the history and method is entirely explained using guidelines from ACI. In a second part, the expansion of the method towards three-dimensional members, more specifically pile caps, is investigated with a critical analysis. A list of improvements is presented and discussed.

LID VAN ASSOCIATIE KU LEUVEN

Foreword/Resignation

As a last year master's student in Building Engineering at the KU Leuven University, I took part in the Erasmus exchange program and went to Valencia to study at the Universitat Politècnica de València. There I had the opportunity to write my Master's Thesis during semester B in English. Because of the preliminary background knowledge of concrete design that was obtained through courses at the home university, I chose the subject on the strut-and-tie method. This method is an innovative method that still has some improvements to make, which convinced me to work on it.

This thesis was written together with fellow Belgian student Tijs Vancoillie, who also took part of the Erasmus exchange. To gather the prior knowledge on the strut-and-tie method, we performed a literature study together in the form of an extensive state of the art. Later, two specific subjects were further examined and this thesis represents one of them.

I would like to thank both the KU Leuven University and the Universitat Politècnica de València for the opportunity and the cooperation, professors Luc Vanhooymissen (KUL) and Peter Minne (KUL) who provided us with the necessary background knowledge to understand this topic on concrete calculations and behaviour and Enrique David Llacer (UPV) for guiding us during the entire process and research on the strut-and-tie method.

Anthony Demeyere

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1 Introduction

The strut-and-tie method is a simplified method to analyse and design reinforced concrete structures. The STM is used to design the discontinuity-regions, where the rules of Bernoulli don't satisfy anymore. Examples of such regions are pile caps, corbels, deep beams, etc. Generally these are all the places with a disturbed geometry or regions with discontinuity of loads. The main principle is that loads are carried through the concrete until they reach a supporting point. The model is based on drawing traces of the stresses on the structure and dividing the structure into connected struts, ties and nodes.

The STM for bidimensional problems like deep beams and corbels has already been thoroughly investigated recent years and is very well known today. Experiments show that structures behave pretty much the same as calculated with the strut-and-tie model. However, more difficulties occur when calculating structures where no bidimensional patterns can be followed. For example the connection of a column with pile caps, the joint between beams and columns,…

The purpose of this master thesis is to investigate the most recent insights and research works on applying the strut-and-tie method in three-dimensional models. As one of the most known 3D investigated structures are pile caps and as we found most papers on this structural member, the focus of this study is based only on this structure.

In a first part, the method is generally explained in the form of a state of the art. The different design steps according to ACI are illustrated as well. This knowledge was needed before jumping on to the more complicated three-dimensional proposed methods.

The next step is to investigate the STM-based approaches for pile caps that have been proposed so far. These models are new and still developing at the moment. The aim of this thesis is to provide the readers with a table of improvements and proposals throughout the years. The essentials of each proposed method are explained and discussed in a critical analysis.

As a conclusion of this study, suggestions are presented on what approaches could be the most reliable or interesting to keep on developing three-dimensional models for the design and analysis of pile caps using the strut-and-tie method.

2 Strut-and-Tie method

2.1 History

Structures have been built since the old ages composing of wood, brick, stone and even concrete. The use of concrete dates back several centuries, in the time of the Romans, the middle ages. The real driver for the use of concrete was the Smeaton's tower in the years 1756 to 1759. The engineer John Smeaton first used hydraulic lime in concrete, using pebbles and powdered brick as aggregate. In the late 19th century, Joseph Monier pioneered with the introduction of using steel in combination with concrete, because the concrete containers he made, weren't strong enough. The use of steel in concrete as we know today, is to overcome the low tensile strength of concrete. With the introduction of reinforced concrete, structures could be built with a sufficient compressive and tensile strength.

It's only in the year 1899, when reinforced concrete was still in its infancy, that a researcher developed a model for designing reinforced concrete. This first model was called a truss model and was introduced by Wilhelm Ritter. The truss model was used for the visualization of internal stresses, compression and tension, in the structural element and to define the amount of reinforcement. The model describes that tensile forces would be carried by steel rods (ties) and compressive forces would be carried by the concrete (struts), as can be seen in Figure 2.1. In 1902 researcher Emil Mörsch took Ritter's work and refined his model. Ritter used discrete diagonal forces, but Mörsch refined this observation by saying that the forces are in a continuous field of diagonal compression. The adaptation to Ritter's model can be seen in Figure 2.2.

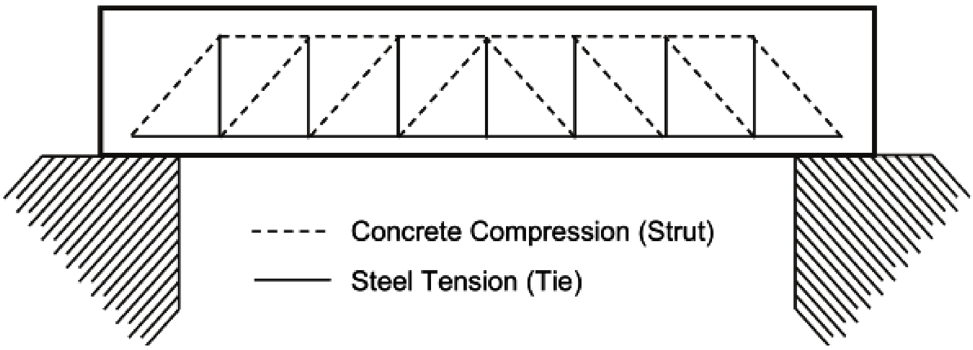

Figure 2.1: Ritter's original truss analogy (Brown, 2005)

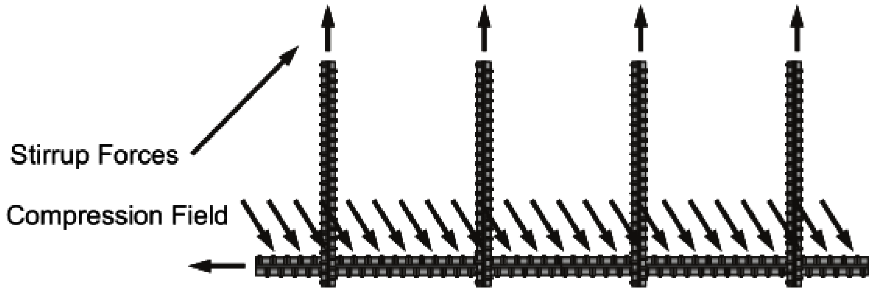

Figure 2.2: Mörsch's adaptation of Ritter's model (Brown, 2005)

This model was further researched by Talbot (1909) and Richart (1927). They studied the effects of shear on the reinforced concrete elements. Talbot discovered that the truss models made overestimations considering the strength of the concrete element. This was due to the neglection of the tensile strength in the truss model. However, the tensile strength of concrete is an important factor when it comes to shear resistance in reinforced concrete elements. Richart further researched this and developed a method of shear design. This method took both steel and concrete contributions into consideration when calculating the shear resistance of the element. The shear resistance was determined by calculating the concrete and steel contribution to shear strength separately and then make the sum of both (V_c+V_s). This method can still be found in the sectional approach.

In the year 1964, Kupfer (1964) expanded the Mörsch's truss analogy by the application of the principle of minimum strain energy. Shortly after, in the year 1965, Kupfer studied the shear reinforcement in concrete beams and slabs and suggested a simple method to reduce the shear reinforcement in those concrete elements.

It was until the early 1970s that the truss analogy, or the now called strut-and-tie method, was really revived in the United States. At that time, a strut-and-tie model was applied for the first time to concrete elements subjected to both shear and torsion. Lambert & Thurlimann (1971) developed an instrument to assess these kind of cases. This instrument consisted of a tubular truss that formed a hollow box around the concrete elements' outside face, see Figure 2.3. This was actually a reinforcing cage, consisting of longitudinal reinforcement, stirrups and concrete compression diagonals.

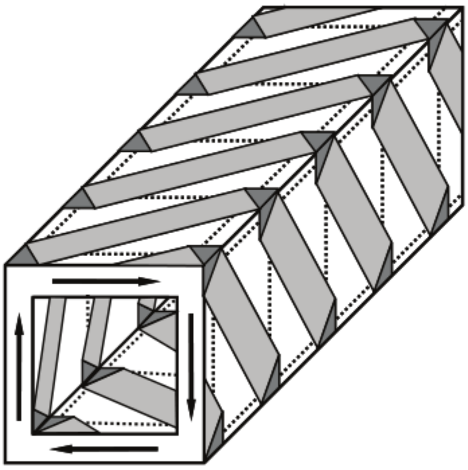

Figure 2.3: Lambert & Thurlimann's tubular truss model(Brown, 2005)

This tubular truss was then further refined to a space truss model by the following references (Lüchinger, 1977), (Ramirez & Breen, 1983) and (Collins & Mitchell, 1986). The refined space truss model could account for bending, shear, torsion and axial load.

Because of the increasing interest in the strut-and-tie modelling, researchers started to publish general methods for the application of the strut-and-tie model for use in discontinuity regions (Marti, 1985a), (Marti, 1985b) and (Schlaich, Schäfer, & Jennewein, 1987). Because of these proposed approaches it became widely accepted and applicable to all kinds of structures. The strut-and-tie method became an effective method to design elements with load discontinuities or geometric changes. The proposed approaches included basic tools that could be applied to complex structures so they could safely design structures using behavioral models. This was seen as the first step towards a unified design method for concrete structures (Williams, Deschenes, & Bayrak, 2012).

Because of this unified design, the strut-and-tie method could be adopted and used in many codes around the world. First of all, it was adapted in the Canadian CSA standard in the year 1984. Later on, it was implemented in the American Association of State Highway and Transportation Officials (AASHTO) in 1989 for the segmental guide specifications and in 1994 for bridge design specifications. In 2002, the American Concrete Institute (ACI) included the strut-and-tie method in the building code requirements for structural concrete. Macgregor in the year 2002 published a special publication (SP-208) with information about the background of provisions included into the ACI code. Nowadays, most countries have the strut-and-tie method incorporated into their design codes for concrete structures (Martin & Sanders, 2007), (Brown, 2005).

2.2 Main principles

In the design of structural concrete there are two limit states that can be considered. The first one is the ultimate limit state (ULS). When designing according to these rules, members are designed for strengths until ultimate failure loads and failing of the structure. Safety factors are then applied to remain conservative. The second one is the serviceability limit state (SLS). This limit state considers serviceability characteristics such as cracks, deflections, deformations etc. Logically, lower maximum strength values are obtained for a same member in this state because these considered phenomena appear before failure.

The STM is a method to design and calculate concrete members in the ultimate limit state. The concrete members are designed to resist a specific ultimate force until failure. Consequently experimental tests conducted to check the STM predictions are applied on the members until failure.

The Strut-and-Tie method is based on the lower bound theorem. The external loads are assumed to be transferred through the concrete mass by internal stresses in the different materials. A model is chosen to represent these stress paths and consequently the internal stress can be calculated in each point of the model. If these stresses, derived from the geometry and external loads, are smaller than the maximum failure loads at each point, then failure will not occur.

2.3 Elements of the STM

To discuss the elements of the strut-and-tie model a combination of following works was used: (Martin & Sanders, 2007), (Brown, 2005), (Williams et al., 2012), (ACI Comittee 318, 2002).

Strut-and-tie modeling is used to design discontinuity regions, also called D-regions, in reinforced concrete structures. The objective of STM is to reduce the level of stress in these D-regions due to the influence of exterior forces. By using STM the complex states of stress within the elements are reduced into a truss existing of simple states of stress. These are uniaxial stress paths. Each of these simple uniaxial stress paths are parts of the STM model. A strut-and-tie model exists of three elements: struts, ties and nodes. The forces in these elements need to be known and can be calculated using the simple truss geometry. Once these forces are known, the resulting stresses in the elements are also known. These can then be compared with the codes specifications if it’s permissible. Because of the uniaxial tension and compression within the element, the appropriate reinforcement (in the form of steel bars, meshes, etc.) is essential.

In the strut-and-tie model you have three major components as mentioned above: struts, ties and nodes, see Figure 2.4.

- Struts: The elements of the STM that represents the compressive stresses.
- Ties: The elements of the STM that represents the tensile stresses.
- Nodes/nodal zones: The elements of the STM where the struts and ties are connected.

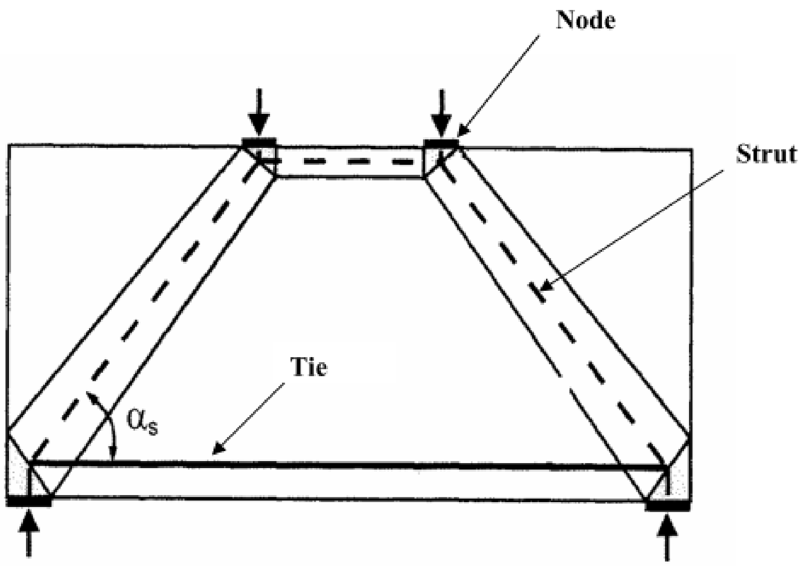

Figure 2.4: Elements of a STM model (Martin & Sanders, 2007)

2.3.1 Struts

Struts are the elements of the STM that represent the compressive stresses in the concrete structure. These struts transfer the forces from the loads on the element to the supports of the element. Struts vary widely in geometry, depending on the specific force path that arises from each single strut. Even if these struts can vary widely, three major geometric shape groups can be found for struts. These are prismatic, bottle shaped and compression fan shaped struts. The strut shapes are illustrated in Figure 2.5 for deep beams.

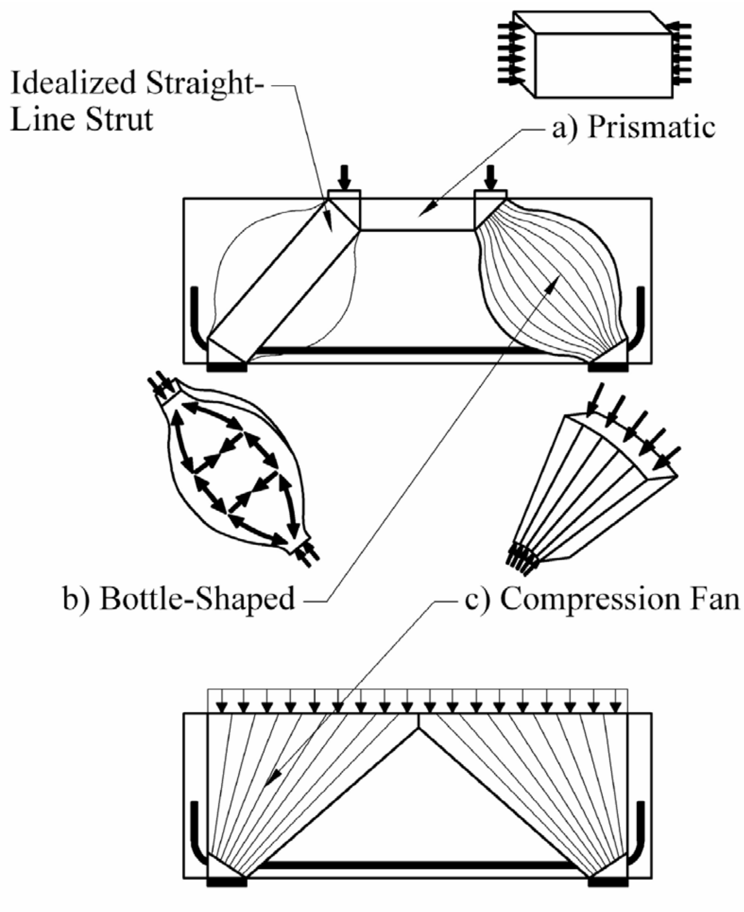

Figure 2.5: Different types of struts (Martin & Sanders, 2007)

Prismatic shaped struts are the most basic ones. They can be found where the loads on the element are uniform, therefore the stresses are uniform. Because of the uniform loads on this specific strut, the cross section of the struts are also uniform. These kind of struts are located in the compression area at the top of the deep beam if there is positive bending.

Bottle shaped struts are formed when the geometrical conditions at the ends of the strut are well known, but in the middle of the strut are not confined to a part of an element. This means that they're located in a part of an element where the middle of the strut can spread out. Forces applied to the ends of these struts lead to compression stresses. As the compression stresses disperse from both ends, they change direction and create an angle. The spreading of the stresses is not desirable because this leads to tension fields at the place of dispersion. For bottle shaped struts, designers should consider ties to represent these tensile forces as shown in Figure 2.6. The bottle shaped struts can be simplified into prismatic shaped struts. Transverse reinforcement is then needed to counter the transverse tension. If the tensile stresses in the bottle shaped struts are too big, the occurrence of cracks in the concrete element is possible. The cracking in the strut has been researched by Sclaich et al. (1987) and Reineck (2002). The research concluded that this type of cracking occurs when compressive stresses exceed 0,55 f_c' at the end of bottle shaped struts.

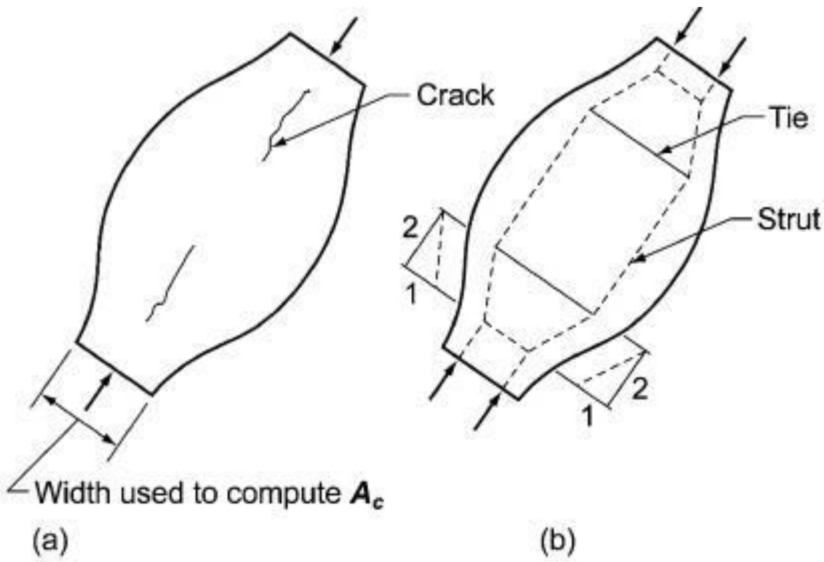

Figure 2.6: Ties in bottle shaped struts(ACI Comittee 318, 2002)

Compression fan shaped struts are formed when stresses from a bigger area flow to a smaller area as shown in Figure 2.5. Stresses are focused on a small area. These kind of struts have zero to none curvature and consequently they don't develop transverse tensile stresses. A simple example of a compression fan strut is a strut that transports a uniform load to a support point.

Struts can fail duo to:

- The cracking/splitting of struts
- The buckling of struts
- Compression failure of the concrete
- Bursting of struts due to transverse tension

2.3.2 Ties

Ties are the elements of the STM that represent the tensile stresses in the concrete structure and represent the equivalent tensile forces. As known, concrete has a small tensile capacity, which is around 10% of the compression capacity. But this tensile capacity of concrete is in most cases neglected because of strength concerns. A tie consists of steel reinforcement rebars and a hypothetical prism of concrete around the reinforcement bar. Because only the steel reinforcement bar contributes to the tensile resistance, it's easier to determine the geometry and capacity of the tie. The capacity of the tie depends on the yield strength of the steel. The tie's geometry will be the same as the steel reinforcement bar. Hereby it's important that the steel reinforcement bar is placed so that the centroid of the reinforcement coincides with the axis of the tie. The area of the steel reinforcement bar A_{st} can be calculated with the following equation: (ACI Comittee 318, 2002)

$$A_{st} = \frac{F_u}{\emptyset f_y}$$

F_u is the force in the tie, f_y is the yield strength of the steel and $\emptyset$ is a reduction factor.

The anchorage of the ties are also important. The anchorage needs to be provided beyond the point that the yield force of the tie is expected, which will be further explained more into detail.

Struts can fail due to:

- Insufficient end anchorage
- Lacking of reinforcement quantity

2.3.3 Nodes

Nodes/nodal zones are the elements of the STM where the struts and ties are connected. The point where the struts, ties and forces of the struts and ties intersect are the nodes. The area of concrete around these nodes are the nodal zones. Three forces always have to act on the node otherwise the equilibrium of vertical and horizontal forces is not in balance. Calculations are made easier by dividing the reaction force into two forces (R => R_1, R_2).

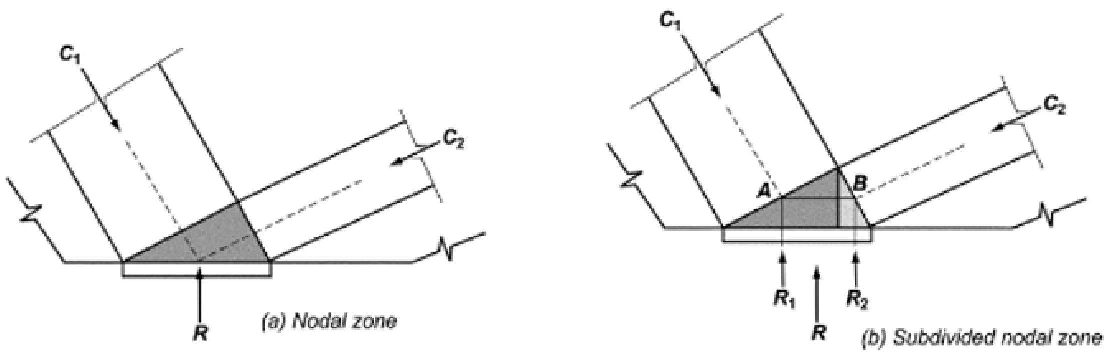

Figure 2.7: Representation of a nodal zone (ACI Comittee 318, 2002)

Nodes are described by the elements, thereby the forces, acting on the node. Three major node types can be found: C-C-C nodes, C-C-T nodes and C-T-T nodes, these can be seen in Figure 2.7. C-C-C nodes are nodes where only struts intersects. C-C-T nodes are nodes where there is only one tie that is intersecting with struts. C-T-T nodes are nodes where there are more than two ties and only one strut intersecting. There is also a fourth option, T-T-T nodes where only ties intersect, however most design specifications don't identify these kind of nodes. The geometry of nodal zones are based on the bearing conditions, the details of anchored reinforcement and the geometry of struts and ties intersecting in the node. As known, concrete has a great compression capacity and therefore the C-C-C nodes have a greater concrete efficiency, bigger strength, of all the types of nodes. The types of nodes are shown in Figure 2.8 below.

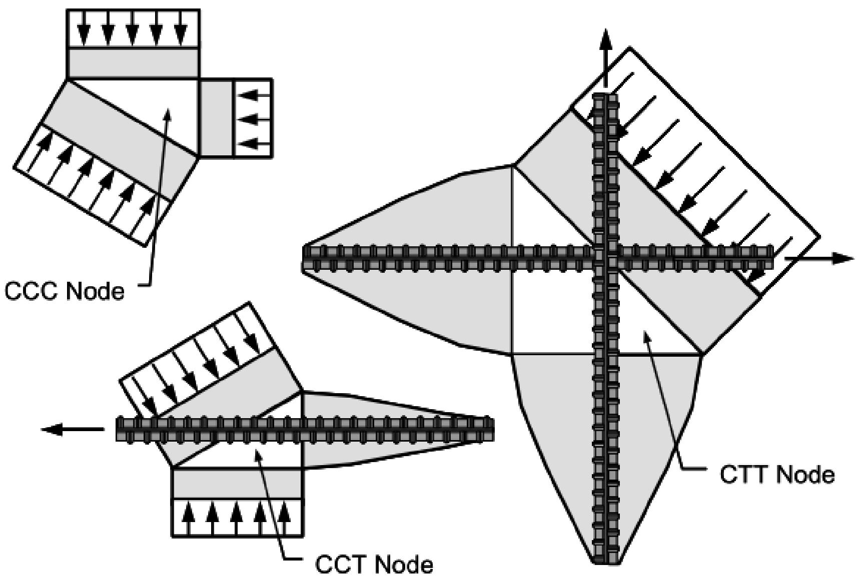

Figure 2.8: Different types of nodes (Brown, 2005)

There are three major types of nodes, but each type of node can be detailed as a hydrostatic node or a non-hydrostatic node, see Figure 2.9. In hydrostatic nodes, the stress on each side of the node is equal and perpendicular to the face of the node. Because of the fact that the stresses are perpendicular to the faces of the nodes, there is no presence of shear stresses on the faces of the nodes. Successfully achieving hydrostatic nodes in STM is almost impossible and most of the time non-viable. Because of the impossibility and impracticality, STM uses non-hydrostatic nodes. When the node is non-hydrostatic, the stresses aren't equal and perpendicular to the faces of the node. Instead of equal stresses, they are proportioned based upon the stresses on the node. Schlaich et al. (1987) stated that, for non-hydrostatic nodes, the ratio of the maximum stress on a side of the node to the minimum stress on a side of the node needed to be lower than two.

The size of a hydrostatic node can be determined using the stress and force on the node. Based on Figure 2.9, the next equation can be utilized to determine the size:

$$w_1 = \frac{F_1}{\sigma_1}$$

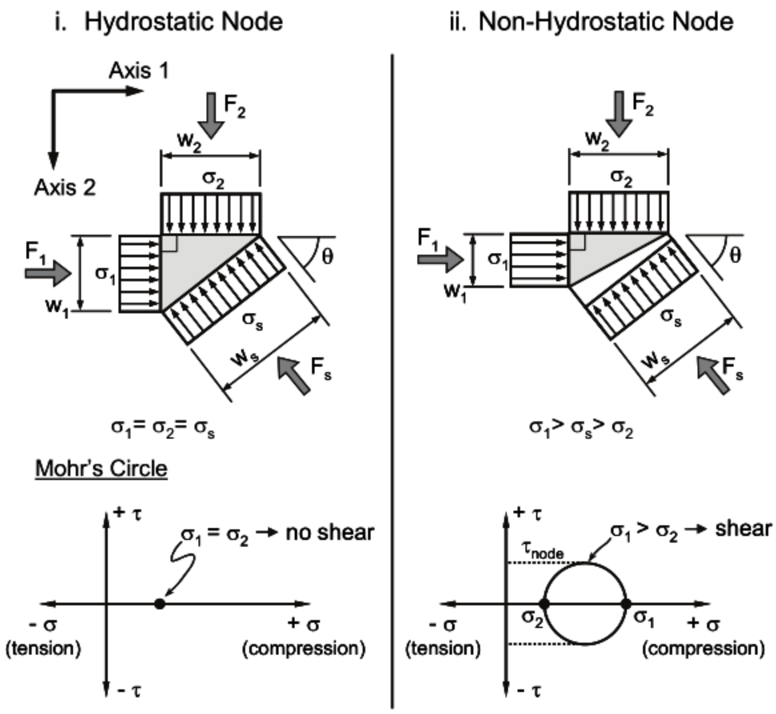

Figure 2.9: Difference between hydrostatic and non-hydrostatic node (ACI Comittee 318, 2002)

As stated above hydrostatic nodes are impractical and impossible to realize, this refers to the impracticality to place steel reinforcement and the unrealistic geometries of the nodes (Williams et al., 2012).

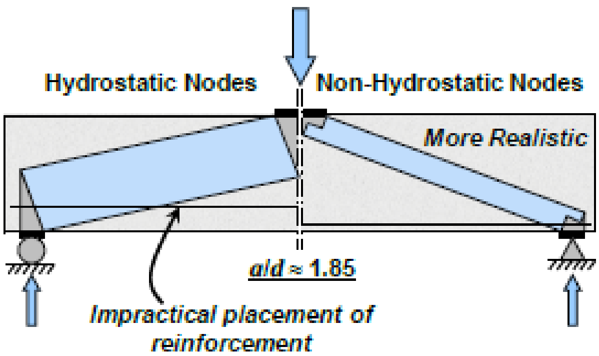

Figure 2.10: Effect of different nodes on strut geometry (Williams et al., 2012)

2.4 Design according to ACI

Nowadays, there are many code provisions that offer guidance in using STM to design and analyze D-regions. The most important European codes are the EC2 (Eurocode 2, 2005) and fib (The International Federation for Structural Concrete, 2013), the most important American codes that describe STM are ACI 318 (ACI Comittee 318, 2014) and AASHTO LRFD (American Association of State Highway and Transportation Officials, 2017). The “Strut-and-Tie Model Design Examples for Bridges: Final Report” (Williams et al., 2012) provides a design flowchart where users of STM can base their design on. We used this flowchart together with the ACI 318-14 to explain the main steps in STM design. These provisions are explained because most papers that we used in this study often refer to or even adapt formulations from this code. STM design specifications have first been adopted into this code in 2002 in the Appendix A for the design of members that have not been explained in the core text. Since then, STM has been given a proper chapter.

For us it was important to first get to know this procedure before jumping into our critical analysis of the most current recommendations that are made in the papers that we discussed. By knowing the STM design procedure, we could better understand the different steps and consequently we were able to locate the problems that are discussed in the papers to the right place of the procedure.

2.4.1 Design flowchart

Different authors propose some flowcharts to visualize the different steps in STM designing. A flowchart that is often used and represents well all the steps is given by Brown et al. (2006).

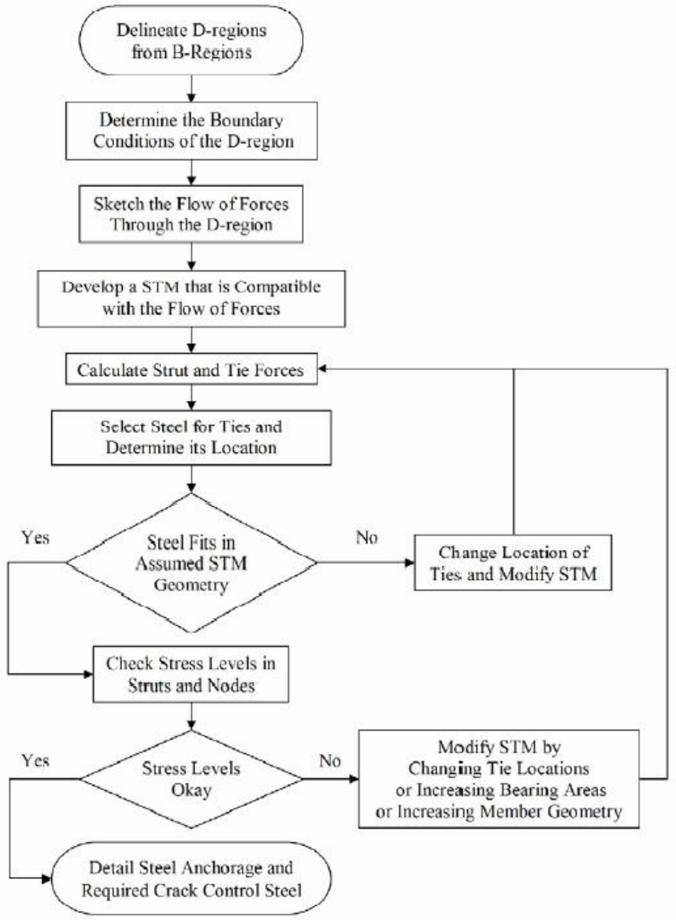

Figure 2.11: Flowchart illustrating STM steps (Brown et al.,2006)

We can summarize this modeling procedure into a smaller amount of steps which are described below.

Step 1: Analyze the structure and the loads

Concrete structural members can be divided into B- and D-regions. The B-regions (Bernoulli regions) are the sections in the concrete member where the beam theory is valid. Assumptions are made that plane sections remain plain after loading (Euler-Bernoulli) and this is valid for those regions. The stresses within a cross-section of the concrete are linear. D-regions however don't show this linear distribution of stresses. It is for these regions that STM is used. To determine B- from D-regions, St. Venant's principle is used. Discontinuity regions occur on those places where there is a change of loads or a change in the geometry of the structure. St. Venants principle explains that the stress due to axial loading and bending becomes a linear distribution again on a small distance away from the discontinuity. The value that is proposed is the depth of the cross-sectional member h, away from both sides of the discontinuity as is illustrated in Figure 2.12.

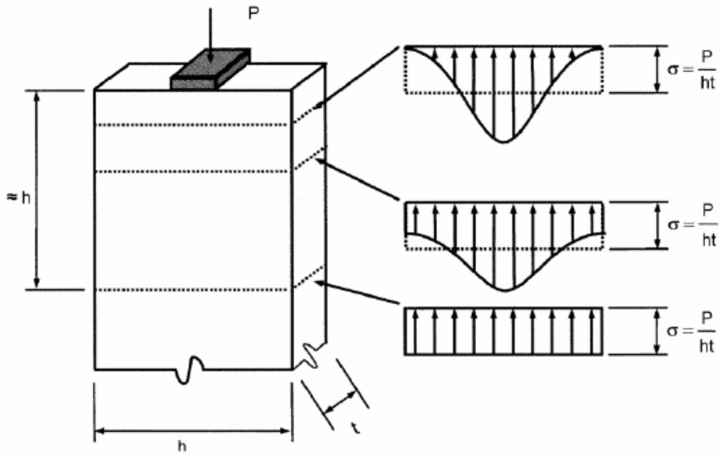

Figure 2.12: St. Venant's principle (Brown et al.,2006)

When applying these principles to the concrete structure, it can be divided into the both zones as in Figure 2.13. When these zones are determined, the boundary conditions should be derived. This means all acting forces on the surface between the B/D-region should be calculated. This can be done easily by using the sectional approach and transferring these internal forces as new loads at the ends of the D-regions.

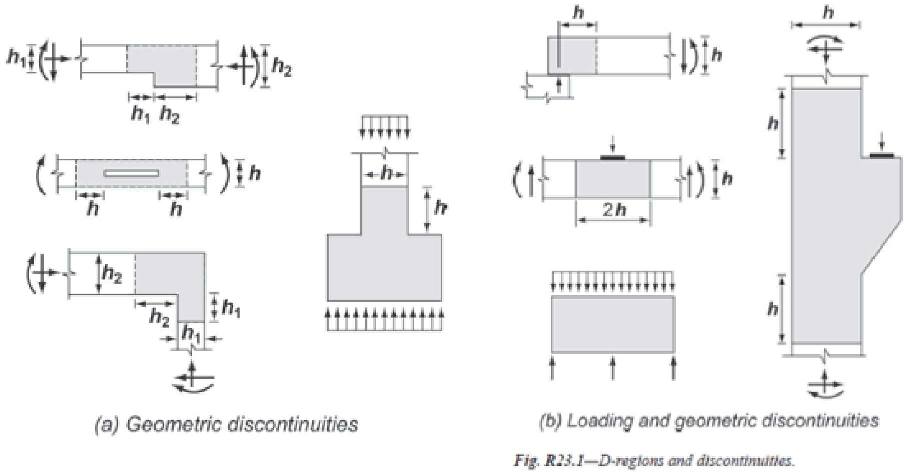

Figure 2.13: D-regions and discontinuities (ACI Comittee 318, 2014)

Step 2: Develop a STM

There is not a single model that can be developed for any structural member. The only thing that has to be reassured is the lower bound theorem. Loads are transferred through the structure to the supporting points by stress in the concrete and the reinforcement. As long as the external forces don't cause exceeding of the maximal stress, then failure will not occur. For two-dimensional members like deep beams, already a lot of experiments have been done and there exist many different models. But the main idea is that struts must represent the compressive load paths as close as possible and ties must be placed where the tensile stresses are located. These stress paths are traditionally found by the use of elastic stress trajectories.

For more three-dimensional members, like pile caps, this becomes more of a difficulty. The visualization of the stress trajectories in these highly disturbed, non-linear D-regions are almost impossible to attain. Another possibility that has been developed is topology optimization techniques. The main idea of this technique is detecting finite elements within the mass of concrete that are 'active', which means those that are applied with stresses. The inactive elements are deleted and consequently a solution for the geometry is derived. This method has some shortcomings as well for practitioners, because FE modeling is needed. For these reasons, researchers are still developing and refining STM models with their own interpretations and ideas.

Little guidance is given on the construction of a STM model in ACI 318-14. A list of some recommendations are listed below:

- The minimum angle between elements is 25°
- Follow the known cracking pattern of the structure being designed if such information is available (MacGregor & Wight, 2005)

- The path that the loads choose is dependent on the length of the path and the deformations that occur. The loads will choose the shortest path and the path with the fewest deformations (MacGregor & Wight, 2005)
- Struts cannot overlap, but ties can cross struts
- Use of a statically determinate model is recommended (MacGregor & Wight, 2005)

Step 3: calculate member forces

When a statically determinate model is used, the forces in the members can be easily calculated regarding the geometry of the model and the external forces that are applied.

Step 4: determine reinforcement in ties and check stress limits

The required amount of reinforcement for the ties can be computed by dividing the force in the tie by the product of the yield stress of the steel. The rebars must then be placed in a way that the centroid of the reinforcement coincides with the location of the tie in the STM. If the geometry of the member doesn't allow this position, then a new location should be chosen which results in a modified STM model and consequently member forces need to be recalculated.

ACI 318-14 provides the following equation to determine the strength of the ties, from which the amount of reinforcement can be calculated:

$$F_{nt} = A_{ts}f_y + A_{tp}(f_{se} + \Delta f_p)$$

Where A_{tp} is zero for non-prestressed members.

When using bottle-shaped struts, there must be a minimum amount of web reinforcement crossing the struts to prevent them from splitting due to transverse tensile stresses. The amount is given by:

$$\sum \frac{A_{si}}{b_s s_i} \sin \alpha_i \geq 0.003$$

Where A_{si} is the required reinforcement, b_s the width of the strut, s_i the spacing of this additional reinforcement and α_i the angel of the corresponding strut, see Figure 2.14.

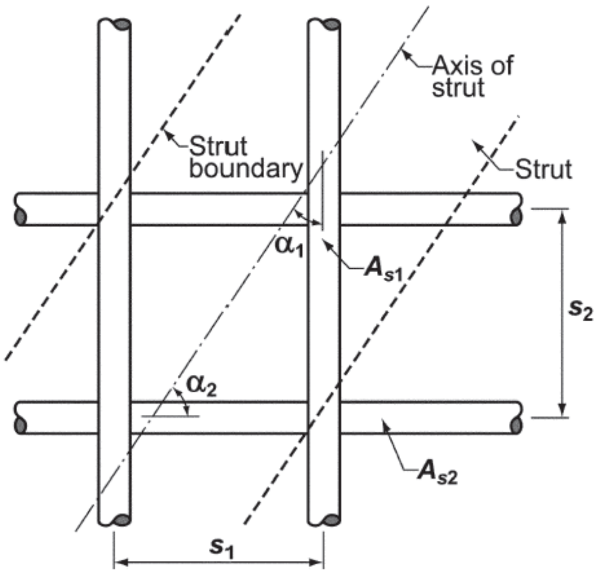

Figure 2.14: Reinforcement crossing a strut (ACI Committee 318, 2014)

The strength of the struts is given by the following equation in ACI 318-14 chapter 23:

$$F_{ns} = f_{ce} A_{cs}$$

$$f_{ce} = 0.85 \beta_s f_c'$$

In this formulation the area A_{cs} for two-dimensional models like deep beams, is calculated by the projection of the bearing area perpendicular to the axis of the strut. We can already remark here that the given formulation is hard to deal with in three-dimensional members.

The efficiency factor β_s depends on the shape of the strut and is given by the following provisions in ACI:

Table 23.4.3—Strut coefficient β_s

Strut geometry and location	Reinforcement crossing a strut	β_s	
Struts with uniform cross-sectional area along length	NA	1.0	(a)
Struts located in a region of a member where the width of the compressed concrete at midlength of the strut can spread laterally (bottle-shaped struts)	Satisfying 23.5	0.75	(b)
	Not Satisfying 23.5	0.60λ	(c)
Struts located in tension members or the tension zones of members	NA	0.40	(d)
All other cases	NA	0.60λ	(e)

Figure 2.15: Strut coefficients (ACI Comitee 318, 2014)

The strength of nodal zones is similarly assessed by the code provision with similar equations:

$$F_{nn} = f_{ce} A_{nz}$$

$$f_{ce} = 0.85 \beta_n f_c'$$

The area A_{nz} which represents the considered area of the nodal zone, is given by the area perpendicular on the axis of the strut that enters the nodal zone. Again we can make the remark that no special specifications are given for three-dimensional nodal zones accept that it should be at least the size of which is explained for two-dimensional nodal zones.

The efficiency factor β_n for calculation of the effective concrete strength of the node is given in the following table from ACI.

Table 23.9.2—Nodal zone coefficient β_n

Configuration of nodal zone	β_n	
Nodal zone bounded by struts, bearing areas, or both	1.0	(a)
Nodal zone anchoring one tie	0.80	(b)
Nodal zone anchoring two or more ties	0.60	(c)

Figure 2.16: Nodal zone coefficients (ACI Comittee 318, 2014)

Step 5: provide anchorage for the ties

ACI318-14 states that the tie reinforcement shall be anchored by mechanical devices, post-tensioning anchorage devices, standard hooks or straight bar development. Because of geometrical limitations, the most used method is with standard hooks. The reinforcement should be anchored before it exits the extended nodal zone as shown in Figure 2.17.

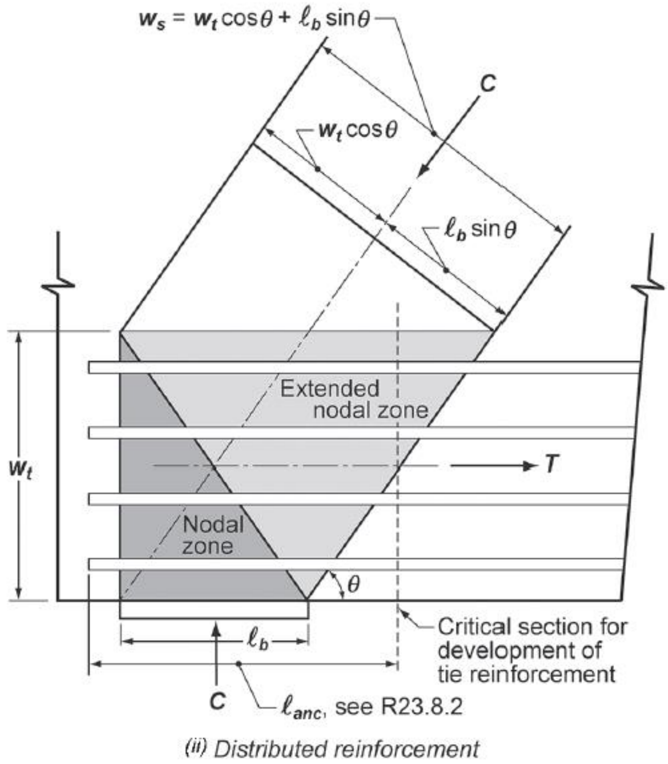

Figure 2.17: Extended nodal zone (ACI Comittee 318, 2014)

When using standard hooks, the development length that is necessary for anchorage of the tension ties is given by the minimum of the following values from section 25.4.3 (ACI318-14):

a) $\left(\frac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f_c'}}\right) d_b$

b) $8d_b$

c) $6in.$

Where d_b is the bar diameter, f_y the yield stress of the reinforcement, λ a factor to account for light or normal weight concrete and ψ are modifications factors which can be found in Table 25.4.3.2 (ACI318-14).

3 Improvements on Pile Caps

Pile caps are a very important structural member because it forms the connection between the upper structure and the substructure. In a lot of situations, it is applied by the load of one column and it carries the forces to several piles. Although the high importance of this structural member, current design codes do not provide accurate specifications for the design and analysis of pile caps. A first method that was used was providing enough depth to account for the shear strength of the pile cap and simultaneous providing enough longitudinal reinforcement based on simple beam theory to provide flexural capacity. However, pile caps are disturbed regions (D-regions) and consequently this beam theory where plane sections are assumed to remain plain can not be used. These sectional approaches can lead to unconservative and inadequate predictions.

The second method to design pile caps is using the strut-and-tie method which captures the non-linearity of the stress distribution. Concrete compressive struts between the piles and the column represent the compressive stress fields in the concrete, while the ties account for the reinforcing steel between the piles. Several researchers have done experimental tests throughout the years to investigate interesting parameters of pile cap configurations to determine the ultimate strength. Based on these tests and conclusions, some STM methods are proposed and tested for effectiveness.

In this part of the thesis, a table is presented as a collection of the most important and most recent proposed methods based on the STM to design and analyse pile caps. The main principles of each research are explained and a critical analysis is performed for each paper.

3.1 Table of improvements

This table below gives a full overlook of the used papers in this study. They are numbered according to the subtitle in which they are explained and discussed by a critical analysis. The titles of each paper are adopted below as headings for our critical analysis.

Table 3.1: List of proposed STM methods for pile caps

Nr.	Year	Authors	Improvement/recommendation	Remarks
1	1996	Adebar & Zhou	Proposed STM method	One of the first
2	2007	Miguel et al.	Proposed stress limit in nodal zones	Only an experimental campaign on three-pile caps
3	2008	Park et al.	Proposed STM approach	Principles explained below
4	2009	Souza et al.	Proposed model for four-pile caps	Model explained below
5	2015	Guo	Proposed spatial strut-and-tie model (SSTM) for evaluating punching-shear	Can only be used for grid reinforcement layout
6	2016	Yun & Ramirez	Proposed method to calculate effective strength of concrete strut in 3D members	Uses finite element
7	2016	Melendez et al.	A finite element analysis tool for 3D concrete members	A tool is developed for practical use of FE
8	2017	Melendez	Refined STM model for four-pile caps	Innovation lies within non-fixed upper nodes
9	2017	Mathern	An enhanced STM model for pile caps, refining nodal zones and their geometry	Needs iteration and FE
10	2018	Miguel et al.	- Secondary reinforcement increases capacity and is used for crack control - A renewed formulation is given for punching shear failure based on EC2	Experimental tests have been conducted on a small dataset
11	2018	Yun et al.	Proposed 3D grid STM for design of concrete D-regions	Uses finite element and considers triaxial stress state of concrete

3.1.1 Design of deep pile caps by strut-and-tie models

In this paper (Adebar & Zhou, 1996), the authors propose a simple and rational design procedure for deep pile caps, in which the best indicator for shear strength is the maximum bearing stress rather than the shear stress. The maximum bearing stress that can be applied without the splitting of the compression struts, as defined by the authors, is dependent on the quantity of confinement and the height to width ratio of the compression struts.

3.1.1.1 Outline of the research

A simple three-dimensional strut-and-tie model for four-pile caps is used in this study, see Figure 3.1. The column load is directly transferred to the support through inclined compression struts and to ensure that the piles aren't spread apart due to the column load, there are horizontal ties that connect the piles. The horizontal ties represent longitudinal reinforcement in the pile cap.

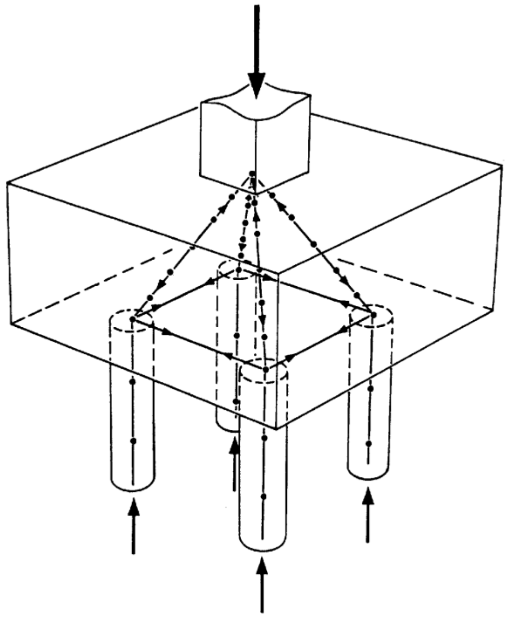

Figure 3.1: STM model for four-pile cap

It isn't evident to put enough horizontal and vertical reinforcement in pile caps to ensure crack control, because of this diagonal cracking of compression struts should be avoided by limiting the concrete tensile stresses. Adebar & Zhou (1993) proposed bearing stress limits of to avoid transverse splitting of the concrete compression struts enclosed by plain concrete. With these stress limits used on deep pile caps, flexural design and shear design of deep pile caps can be analysed with strut-and-tie models.

The shear design of pile caps by utilizing a strut-and-tie model comprises of limiting the concrete stresses in compression struts and nodal zones to ensure that the tension ties yield before there is any diagonal cracking in the compression struts confined by plain concrete. Schlaich et al. (1987) stated that concrete stresses within a D-region are safe if the maximum bearing stresses in nodal zones are less than a certain limit. Therefore the bearing stress limits of Adebar & Zhou (1993) are applied to the concrete stresses in the nodal zones of deep pile caps. These bearing stress limits are:

$$f_b \leq 0.6f'_c + \alpha\beta 32\sqrt{f'_c}\ in\ kips$$

- f_b is the bearing stress
- f_c' is the concrete stress
- α accounts for the confinement of the compression strut
- β account for the geometry of the compression strut

$$\alpha = \frac{1}{3}(\sqrt{A_2/A_1} - 1) \leq 1.0$$

$$\beta = \frac{1}{3}(\frac{h_s}{b_s} - 1) \leq 1.0$$

- A_2/A_1 ratio can be seen in Figure 3.2.
- h_s/b_s is the height to width ratio of the compression strut.

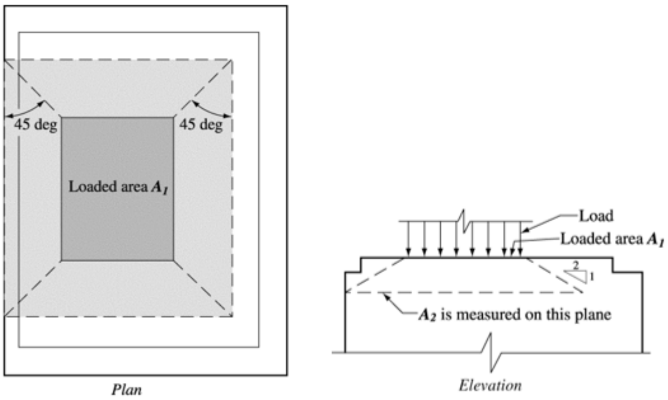

Figure 3.2: Aspect ratio

To calculate the maximum bearing stress beneath a column, where more than one strut meet, h_s/b_s can be obtained by using following equation.

$$\frac{h_s}{b_s} = \frac{2d}{c}$$

Where *d* is the effective depth and *c* is the dimension of a square column (in case of a round column, the diameter of the column can be used for *c*). To calculate the maximum bearing stress above a pile, where only one strut ends, h_s/b_s can be obtained by using following equation.

$$\frac{h_s}{b_s} = \frac{d}{d_p}$$

Where d_p is the diameter of the pile.

To demonstrate the use of STM predictions over sectional approaches for shear and flexural design, the authors performed a comparative study with predictions from these codes.

It is also noted that bunched reinforcement gives a substantially higher flexural capacity than when uniform grid reinforcement is used. However the uniform grid reinforcement helps to control the cracking.

3.1.1.2 Discussion

This is one of the first studies to introduce a rather simple but rational strut-and-tie model to design deep pile caps. In the year 1996 there were almost no provisions to calculate pile caps with correct and accurate results at that time. Their design was based on sectional approaches. Therefore, it was an improvement to introduce the strut-and-tie model to design deep pile caps. Because it was one of the first strut-and-tie models to design pile caps, a lot of parameters weren't taken into consideration, such as shear span-depth ratio for example. The identification and introduction of these other parameters were only later and more recently identified and researched. Nowadays strut-and-tie models are far more correct and accurate but they are still not perfect because pile caps are a rather difficult concrete element to design and evaluate.

From the test results, the authors could see that a lot of tests were reported to fail in shear, while being designed for flexural failure by the sectional approaches. This indicates that a better definition for the shear strength was needed, which has been given by the author. The comparison between the proposed strut-and-tie model and the sectional approaches shows that there is less scatter on the results by the proposed method. This indicates an improvement, but the still high strength prediction ratio of 1.55 also indicates that the model isn't perfect.

Though this model was not perfect, the strut-and-tie model was an easy method to design deep pile caps. The parameters of the proposed model are easy to become and the equations aren't difficult to solve. Because of this, the proposed strut-and-tie model could be easily implemented in future code provisions to design deep pile caps at that time. It was only in the year 2002 that a strut-and-tie model, not their model, for pile caps was introduced in the ACI code. We consider this model as an important step to enhance the interest and to prove the importance towards three-dimensional strut-and-tie modelling.

3.1.2 Structural behaviour of three-pile caps subjected to axial compressive loading

In this paper (Miguel, Takeya, & Giongo, 2007), a comparative study can be found regarding the structural behaviour of three-pile caps that are subjected to axial compressive loading. The main reinforcement was the same throughout all the specimens, namely bunched reinforcement that connects the piles. The cracks and the modes of rupture were mainly observed in this comparative study. The load, when there is rupture failure, never exceeded 1.12 times the calculated load. The rupture failure was either caused by cracking of the compression strut or yielding of the reinforcement.

3.1.2.1 Outline of the research

The authors tested four kinds of specimens, they all had the same main reinforcement connecting the piles, but their secondary reinforcement was different. Specimens A1 (a) only had the main reinforcement, specimens A2 (b) had extra secondary reinforcement going through the projection of the column and the centre of the piles, specimens A3 (c) had grid reinforcement added as secondary reinforcement and specimens A4 (d) had secondary reinforcement containing horizontal and vertical stirrups. The different kinds of reinforcement layouts can be seen in Figure 3.3.

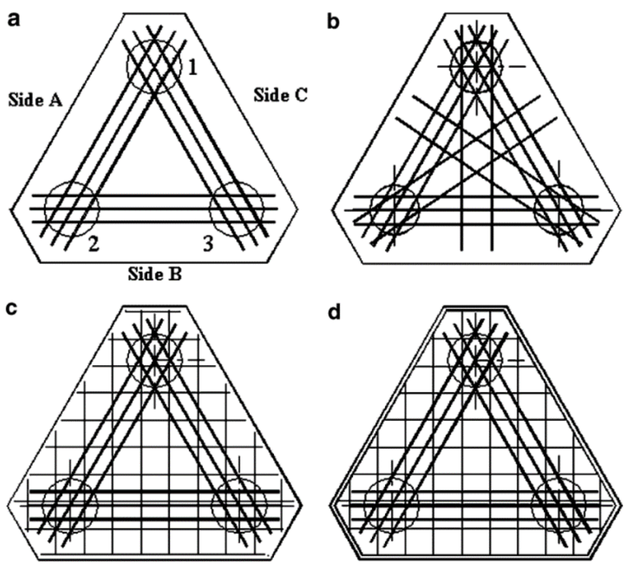

Figure 3.3: Reinforcement layouts

As mentioned in the introduction of this paper, the main goal was to investigate experimental behaviour of three-pile caps. The authors did not propose improvements on a strut-and-tie method themselves. They calculated the theoretical loads using the strut method of Blévot & Frémy (1967).

The experimental loads were always greater than the theoretical values, with a minimum margin of 12% and therefore rather conservative. When reducing the pile diameter, only the A3 specimens presented the same ultimate loads. Pile caps using piles with a diameter of 20 cm and being A4 specimens produced the highest ultimate load, while pile caps using piles with a diameter of 30 cm and being A2 specimens produced the highest ultimate load.

The average value of stresses in the lower nodal zones vs the concrete strength (σ_{lnz}/f_{cm}) of the specimens were below the recommended value, therefore the specimens didn't break down because of compression stress in the lower nodal zones. This is the same case for stresses in the upper nodal zones.

After consulting their data on the failure mode, they came to the conclusion that the specimen failed due to cracking of the concrete compression struts and this was quickly followed by yielding of the reinforcement bars. To prevent rupture due to cracking of the concrete compression struts, the authors proposed following stress limits for nodal zones.

$$\sigma_{unz} \leq 0.40 f_{cm}\ (Upper\ nodal\ zone)$$

$$\sigma_{lnz} \leq 0.50 f_{cm}\ (Lower\ nodal\ zone\ with\ pile\ diameter = 20cm)$$

$$\sigma_{lnz} \leq 0.30 f_{cm}\ (Lower\ nodal\ zone\ with\ pile\ diameter = 30cm)$$

- f_{cm} is the average concrete compression strength
- σ_{lnz} is the compression stress in the lower nodal zone
- σ_{unz} is the compression stress in the upper nodal zone

3.1.2.2 Discussion

Considering that the strut-and-tie analysis is quite acceptable and adaptable to the three-pile caps study, the use of a quite initial model as the one published by Blévot & Frémy (1967) in 1967 seems to be outdated at the time of the research. There were some other recently adapted studies such as Adebar et al. (1990) and Adebar & Zhou (1996). This research was done in the year 2007, so if they used a more recent model, they could have gotten better results with a lower effectiveness ratio for the strength predictions compared to the experimental failure loads. Now they were ranging from 1.12 to 2.64, these results are really scattered and in most cases the pile caps strength is really underestimated.

Regarding the limits they proposed, they couldn't be found in either the ACI code or EC2. The limits were determined by analysing their experimental data and were far less conservative then those proposed by Blévot & Frémy (1967) and as Adebar et al. (1990). The proposed limits were easily determined because there was only one parameter involved in the calculation, namely the average concrete compression strength.

On the other hand we found it difficult to determine the compression stresses in the lower and upper nodal zones to meet the limits. Because these compression stresses were measured by having strain gages attached to the specimen when performing the tests. In case it could be adopted by a code provision there should be a direct equation to calculate these stresses. The proposed stress limits in the nodal zones are specifically for the tested database of experiments. However, more important in their research was to test the experimental behaviour of the specimen and compare it to STM predictions. The results showed indeed that the mode of failure was by cracking of the concrete struts which was predicted by STM as well. Out of this research, the use of STM as a valuable tool for determining strength of pile caps was again illustrated.

3.1.3 Strength predictions of pile caps by a strut-and-tie model approach

The strut-and-tie model that is presented by these authors (Park, Kuchma, & Souza, 2008) is for calculating the strength of reinforced concrete pile caps. The main principles of the approach consider constitutive laws for cracked concrete and both equilibrium of forces and strain compatibility. To test the effectiveness, a methodology for evaluating the capacity of pile caps was developed considering these principles for evaluating the strength of struts.

3.1.3.1 Outline of the research

The geometry of the STM model isn't described into detail and can be seen in Figure 3.4 below. Four upper nodes are connected with diagonal struts to the four lower nodes, which are connected with ties that represent the steel reinforcement. The top nodes are connected at the base of the column with concrete struts. The upper nodes of the model are assumed to be located at the column quarter points at half depth of the compressive stress block.

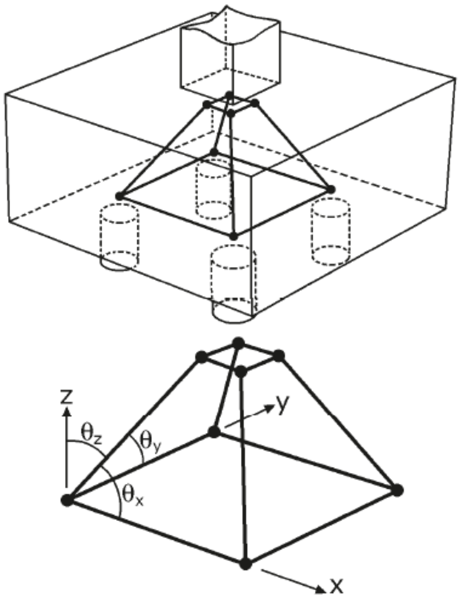

Figure 3.4: STM model for four-pile cap

The proposed method uses the following list of principles:

- Equilibrium: The model is statically determinate and this way the member forces can be calculated easily using the geometry of the truss.
- Compression softening: One of the most important implementations in this model is the effect called compression softening. High tensile strains perpendicular to the compression in cracked concrete cause a reduction of strength and stiffness of the concrete. In this approach the compression softening model proposed by Hsu and Zhang (1997) is adopted.
- Tension stiffening: The tie strength is augmented using tension stiffening of the steel ties. An additional tensile force in the tie is permitted because of defining a concrete tie which can take a part of the tensile strains, reducing the tensile strain in the steel tie. The formula that is used for describing the tensile force in the concrete is adopted from Vecchio and Collins (1986).
- Compatibility relations**:** The sum of normal strain in two perpendicular directions is used. As the horizontal and vertical reinforcements were not available, they calculated conservatively using 0.002 for these strains.

Proposed method:

To evaluate the capacity of the pile caps, a procedure was developed using all the above mentioned principles. More detailed formulae can be found in the paper itself. It isn't the purpose of this master thesis to go more in depth on all the formulations described. Further, we will go deeper in some of the main equations that are still understandable for both the authors and the readers of this thesis.

Three modes of failure are described:

(1) Failure of the diagonal concrete strut (by crushing or splitting, shear failure)

$$P_n = 4\zeta f_c' A_d \cos\theta_z$$

(2) Failure of horizontal concrete strut (horizontal compression zone at base of the column)

$$P_n = 0.85 f_c' \frac{hc}{2} \frac{\cos\theta_z}{\cos\theta_x}$$

(3) Failure of the tensile ties (by yielding of the reinforcement or cracking of the concrete tie)

$$P_n = (2 f_y A_s + 4 F_{ct}) \frac{\cos\theta_z}{\cos\theta_x}$$

Explanation of the used parameters in these equations can be found in the paper.

The predicted strength by the proposed method is eventually the minimum of these three modes of failures.

3.1.3.2 Discussion

This model seems quite solid because it takes some critical principles of cracked concrete into consideration. The first one is the compression softening factor ζ which they adopted from Hsu and Zhang. The authors have proven in a previous work that this factor is conservative and therefore we conclude that there is no problem in adopting this factor. Moreover taking compression softening in consideration is a must according to us, because it is a well-demonstrated phenomena in experimental studies and it has been adopted in many other models.

Considering the strength of the ties, the authors don't follow the provisions from ACI318. In their equation to check the failure of tensile ties they allow the concrete to enhance the tie strength with its tensile stress properties. The tensile capacity of the concrete is neglected in traditional STM's for simplicity considerations. The formulation for concrete tensile strength is adopted from Vecchio and Collins and we see no problems with this adoption. However, we don't feel immediately confident with this tension stiffening effect because of the possible unconservative outcome this could cause. The tensile strength of the concrete is highly dependent on the execution of the concrete mix.

Overall the test results showed that the proposed model led to the most accurate predictions with the lowest scatter among results (Figure 3.5) compared to current code provisions. What is really clear is that STM-based predictions and design methods are recommended over sectional approaches for the design of pile caps. The model in this paper is already a little outdated but it was seen as a good introduction to three-dimensional modelling of pile caps.

		P_{test}/P_n					
Specimen	P_{test} (kN)	(a)	(b)	(c)	(d)	(e)	(f)
SS6	280	1.71	1.71	2.34	1.76	1.16	1.52
SG1	50	—	—	—	—	—	1.53
SG2	173	1.43	1.43	3.11	2.49	1.2	1.97
SG3	177	1.46	1.46	3.2	2.55	1.23	2.01
Average	—	**1.97**	**1.96**	**1.73**	**1.74**	**1.44**	**1.41**
Coefficient of variation	—	**0.17**	**0.17**	**0.24**	**0.2**	**0.18**	**0.18**

Note: (a), Special provisions for slabs and footings of ACI318–99 (ACI Committtee 318 1999); (b), *CRSI design handbook* 2002 (CRSI 2002); (c), strut-and-tie model of ACI318–05 (ACI Committtee 318 2005); (d), strut-and-tie model of CSA A23.3 (CSA 2004); (e), strut-and-tie model approach of Adebar and Zhou (1996); (f), proposed strut-and-tie model approach.

Figure 3.5: Strength prediction ratios

3.1.4 Adaptable strut-and-tie model for design and verification of four-pile caps

In this paper (R. Souza, Kuchma, Park, & Bittencourt, 2009), the authors present a new approach for developing a three-dimensional strut-and-tie model. The model is calibrated on a large set of experimental tests gathered from older researchers. The aim of the research is to better predict the mode of failure and thereby contributing to the development of guidelines to design these important three-dimensional configurations. The model has shown to be accurate on the prediction of failure modes, yielding, cracking and failure loads on this large database.

3.1.4.1 Outline of the research

The authors used the proposed model of Souza et al. (2007) but they calibrated it to a simplified version by setting $e_{x,k}$=$e_{y,k}$=$M_{x,k}$=$M_{y,k}$=0. This means there is no eccentricity and no moments in the four-pile cap, meaning that the axial load is the only applied force. The simple geometry of the proposed model can be seen in Figure 3.6.

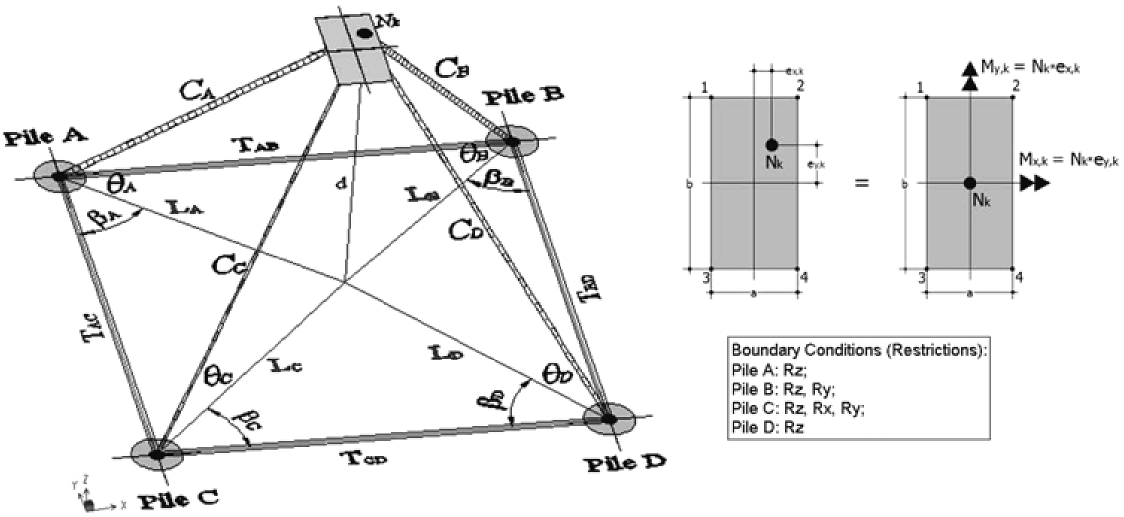

Figure 3.6: Proposed STM model for four-pile cap

Using the simplified approach, the reactions on the piles, the forces in the struts and ties and the internal angles can be easily calculated because the model is statically determinate.

The author only considers one single criterium to evaluate the strength of the pile caps by considering only two modes of failure:

(1) Failure of the tensile ties (by yielding of the reinforcement, flexural):

$$N_{y,a} = \frac{-4\phi_y A_{sD} f_y d}{e}$$

(2) Failure of the concrete struts (by splitting, shear):

$$N_{fs,a} = -4f_t(b+b)d = -2.08bdf_c^{\frac{2}{3}}, f_c \; in \; MPa$$

The first mode of failure is easily derived from the earlier calculated forces in the ties, where A_{sD} represents the steel area in the section, f_y the yield strength, *d* the depth and *e* the pile spacing.

The shear failure mode is adopted from Siao (1993) which stated that splitting of the concrete struts depends on the dimensions of the columns and piles (*b* and *d*) for four pile caps as well as on the tensile strength of the concrete f_t. They adopted the simple formulation for the tensile strength from the CEB-FIP Model Code (1993) which depends on the concrete compressive strength f_c.

The ultimate strength of the four pile caps is determined by the minimum of these two failure modes.

The authors eventually compared their predictions to a large dataset of experimental results on four-pile caps. Another remarkable implementation into their proposed model is the fact that they introduced calibration factors Φ to provide the lowest possible coefficient of variation among the predictions.

3.1.4.2 Discussion

The proposed model of the authors was developed to be able to design and evaluate a big and extensive array of pile cap dimensions, reinforcement conditions and span-depth ratios. Therefore the authors expected it to be generally applicable. The proposed model by the authors is rather good to design four-pile caps as shown by their experimental data comparison, Figure 3.7. Of course there are specimens that failed before the design load was reached but if safety factors were applied, as done in the ultimate limit state, then it could be that all specimens are above an average of 1.00 for the effectiveness ratio predictions.

The authors also did well to implement the dimensions of the column and the tensile strength of the concrete into the formula to predict failure modes and loads. Because most of the times the tensile strength of concrete is neglected but as proven in this paper, for stocky pile caps ($a/d < 0.6$), it has a benefit to the load capacity of pile caps and to reduce the possibility of shear failure. As for the dimensions of the column, which are really easy to determine, the implementation in the formula to predict failure modes and loads is very well chosen. This is because as the authors said, the shear failure is dependent on the pile/column dimensions as well as the tensile strength of concrete.

For considering the proposed method into design codes of practice, which was the main goal of the authors, we have some remarks. First of all, all parameters can be understood and calculated easily. All factors make sense based on both theoretical and experimental background on four-pile caps. This is a first improvement compared to rules of thumb or other sectional approaches that were considered at that time to design pile caps. Another advantage is the consideration of shear failure which isn't adopted in traditional STM models. As experimental results show the brittle failure of the specimens, it is proven that a shear check should definitely be included in designing with STM.

On the other hand the authors proposed method has used calibration factors for the strength predictions. By doing this, we should be careful when analysing the effectiveness ratio of the predictions. We can't be blind for the fact that the rather good predictions are due to this fact. When leaving out the calibration factors, it is not proven that the proposed method is still effective.

We can conclude with some extra remarks on the shear failure mode. The results of this paper provide important insights into shear failure of the four-pile caps and the authors recommended, in order to prevent shear failure, that the compressive stress is lower than $1.0f_c$ and the shear span-depth ratio is lower than 1.0. These recommendations lead to ductile failure, which is safer than shear failure. Ductile failure means yielding of the reinforcement, the yielding means that the steel has an extensive plastic deformation but this deformation is "stable" if the force isn't increased. Shear failure is a brittle failure, which means that the concrete cracks and keeps cracking even if the force isn't increased. By setting these limits, the authors makes sure that the safer fail mechanism takes place.

Table 7—Precision of proposed analytical model based on experimental results[14-19]

	Experimental loads/analytical predictions		
	Cracking	Yielding	Failure
Average	1.00	1.01	1.01
Coefficient of variation	0.14	0.15	0.23
Variance	0.02	0.02	0.05
Standard deviation	0.14	0.15	0.23

Figure 3.7: Strength prediction ratios

3.1.5 Evaluation of column load for generally uniform grid-reinforced pile cap failing in punching

This paper (Guo, 2015) addresses the problem that previous methods evaluate the punching shear failure of pile caps on an empirically way. The existing STM models that are conservative of nature and with a difficult configuration hinder a rational solution. Therefore the author proposes a new spatial STM approach to evaluate the punching shear capacity of general pile caps with a uniform grid-reinforcement layout which he calls TPM. The aim is on proposing strut strength derivation which is more related to the three-dimensional behavior of pile caps from which the punching failure can be calculated.

3.1.5.1 Outline of the research

The location of the nodes in the truss is well described by the author. There is one upper node in the SSTM located at 0.1 times the effective depth downwards from the column center beginning from the top of the surface. The lower nodes are located on the level of the reinforcement centroid just outside the center of the piles. This is explained more in detail as can bee seen in Figure 3.8.

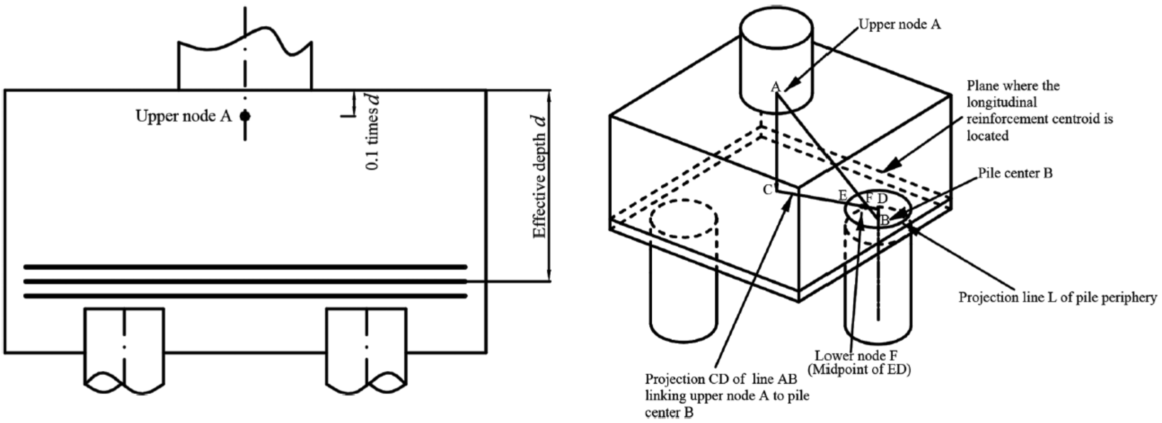

Figure 3.8: Proposed SSTM

According to the authors there are two main factors that influence the strut strength, these are the punching-span ratio λ and concrete strength f_c'.

The cross-sectional area at the strut end (*S*) is assumed to be 0.6 times the diameter of that pile where the strut ends. The effective range of the tension tie ($2D_p$) is twice the pile diameter that is concentric with the lower node of the spatial strut-and-tie model and the punching-span ω is the distance between the middle of a pile and the outer edge of the column supported by the pile cap.

Knowing these parameters, the punching-span to depth ratio λ is given below:

$$\lambda = \omega/d$$

The reinforcement ratio of the tension tie ρ is as follows.

$$\rho = \frac{A_s}{2D_p d}$$

Where A_s is the sum of the cross-sectional areas of the longitudinal reinforcements within the effective range of the tension tie, D_p is the pile diameter and d is the effective depth.

The authors stated that the evaluation of the punching shear resistance is actually the evaluation of the strut bearing load because strut failure is an sign of loss of punching shear resistance. The strut bearing load F is the cross-sectional area at the strut end (*S*) multiplied by the strut strength f_{ce}. The equation is as follows.

$$F = S \times f_{ce} = 0.6\pi R^2 \times \frac{f_{ce1} + f_{ce2}}{2}$$

R is the diameter of the pile. And f_{ce1} and f_{ce2} are the strength at the ends of the strut.

The derivation for an expression of f_{ce} can be found in the paper. The authors used a nonlinear finite element program called ADINA, which has been proven to be effective for determining the effective concrete strut strength in pier decks. The two factors influencing the strut strength are the punching-span ratio and concrete strength. With the use of these two parameters and $\gamma = f_{ce}/f_c'$ following expression could be derived with the aid of the least-square method in the software.

$$f_{ce} = \gamma \times f_c' = \alpha(f_c')\beta(\lambda)f_c'$$

This equation can be then substituted into the equation of strut bearing load *F* resulting in the punching shear strength prediction. Different values for α and β are explained more into detail in the paper. It's not the purpose to copy them herein.

This proposed model for shear resistance of pile caps was than compared with punching shear predictions from other methods such as sectional methods from ACI.

From this research, the authors had some main conclusions:

- The smaller the punching span is, the larger the column load will be.
- The reinforcement ratio of the tension tie ρ (the longitudinal reinforcement) of pile caps with uniform grid reinforcement has very little effect on the punching shear resistance of the pile cap.
- This method is not limited to a certain number of piles or a certain pile arrangement. Therefore it's widely applicable.

3.1.5.2 Discussion

This proposed model for the evaluation of the punching shear resistance of pile caps with uniform grid-reinforcement is really promising because it's more accurate than the already existing methods proposed in codes. It's also widely applicable because it's not limited to a certain amount of piles or a certain pile arrangement what a really good characteristic is for the proposed method for evaluating the punching shear resistance. As mentioned in the discussion of another paper, there are always going to be specimens where the experimental load is going to be lower than the calculated load, but with the use of safety factors in the codes, this problem could be solved.

The method to calculate the strut bearing load F is rather easy because the parameters are well known. Most of the parameters are easily taken from design drawings and the factors $\alpha(f_c^{'})$ and $\beta(\lambda)$ can be easily taken from their research. There are no difficult calculations, analysis or studies needed to determine the strut bearing load F. Because of this, this method to calculate the punching shear resistance of pile caps with uniform grid-reinforcement can be adopted into codes of provisions with ease.

Pile caps reinforcing patterns are repeatedly designed according to a grid at the bottom of the cap. This is because it is straight referred to the sectional calculation system, and the models to predict the bunching effect are not straightforward. Therefore a lot of people could use this model when designing pile caps, which is an advantage of the proposed method.

3.1.6 Strength of concrete struts in three-dimensional strut-tie models

The emphasis of this research (Y. M. Yun & Ramirez, 2016) is on giving a better definition for the effective strength of concrete struts. A consistent and general method is proposed for three-dimensional strut-and-tie models. The newly developed method considers the factors that influence the 3D behaviour of the concrete in the pile cap.

3.1.6.1 Outline of the research

The proposed method of the author to determine the strength of concrete struts in three-dimensional strut-tie models considers the influence of the triaxial state of the stress and strain conditions at the location of the strut, the length of a strut, deviation angles between the strut's longitudinal axis and compressive stress trajectories, concrete compressive strength and steel confinement.

The steps that need to be followed to determine the effective strength of a concrete strut can be seen in Figure 3.9 and is fully explained in the paper itself.

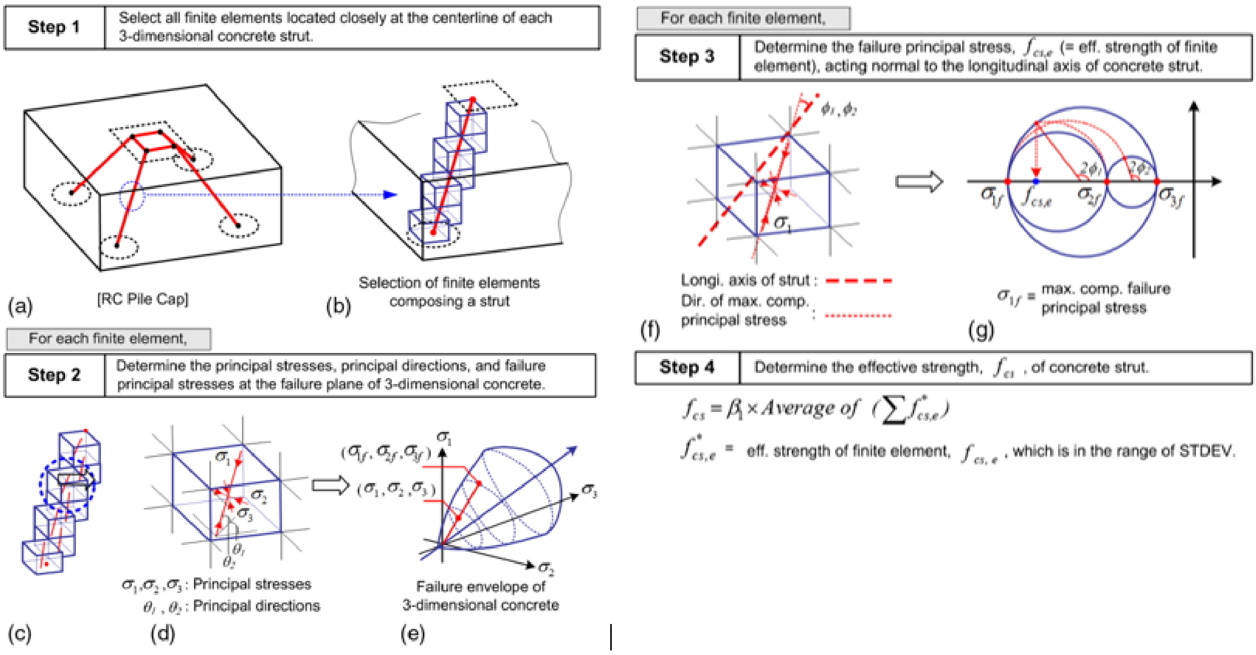

Figure 3.9: Proposed steps to determine effective strut strength

The different steps in deriving the effective strut strength embody the main principles that are mentioned earlier and are explained below:

<u>Equilibrium</u>: In step 1, the practitioner selects all the finite elements that are close to the chosen STM model. The chosen model in this study is statically indeterminate. For this reason the forces in the struts and ties have to be calculated using an iterative approach considering the maximum available and required struts and ties areas and the effective compressive strength.

<u>Deviation angle between strut orientation and compressive principal stress flow</u>: The direction of the chosen STM model are probably not parallel with the principle stress flow which is derived in step 2. Therefore an adaption of the local orientation of the stresses in the finite elements is performed to take this into consideration.

<u>Longitudinal length of a strut</u>**:** In step 3 the effective compressive strength of each finite element is calculated based on the failure stresses. By taking the average of the effective strengths of the finite elements over the full length of the strut in step 4, the authors take the length in consideration.

<u>Tensile strains of reinforcing bars crossing a strut</u>: In step 1 to 4, a unreinforced concrete model is calculated. To implement the effect of tensile stresses by ties crossing the struts, these forces and external forces are again put into the model in step 1. This iterative procedure is described in step 5.

Regarding step 4, it's also possible to only take the average of the effective strengths along the longitudinal axis of the strut if the variation is negligible instead of the average of all the effective strengths of the finite elements. If the strut is rather long, it's better to take a strut with a variable cross-section, such as a bottle-shaped strut, instead of a prismatic-shaped strut.

Step 5 is to analyse the strut-tie model using the calculated effective strengths of the concrete struts. Afterwards you can apply the cross-sectional forces of the steel ties and external forces to the finite element model and then you can iterate the process from step 1. In the end, the finite-element analysis of the 3D structure can be used to obtain the effective strengths of the concrete struts when there is reinforcement in place. The effective strengths of the concrete struts are going to change because of the added reinforcement. Therefore the aforementioned steps need to iterated until a tolerance limit is reached between the newly determined and previously determined cross-sectional forces of the steel ties. There isn't a certain number of iterations that need to be done but most of the time two or three iterations are fine.

Kim et al. (2013) has reported that the statically determinate model (no concrete ties) overestimates greatly the failure strength of pile caps. Therefore the author chooses explicitly and well-thought for the statistically indeterminate 3D strut-and-tie model with concrete ties in the bottom of the model. By having concrete ties in the bottom, they consider that the concrete ties in tension regions have a load-carrying capacity. In Figure 3.10, the exact location of the nodes and the selected indeterminate 3D strut-and-tie model can be seen.

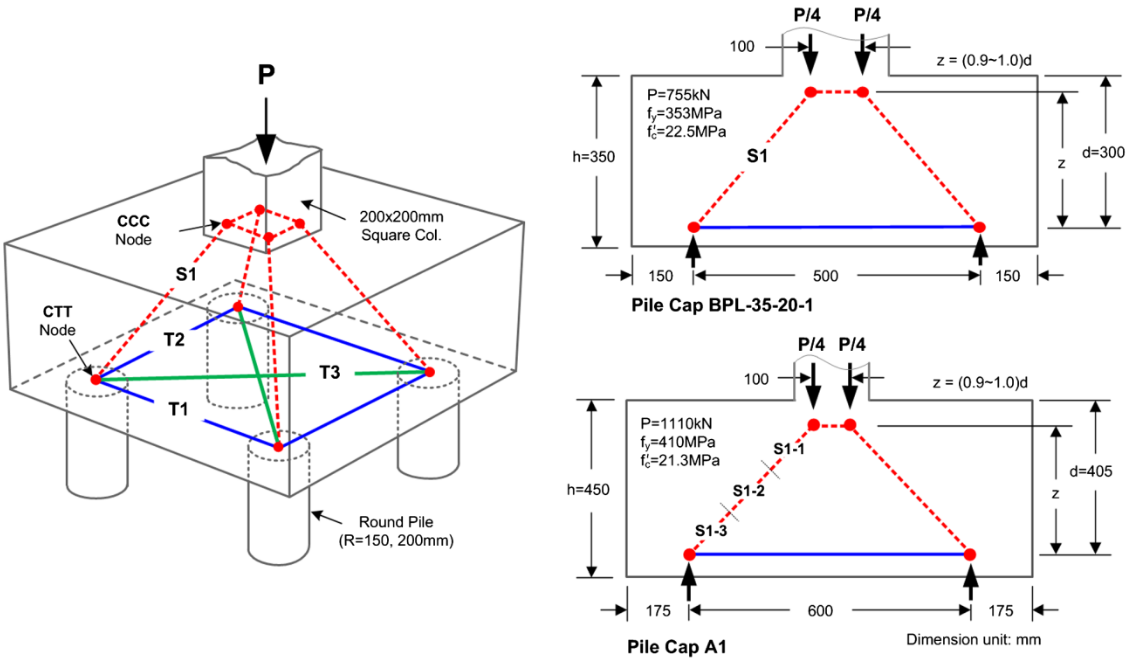

Figure 3.10: Indeterminate STM model for four-pile caps

The load-carrying capacity of the strut-and-tie model was determined by taking the maximum cross-sectional area and comparing that with the required cross-sectional area. These cross-sectional areas were determined by using a simple iterative method, this method required that the calculated stresses of strut and ties were the same as the effective strengths of the struts and ties. The areas of nodal zone boundaries and concrete struts could be obtained by considering the load size and bearing plates and the strut-and-tie model's geometry.

3.1.6.2 Discussion

First of all, the authors stated that the proposed method can be used to assess and design all 3D structural concrete elements with D-regions. This seems to be a bit of a daring statement, considering the wide range of options we can find in several structures, so we should be aware of the meaning of it. This is because the authors only tested their proposed model on pile caps with four piles. We do not think it's advisable to say this if you haven't tested it yet or applied the proposed model to specimens with another amount of piles or other 3D structural concrete elements such as a corbel. They should have just stated that it's applicable to four-pile caps until further testing or analysis.

Despite this remark, we think that the proposed model does actually predict the ultimate strength of the four-pile caps rather well and accurate. When considering adoption into codes of practice, it's not really possible to implement it. However, the ideas and principles can be adopted into more simple three-dimensional models to predict the ultimate strength.

As stated above the proposed model includes the influence of parameters that influence the 3D strut stress behaviour, this is a good decision of the authors because it makes the proposed model more correct and accurate. This effect of the inclusion of these parameters is obviously shown in comparison between different approaches regarding the ultimate strength of the four-pile caps. As seen in Figure 3.11, the proposed method, using the effective strengths of the concrete struts, was better and more accurate than the other methods they compared it with. They also compared the use of a prim-shaped strut and bottle-shaped strut, from this comparison they could conclude that the use of a bottle-shaped strut in the 3D strut-and-tie model results in better average and lower standard deviation, which means that it's more accurate.

Also the use of the indeterminate model instead of the determinate model is a good decision of the authors, because it ensures that the failure strengths of pile caps isn't overestimated as Kim et al. (2013) proved in their research.

	Effective strength								
Model shape	Marti (1985)	MacGregor (1997)	Eurocode 2 (CEN 2004)	fib (2010)	AASHTO (2010)	ACI 318 (2014)	Present method[a]	Present method[b]	Nodal zone shape
Model A, $z = 0.90d$	1.46	1.61	1.57	1.76	1.42	1.63	1.27	1.16	ACI 445 approach
Model B, $z = 0.95d$	1.69	1.92	1.85	2.13	1.64	1.94	1.36	1.15	
Model C, $z = 1.00d$	2.17	2.51	2.43	2.80	2.14	2.55	1.66	1.28	
Total Mean	1.71	1.94	1.88	2.14	1.66	1.96	1.38	1.18	
Standard deviation	0.93	1.10	1.06	1.27	0.95	1.13	0.59	0.27	
CR (%)	71.9	69.6	71.0	67.8	75.1	69.0	79.4	81.7	
Model A, $z = 0.90d$	1.19	1.30	1.27	1.41	1.16	1.32	1.07	1.04	PCA approach
Model B, $z = 0.95d$	1.47	1.69	1.63	1.87	1.43	1.71	1.19	1.03	
Model C, $z = 1.00d$	2.17	2.51	2.43	2.80	2.22	2.55	1.64	1.24	
Total Mean	1.49	1.69	1.64	1.86	1.43	1.71	1.22	1.08	
Standard deviation	0.74	0.87	0.84	1.01	0.70	0.90	0.47	0.26	
CR (%)	71.6	69.0	69.0	65.8	73.6	69.0	78.8	81.2	

Note: CR = concordance rate of failure mode; d = effective depth of pile cap.
[a]Prism-shaped strut.
[b]Bottle-shaped strut.

Figure 3.11: Strength prediction ratios

Overall we would recommend the use of this concrete model for three-dimensional structural members. The method is described in steps that can be implemented in finite element models from other authors and could be combined with their improvements.

3.1.7 A simplified approach for the ultimate limit state analysis of three-dimensional reinforced concrete elements

In this paper (Meléndez, Miguel, & Pallarés, 2016) a new simplified approach is presented for the ultimate limit state analysis of three-dimensional reinforced concrete elements. This proposed model permits the study of the strut-and-tie method for 3D elements because it can adopt different uniaxial stress-strain models such as neglecting tensile strength of concrete. The authors also made a non-linear finite element-based tool for this proposed method. With this proposed model, the flow of forces can be identified and this allowed that the structural response could be better understood.

3.1.7.1 Outline of the research

The model developed in finite element in this study is orthotropic so that 3D response of concrete can be split into three directions and to be able to treat these three directions separately. The proposed model for concrete behaviour can be found in the paper itself.

The authors developed a software tool called FESCA 3D. It is a non-linear finite element based tool and is used to analyse and design three-dimensional concrete structural elements. In this model, it's very important to define well-chosen boundary conditions. A twenty-node serendipity hexahedron was picked for the modelling of concrete.

They compared the method by Souza et al. (2009) and their model with 1) neglecting concrete tensile strength, 2) adopting MFCT-model and 3) adopting Hordijk's model (Hordijk, 1991) for concrete. In this comparison it can be seen that the proposed model with the adoption of Hordijk's model is most accurate.

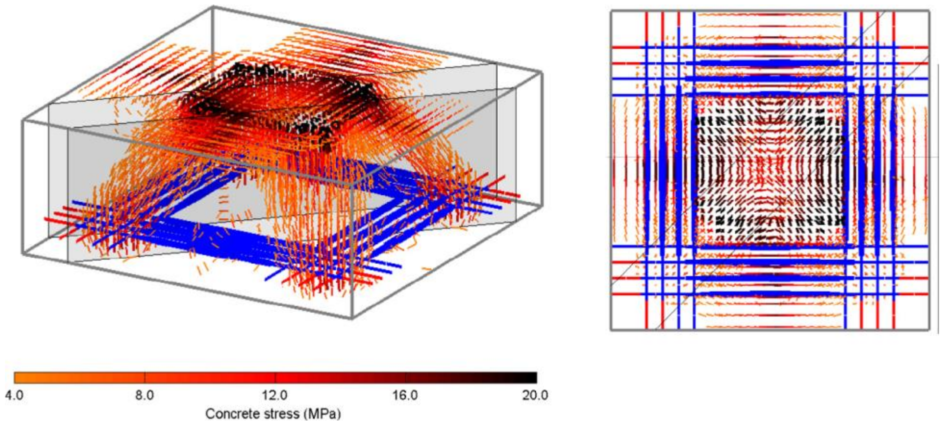

Figure 3.12: Stress representation with FESCA3D

The authors stated that, if the tensile strength of concrete is neglected, the proposed model can be used to ascertain the goodness of the strut-and-tie method in order to analyse concrete structural elements with a three-dimensional behaviour.

3.1.7.2 Discussion

First of all, we want to mention that this paper was rather difficult to understand with our knowledge of three-dimensional structural concrete modelling. For comparing different concrete models, we rely on the ability of the author to conclude which model is best.

Concrete is a brittle aggregate material, thus its behaviour is dependent on the components of the concrete and their interaction. Because of this, concrete never responds the same everywhere in the concrete. Therefore constitutive models are developed to overcome this problem. But these models are often difficult to apply for practitioners because of their complexity. Therefore, concrete response should be idealised for the use in common engineering problems. But the idealisation of the concrete response isn't easy either, despite this, the authors were able to propose a simplified and comprehensive model that adopts uniaxial stress-strain laws to characterise the response of 3D structural elements.

The results that were found, were really good with an average of 1.01 and a COV of 3-4%. This means that the experimental and calculated load are almost the same and with a COV of 3-4%, the scatter of the results was rather good. So these results were not only correct but also really accurate for the proposed method.

However implementation in code provisions is not easy considering finite element approaches. But the tool which was developed in this paper could be useful for future researchers to test different concrete models. Considering the outcomes of different models implemented in the tool, some recommendations can be given for use in more simple three-dimensional STM models. As a matter of fact, a refined strut-and-tie model was developed by the same author based on the conclusions of this paper. The refined model is discussed in the next section of this master's thesis.

3.1.8 Refined strut-and-tie model for predicting the strength of four-pile caps

This chapter out of the doctoral thesis (Meléndez, 2017) presents an alternative STM-based approach for the calculation of the ultimate strength of rectangular four pile caps, without shear reinforcement and loaded by a square column. The innovation of the model is within the consideration of non-fixed truss geometry. The location of the upper nodes is not fixed in advance, making it possible to maximize the pile cap strength considering different failure conditions.

3.1.8.1 Outline of the research

The geometry, as mentioned earlier, isn't fixed. Four lower nodes are located on the level of the centroid of the reinforcement and the four upper nodes within the area of the column (horizontally not fixed) at the surface level of the cap, see Figure 3.13. The model is statically determinate so all member forces can be easily calculated from the geometry.

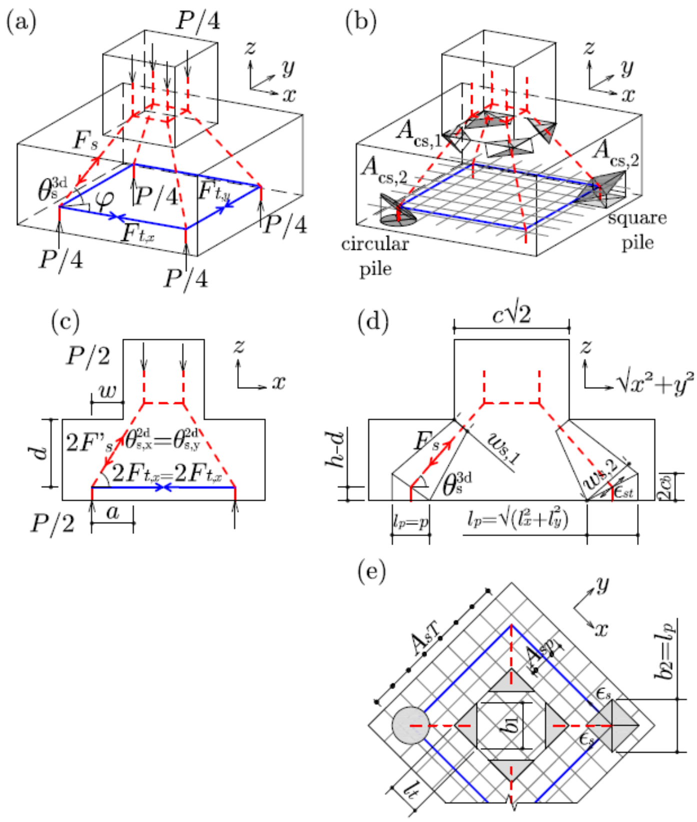

Figure 3.13: Proposed STM model for square four-pile cap

To evaluate the ultimate load of the pile cap, two unknown variables have to be solved. These are the unknown angle of the inclined strut and the ultimate load. To do this the author has proposed to check three failure modes which are solved by maximizing the element strength.

The three failure modes are described below and for all three of them the author derived, what he calls, limit functions that need to be solved:

1) Failing of the reinforcement ultimate stress f_{su}

$$P_{nt} = 2\sqrt{2}\tan\theta_s^{3d} A_{sT} f_{su}$$

2) Crushing of the diagonal strut at the base of the column

$$P_{ns,1} = 4\sin\theta_s^{3d} A_{cs,1} f_{cp}$$

3) Splitting of the diagonal strut due to transverse cracking

$$P_{ns,2} = 4\sin\theta_s^{3d} A_{cs,2} \zeta f_{cp}$$

With f_{su} the ultimate steel strength, ϑ_s^{3d} the strut inclination, ζ the compression softening factor and A_{sT}, $A_{cs,1}$ and $A_{cs,2}$ respectively total steel area and strut areas at top and bottom. More details on these parameters can be found in the paper.

It is the extra equation $P_{ns,1}$, that accounts for crushing of the concrete at the top of the pile cap, that permits this solution. In existing STM methods the flexural (P_{nt}) and shear ($P_{ns,2}$) strength are derived separately and the minimum represents the ultimate load. In the proposed approach, it is the intersection of these two functions with the 3rd extra function $P_{ns,1}$ that shows the mode of failure according to the traditional methods. When adding the yield limit function P_{yt}, to indicate the yielding of the reinforcement, it can be visually seen if failure occurs before or after failure as shown in Figure 3.14.

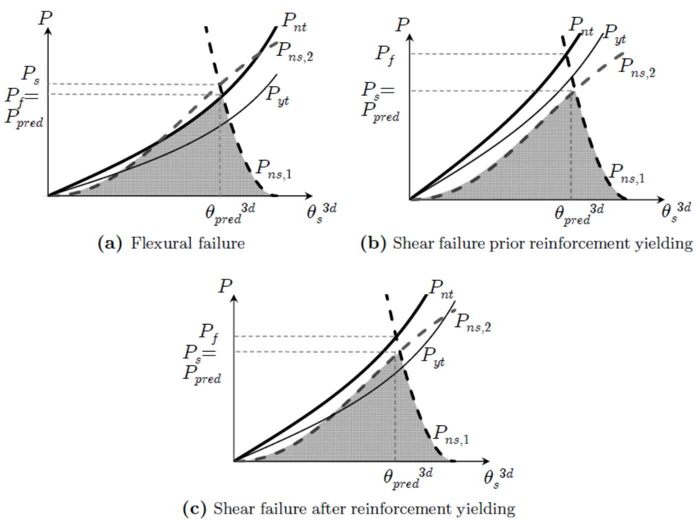

Figure 3.14: Limit functions to predict failure mode

3.1.8.2 Discussion

The ideas in this particular chapter from the doctoral thesis of C. Melendez (2017) are adopted from previous chapters. In these chapters, the author has tested the effect of a refined concrete model or a simple concrete model for deep pile caps in his own developed FE-based tool. The conclusions were that a refined model that considers concrete softening gives better predictions. Relying on the accuracy of this tool, we conclude that the author has been able to prove that his concrete model is both accurate and simple for the prediction of pile caps, which is an advantage to implement in a STM model.

The first mode of failure (1) by exceeding the ultimate strength of the reinforcement doesn't need further explanation and is easily understandable. The assumptions that are made to determine the area of reinforcement is both simple and logical. The reinforcement is assumed to be bunched over the pile caps, symmetrical in both directions. This fits well with provisions on STM.

Mode of failure (2) needs more attention because of some assumptions. The first one is the use of the plastic strength of concrete f_{cp}. The author assumes that this top area below the base of the column can be represented by an uncracked uniaxial compressive state. This assumption was verified by experiments and FE and is therefore conservative. However, we want to remark that other authors recently added their opinions that this zone could be better represented by a triaxial state of stress. This makes the concrete more efficient and can enhance the capacity of the pile cap. In this case, the author has chosen for simplicity instead of accuracy, considering the practical use of his model.

Mode of failure (3) represents splitting of the inclined strut. Because this region is affected by tensile strains perpendicular to the strut axis, the author introduced the concrete softening coefficient ζ, which he adopted from Vecchio and Collins. This model seems to be acceptable as it is a well-known factor and the author was able to extend the formulation to 3D by introducing a strain compatibility condition into the model. In this compatibility condition, the author has succeeded to implement the effect the reinforcement layout. It isn't the purpose of this thesis to repeat the equations herein. We can only conclude that the author has proposed a simple and theoretically derived way to introduce the three-dimensional effects into his proposed method and only therefore it could already be an improvement on current code provisions on defining the effective concrete strength of struts for pile caps.

As mentioned earlier, all of these limit functions are a function of the angle of the inclined strut. The idea of taking this angle as an extra variable is both innovative and effective. This idea is really something that future researchers should take into consideration in constructing more sophisticated models. It has already been considered in another research by A. Mathern et al.

(2017) that we also discussed in our thesis. Both authors use the same idea of trying to maximize the capacity by refining the location of the upper nodes.

A first remark that we can make on this proposed approach, is the definition of the strut areas at the bottom and top of the cap near the column and above the piles. The areas are determined by simple projections of the column and the piles respectively. This way could be seen as an advantage because it is simple for practitioners. On the other hand, this is based on current code provisions that are made for two-dimensional members. Consequently the assumptions that are made here are possibly inaccurate. However, is was not the purpose of the author to focus on this matter.

Overall the author has succeeded in developing a refined STM model. All parameters described can be calculated easily in a spreadsheet and no software is needed. The only thing that needs iteration is the transverse tensile strength affecting the effective strength of concrete. Limitations on the derived equations for the model is that they are derived specifically for square, symmetrical four-pile caps. The model could be extended to other configurations when taking care of the proposed assumptions. To prove the effectiveness of the model, the author has compared strength predictions with the ones from other authors and the ACI STM. Figure 3.15 shows the comparison between the proposed model and the ACI STM, other comparisons can be found in the doctoral thesis. It's clear that the proposed model gives more effective and still conservative predictions. Current ACI STM provisions should give specific guidelines for three-dimensional members, which they don't do at this point.

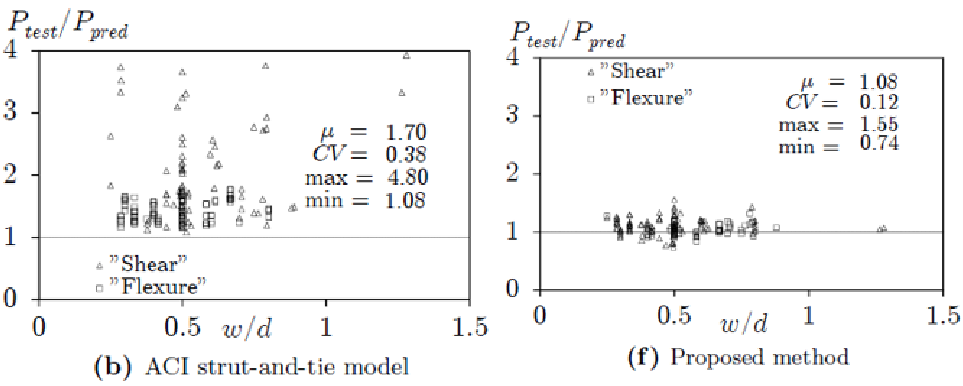

Figure 3.15: Strength prediction ratios

3.1.9 Enhanced strut-and-tie model for reinforced concrete pile caps

These authors (Mathern, 2017) present a new enhanced three-dimensional strut-and-tie model for the design and verification of four pile caps. Although the STM is accepted in many codes and applied for many years for pile caps, existing models do not expand on detailing specifically for three-dimensional issues. For example, the verification of nodal zones is often disregarded in the analysis although it is one of the most important steps. The study presented here developed a methodology to consider the intersection of ties and struts in nodal zones to enhance the definition of the geometry and area of them.

3.1.9.1 Outline of the research

The truss of this model consists of four upper nodes and four lower nodes. The vertical location is respectively fixed at the centroid of the upper concrete struts and the centroid of the flexural reinforcement. The horizontal positions of the upper nodes is explained to be at the corners of the column as shown in Figure 3.16 and described more in detail in the paper.

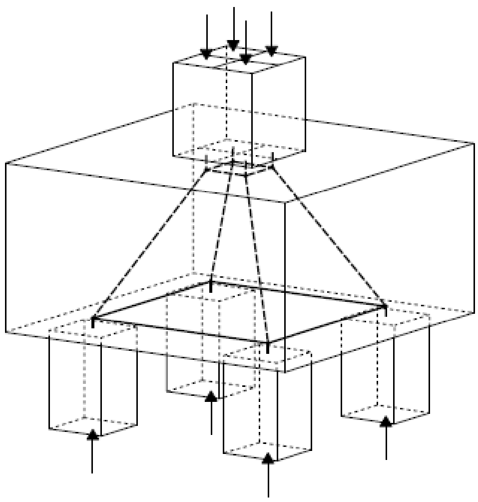

Figure 3.16: Proposed STM model for four-pile cap

The focus of this paper is most on trying to give better definitions of geometry of nodal zones and simultaneously this affects some principles considering the concrete strength. The main treated principles which are implemented in the proposed model are listed below:

- Equilibrium: The truss model is statically determinate so forces can be calculated from the geometry.
- Refined three-dimensional nodal zones: A new definition for the geometry of the nodal zones is proposed using a parallelepiped from which the strut area becomes a hexagon area, see figure. Also the horizontal position of the nodal zones are optimized by iteration.

- Triaxial strength of concrete: Tor the maximum strength limitation of the concrete in critical sections, the more favourable triaxial compression state of stress in nodal zones is used based on EC2.

- Confinement of concrete: A bottle-shaped geometry of the struts is assumed considering the confinement by plain concrete which enhances the strength of them. The strength criterium based on this effect is adopted from Adebar and Zhou (1993) and can be found in the paper.

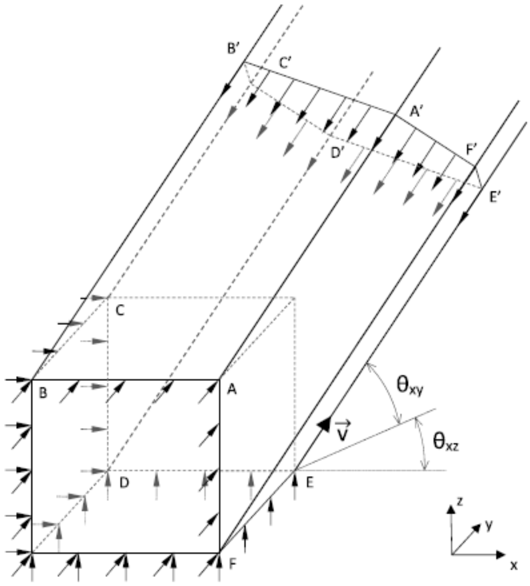

Figure 3.17: Refined nodal zone geometry

To determine the ultimate capacity of the pile cap, the authors propose a design procedure taking into account all the characteristics which are described above. The procedure finds a maximum capacity when all failures listed below do not occur:

(1) Yielding of the reinforcement

(2) Bearing stresses in nodal zones under column and over piles (crushing of nodal zone), with the triaxial stress criteria

(3) Splitting and crushing of the struts (shear failure), with the strength criteria from Adebar and Zhou

3.1.9.2 Discussion

The authors developed an enhanced STM model with the focus on nodal geometry, while previous authors have never done this that extensively before. The reason why did this is because the geometry of the truss and the stress check of nodal zones are two explicitly mentioned steps in a STM design procedure. Other authors often use two-dimensional similarities to base the geometry on and therefore we are convinced by the significance and the need of the research that has been done in this study.

Related to the geometry of the nodal zones, is the location of the nodes within these zones. The optimization that has been proposed, is to locate them outside at the columns corner and consequently the resulting forces in the STM members will decrease. This means that the capacity of the pile cap will be maximized. Unfortunately, this maximizing procedure can only be done iteratively with software and consequently it can not be adopted into design codes for practitioners. However, we believe that refining the geometry of nodal zones is a necessity for three-dimensional structural modelling and future researchers could adopt this idea into their proper models.

Considering the failure modes, the authors make two large assumptions. To check the stress limit in the bearing area of nodal zones they propose the triaxial state of concrete and adopt the stress limit from EC2. When doing this, the strength of the nodal zones increase and consequently the capacity increases. We are nevertheless concerned about the conditions that have to be proven before implementing this stress check. Researchers should take special care when dealing with this issue.

While other STM models sometimes take into consideration a compression softening factor for the concrete inclined struts, these authors don't mention this phenomena. Moreover they adopt the strength criteria from Adebar and Zhou to take the compression strengthening effect of confined concrete into consideration. The model therefore relies on the definition provided by these authors.

When comparing the model for overall strength predictions it seems to lead to conservative and effective predictions. In Figure 3.18, the comparison is made with some alternatives proving the effect of the implemented principles. We are convinced by these results that the aforementioned principles can be useful for future researchers to implement in their models, especially considering the refinement of geometry.

	Enhanced Model	Alternative 1	Alternative 2	Alternative 3
3-D nodal zones	✓	(✓)	✓	
Refinement nodes	✓		✓	✓
Confinement strut	✓	(✓)		✓
Iterative improvement	✓	✓	✓	✓
Average F_{exp}/F_{pred}	1.14	1.53	1.25	1.08
Standard deviation	0.12	0.35	0.15	0.17

Figure 3.18: Strength prediction ratios

3.1.10 Punching shear failure in three-pile caps: influence of the shear span-depth ratio and secondary reinforcement

The emphasis of this research (Miguel-Tortola, Pallarés, & Miguel, 2018) is on the formulation of punching shear failure. Two parameters that are investigated more in detail are the shear span-depth ratio and the use of secondary reinforcement and vertical stirrups on the pile cap strength. The existing STM's aren't able to catch neither the effect of the shear span-depth ratio, neither consider failure by punching shear. The authors eventually proposed some adapted formulations based on EC2.

3.1.10.1 Outline of the research

The geometry specifications following the STM proposed in Spanish standard EHE-08 are adopted as shown in Figure 3.19. There's only one top node located at the centre of the column at 0.85d from the reinforcement axis. Lower nodes are assumed to be above the centre of the piles on the level of the reinforcement centroid.

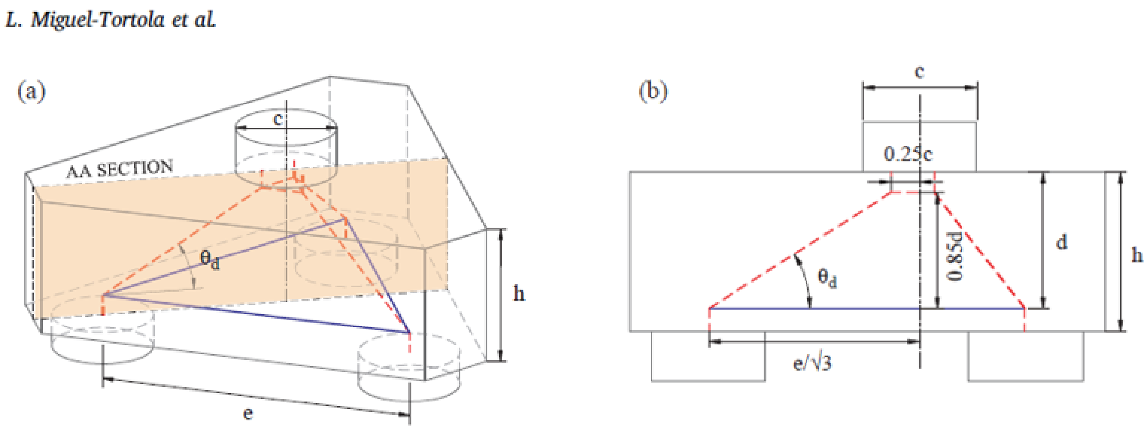

Figure 3.19: STM model for three-pile cap

The authors made 9 specimen divided in three groups with varying depth. Within these groups, specimen were made without secondary horizontal reinforcement (according to STM), with horizontal reinforcement and lastly with extra vertical stirrups as well. The experimental observations are given below:

- There is a clear trend between the shear span-depth ratio and the ultimate load. Deeper pile caps showed higher ultimate loads than the more slender pile caps.
- Adding secondary reinforcement increased the ultimate capacity depending on the depth. Horizontal reinforcement seemed to be most efficient for more slender caps and vertical reinforcement was most efficient for the deeper caps.
- Adding secondary reinforcement clearly reduced the width of the cracks.

On the other hand a new method was proposed to verify the ultimate strength of pile caps. They added a punching shear failure verification to the STM model of Suzuki and Otsuki. The punching shear formulation from EC2 (Eurocode 2, 2005) is adopted and presented below:

$$V_{Rd,c} = \left[\frac{0.18}{\gamma_c}(f_c 100\rho_l)^{1/3} k \frac{2d}{a_v}\right] ud$$

The authors have proposed a better formulation for the basic control perimeter *u* and the shear enhancement factor *2d/a*$_v$ in this equation, which suits better for the specific deep pile caps geometry. The new perimeter is derived from experimental observations as shown in Figure 3.20 and is far from being a conical surface as described in codes.

Secondly the authors propose to apply the shear enhancement factor on a reduced part of this new perimeter, which is given by:

$$u_{eff} = u + (\frac{2d}{a_v} - 1)w_{eff}$$

With w_{eff} the effective width on which the shear enhancement factor is effective, given by the average of the pile and column diameter:

$$w_{eff} = (c + \emptyset)/2$$

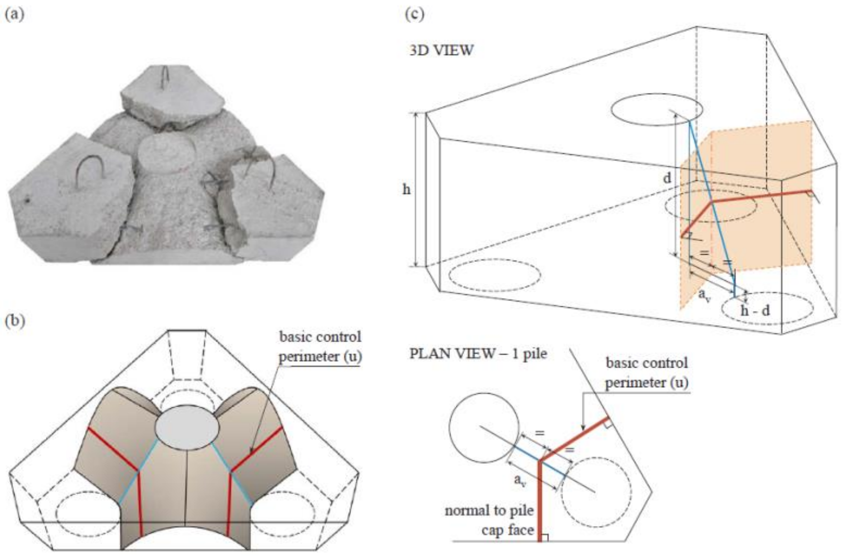

Figure 3.20: Enhanced basic control perimeter

3.1.10.2 Discussion

Based on the experimental observations we conclude that adding secondary reinforcement should be concluded in design codes. Several codes already adopted this as a percentage of the main reinforcement for crack control, but it has been proven in this experimental investigation that it has positive effects on the pile cap strength capacity as well. However, further research is needed to be able to provide a specific formulation for the amount.

The new specific definition for the basic control perimeter for deep pile caps is a necessity as current codes do not provide accurate definitions for this particular case and the punching shear strength is strongly affected by this surface. Current codes are not able to capture the effect of loads near to the column which is the case for deep members. Figure 3.21 from section 6.4.2 of EC2 give the practitioner the only guidance in proposing basic control perimeters. It is clear that designing pile caps based on this information is inadequate. Therefore, we are sure that the given formulation represents better what happens in experiments specifically for pile caps.

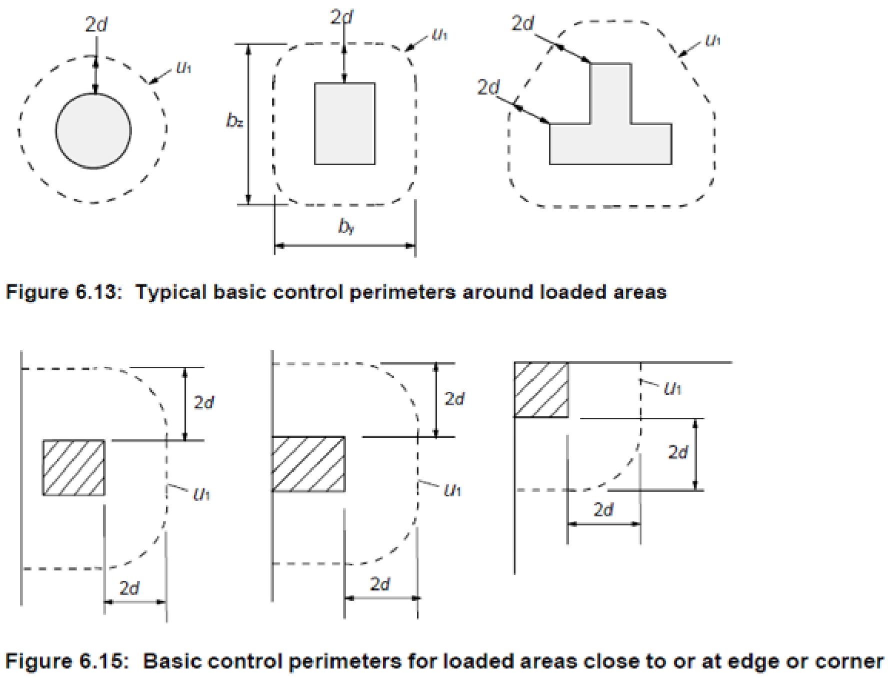

Figure 3.21: Basic control perimeter examples (Eurocode 2)

Moreover the application of the shear enhancement factor on a reduced part of the perimeter is proven to be conservative and more efficient. The proposed effective width is based on the conservative observations of previous researchers as described in the paper and therefore we assume that the formulation is reasonable. Figure 3.22 gives the graphical comparison between the effectiveness of the punching shear formulation of the EC2 and the reduced formulation by the authors. It is very clear that the proposed formulation comes out best with a mean value of 0.99.

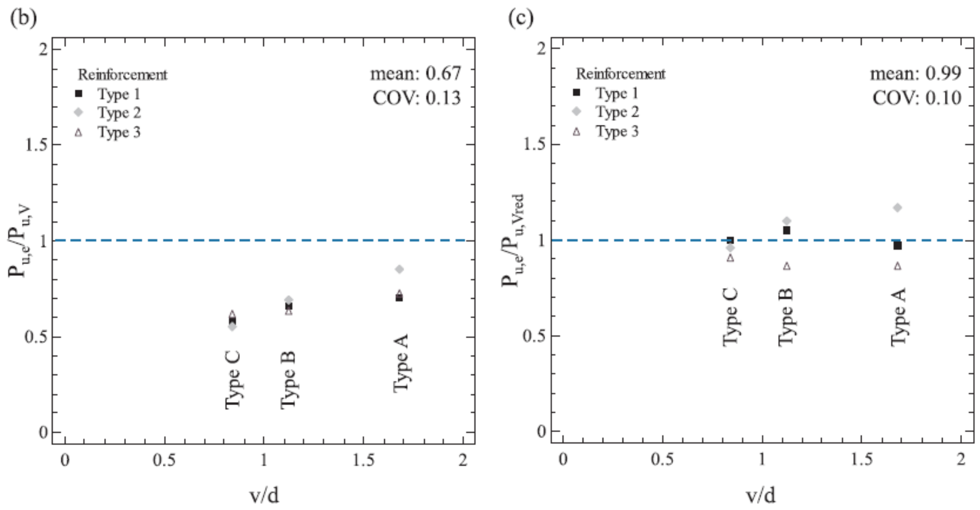

Figure 3.22: Punching shear prediction ratios: b) EC2, c) proposed method

For the prediction of the ultimate capacity of pile caps, the authors propose a combination of the STM predictions by Suzuki and Otsuki and the proposed punching formulation. The STM by Otsuki is chosen because the variable top node elevation makes it possible to account for stress redistribution. Because the authors adopted this STM, they are highly dependent on the effectiveness of this model. We can not ensure the use of this model because it wasn't examined by us, but adding the punching formulation has proven to enhance the predictions of the model as seen in Figure 3.23 for four pile caps. However, we suggest for future researchers that want to predict the capacity even more effective to not just adopt this STM because the mean value for the effectiveness of 1.55 is still pretty high.

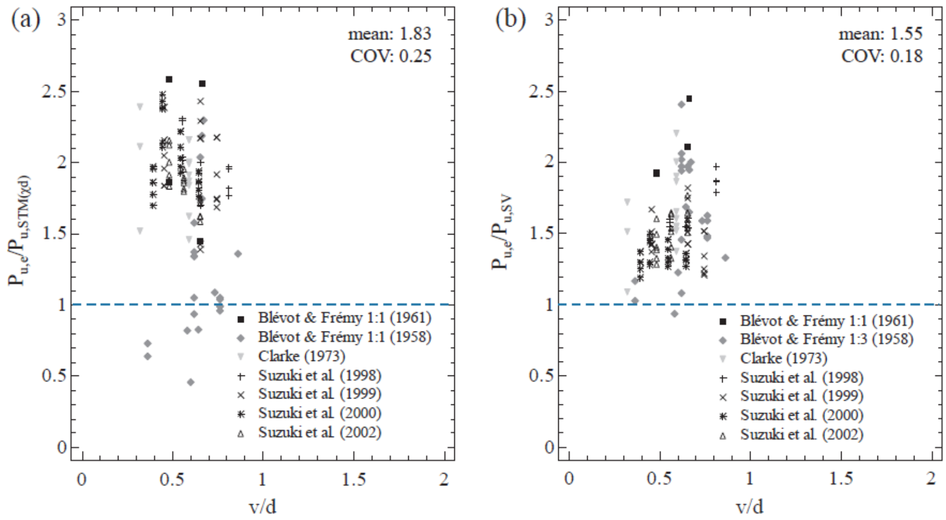

Figure 3.23: Strength prediction ratios: a) STM otsuki, b) Proposed method

3.1.11 Three-dimensional grid strut-and-tie model approach in structural concrete design

The authors (Young Mook Yun, Kim, & Ramirez, 2018) present a 3D grid strut-and-tie model approach for the analysis and design of three-dimensional structural concrete. This new approach consists of three major steps: the use of grid elements to build an STM model, triaxial stress of concrete to determine effective strength of struts and nodal zones and an iterative procedure to evaluate the axial stiffness of struts and ties. Finally a new concept of maximum cross-sectional area of struts and ties is implemented. The model captures the three-dimensional behaviour of pile caps better than existing code STM provisions that are mainly developed for two-dimensional members.

3.1.11.1 Outline of the research

There is not a fixed model that is used in this approach. However, it is the intention to give guidelines to construct a solid 3D model. They do this by combining what they call basic grid elements. One grid element contains in total 28 components in a cube as shown in Figure 3.24. In the first step of the design method, all the finite grid elements are selected that are in the area of the inclined strut.

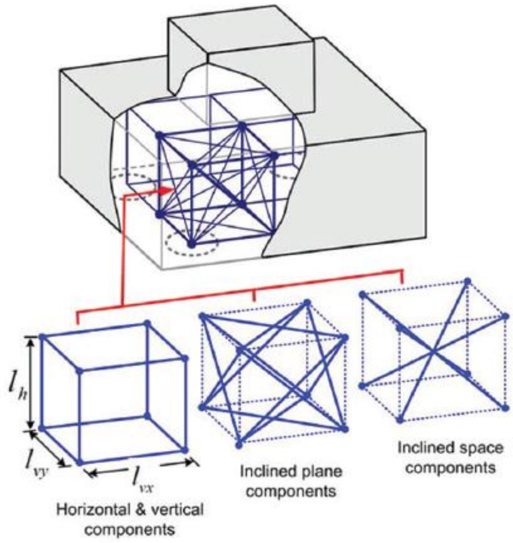

Figure 3.24: Basic grid element

In the next step an iterative procedure is described to determine the effective strength of the concrete strut considering the triaxial stress state of the concrete. This procedure has already been described in an earlier paper that we have discussed in this thesis, thus we will not repeat it in this section for the concrete struts.

To check the strength of the nodal zones, the authors use an approach that is implemented in existing design codes. They compare the maximum surface areas with the required area. The maximum surface area is derived considering the areas of the connecting ties and struts and the considered nodes as described in the codes. Additionally to this, the authors have developed a procedure which is compatible with the geometry of the grid elements. The maximum cross-sectional area of the ties and struts are given by the maximum areas in which the grid elements don't overlap. For further details, we refer to the section "*Geometrical compatibility of grid strut-and-tie model"* in the paper.

The required area is simply derived by dividing the sectional forces in the members by the effective strength of the nodal zones. To compute the effective strength of the nodal zone, the authors once again use a five-parameter failure model of 3D concrete, similar to the procedure described for the effective strength of the struts. Figure 3.25 shows the steps to follow to determine the effective strength.

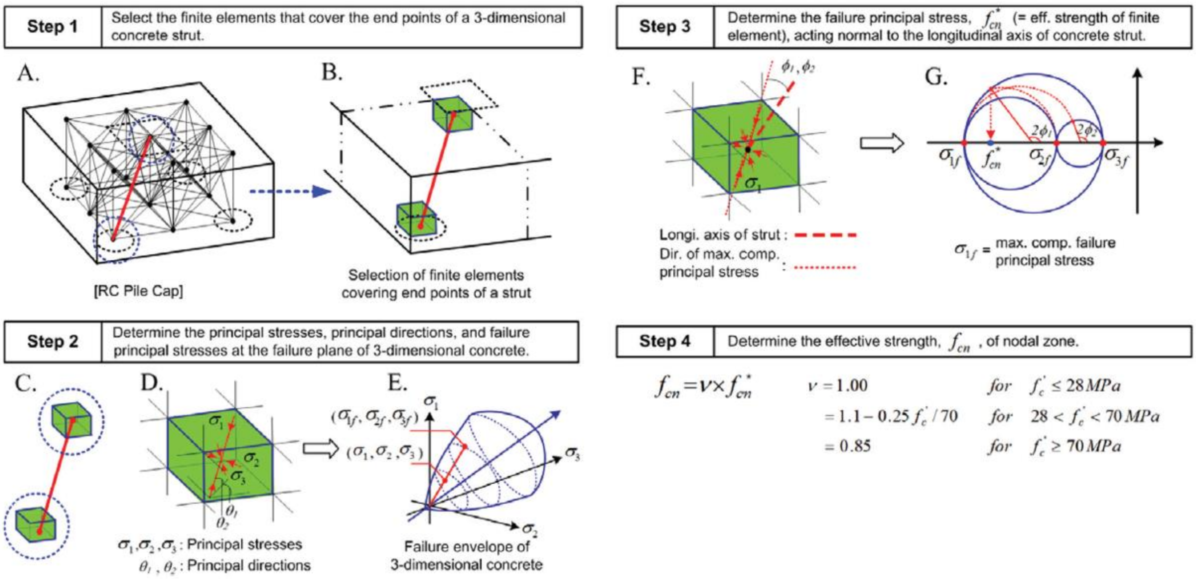

Figure 3.25: Proposed steps to determine effective strength of 3D nodal zones

The proposed STM approach was validated against a dataset of 78 pile caps in a subsequent paper by the same authors. A useful comparison is made with ACI sectional approaches and the ACI STM provisions.

3.1.11.2 Discussion

First of all we have to state that we are no experts in the domain of finite element modelling. It is beyond the scope of this thesis to go further into detail on this matter. In this discussion we have tried to pick up and explain the most innovative ideas behind the 3D grid STM-model, which uses FE, that could be adopted in more simplified geometrical models according to us.

As stated earlier, the model described by the authors can be used to represent the loads paths within the mass of concrete in a better way than previous techniques. Considering the lack of guidance in current codes for three-dimensional members, we are convinced that there is a necessity for such new approaches. The proposed approach gives better guidance in the development than, for example, the approach that considers elastic stress trajectories. This is because the proposed method incorporates better the effects of a 3-D disturbed region with non-linearity. Moreover this method can generate statically indeterminate models. The advantage that these models have over determinate models, which are most often used by previous authors, are that they could potentially present the flow of forces more realistically.

Another innovative idea that is implemented in this model is the renewed way of determining the effective strength of both struts and nodal zones. Struts and nodal zones in this model are permitted to reach a triaxial stress level. It must be noted that proper confined by plain concrete or crossing reinforcement is assumed. By adopting this into their model, they propose the use of this limit state as a general rule for three-dimensional members. This is an idea that isn't described, for example, in the ACI318-14 STM specifications. This is an important adaption to current two-dimensional provisions and could or should be adopted in codes after further development.

These are two great improvements compared with current codes and other simple strut-and-tie models for pile caps. To prove our opinion on considering these improvements for implementation in standard codes, the authors have made a comparison between strength predictions from a simple STM model proposed in ACI 445 following ACI318-14 provisions. The model shown in Figure 3.26 shows the simple STM next to the statically indeterminate developed 3D grid STM model by the authors.

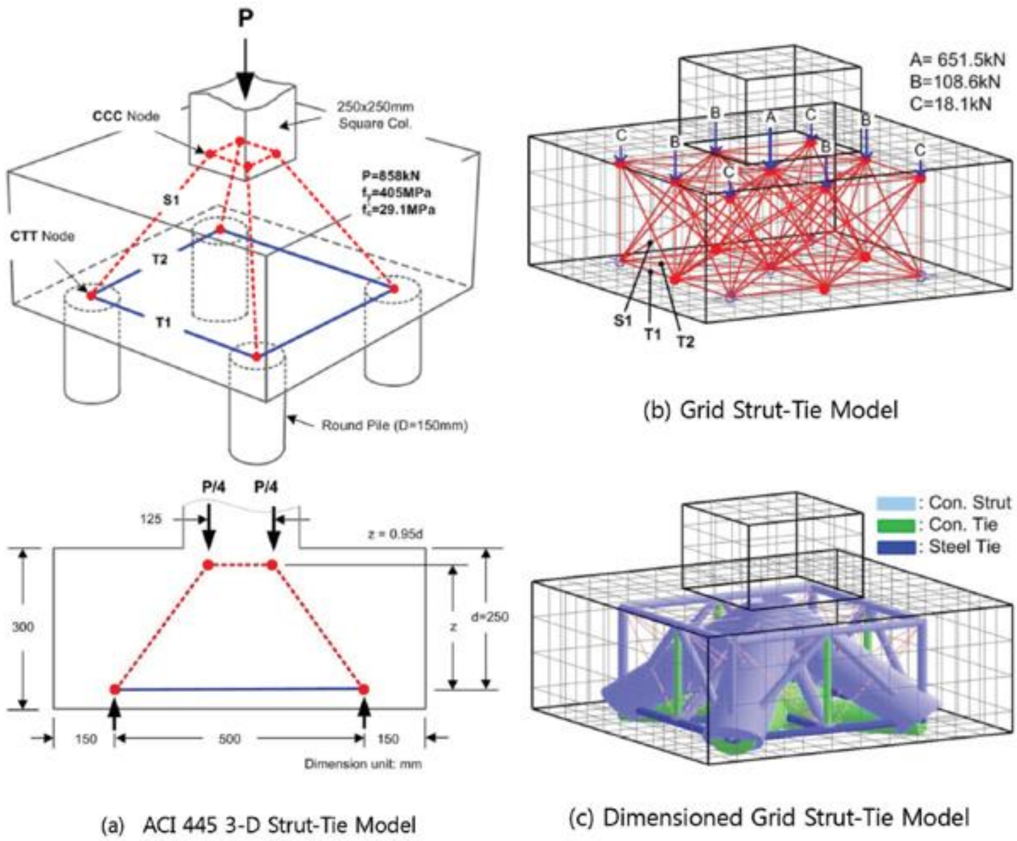

Figure 3.26: Simple vs indeterminate 3D grid STM model

The use of triaxial models of concrete is always quite complicated to implement into the finite element analysis. However, the authors have attained to perform this difficult matter anyway by introducing it in several steps that are easily understandable for adoption into other models. The comparison between the strength predictions from this traditional simple STM model of ACI and the proposed grid 3D grid STM model is given in Figure 3.27. The proposed method gives a more favourable effectiveness factor of 1.12 compared to 2.29 for the considered database.

Our final conclusions are that it is shown that statically indeterminate models could represent the flow of forces better and that the approach for implementing a triaxial model of concrete by the authors are valid for further use. The idea of constructing a model using basic and auxiliary grid elements also permits to construct models for other concrete members than pile caps.

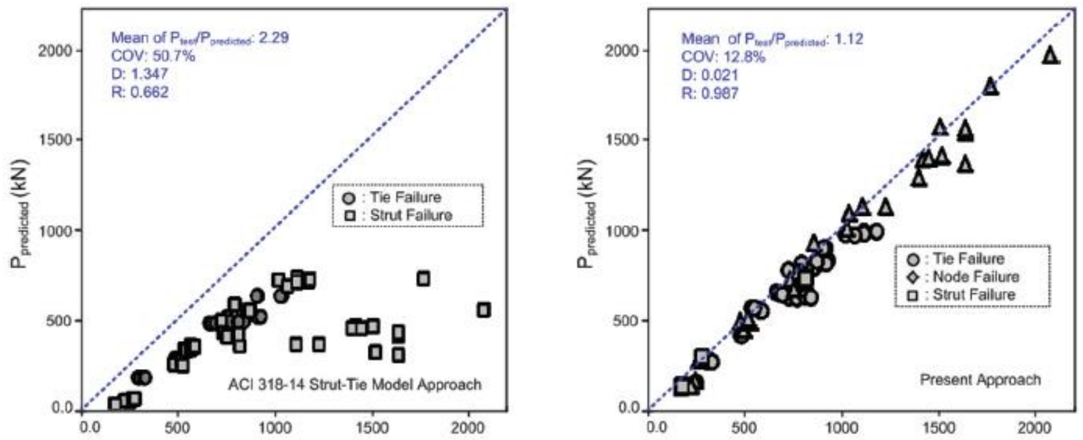

Figure 3.27: Strength prediction ratios

4 Conclusions

All these papers separately have contributed to enhance the interest in developing three-dimensional strut-and-tie models. Throughout the discussions, it became clear that there doesn't exist one unified approach for dealing with STM models for pile caps. Different authors focus on different aspects and often try to develop or enhance one specific three-dimensional related part of the modelling. Therefore, it's not easy to determine which approach gives the overall best representation of the behavior of pile caps for the design and analysis. However, the discussed papers can somehow be divided into groups considering the simplicity of the proposed methods and based on the main goals and improvements that were suggested. In Figure 4.1 below, a proposed STM model with simple geometry is shown, presenting the most important improvements according to us.

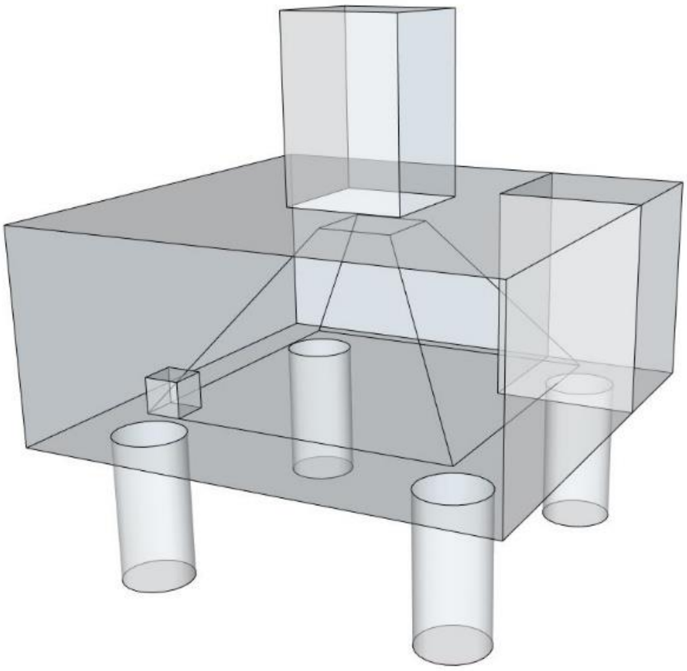

Figure 4.1: Proposed STM model showing the improvements

A first group of papers are those in which simple fixed STM models are used and where the authors decided to focus on improving this model considering different aspects that are essential in the design of pile caps. The oldest examples of these proposed methods are the ones from improvement 1, 2, 3 and 4 from our table of improvements. We must note that these proposals have shown to be useful, but that they are already a little outdated. However, to improve current codes of practice, the authorities should at least be able to implement such simple model specifically for pile caps, which they fail to do at this point in their section describing STM. We have chosen for a simple model with four upper nodes instead of only one because this is used by most researchers and it has the advantage over one single node that the horizontal location of the upper nodes could be a variable.

Within this group of simple models, improvement 5 and 10 have suggested some ways to add a punching shear failure check to STM-modeling. They both use simple truss models, but have suggested improved equations to check punching shear failure specifically for pile caps. Both ideas could be easily adopted into codes of practice according to our discussions. At this point, none of the codes mentions to perform a punching shear failure verification when designing with STM. Figure 4.2 shows a proposed enhanced control perimeter according to the observations of improvement 10.

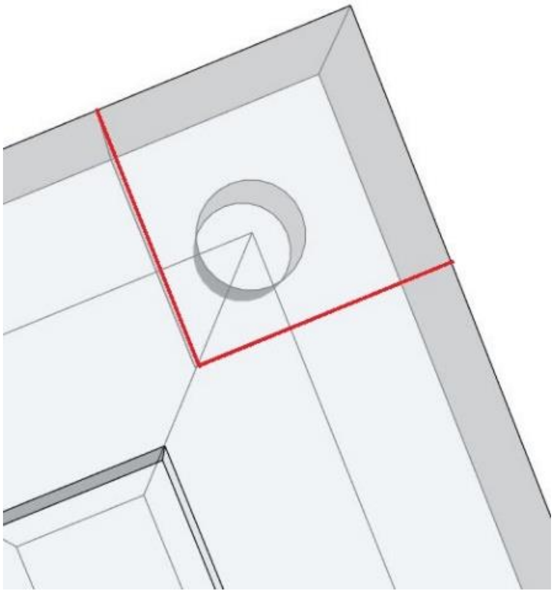

Figure 4.2: Enhanced basic control perimeter for pile caps

The second group of papers are those that focus the most on the three-dimensional behavior of pile caps and have used finite elements for their models. The most important lack of guidance in current codes of practice for three-dimensional STM designing are the definition of the effective strength of concrete struts and the determination of nodal zone geometry according to our findings throughout the discussions. Current codes only offer guidelines for STM that are based on two-dimensional members like deep beams. Logically, designing pile caps with these provisions can't lead to accurate designs and could be even unconservative.

Improvement 6 and 11, from the same authors, have captured the issue of the effective concrete strut strength the best in our opinions. They don't follow code provisions, but have developed a new method to consider multiple factors that affect three-dimensional concrete struts and nodal zones. They have managed to find a way to model the triaxial stress state of concrete in a proper and understandable way with finite element. The method uses basic grid elements like the one presented in Figure 4.1, in the shape of a parallelepiped to represent the three-dimensional stresses. This is a huge improvement because considering the triaxial state of concrete could enhance the capacity of pile caps multiple times, leading to less conservative strength predictions. We would suggest future researchers to implement these principles into their own models or to keep developing this model.

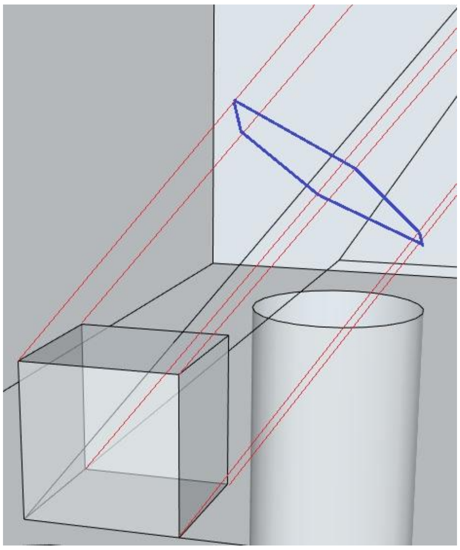

Figure 4.3: Enhanced nodal zone geometry

The second great improvement is on the nodal zone geometry and comes from improvement 9 and in a lesser extend from improvement 8. Both papers point out to refine the location of the upper nodal zones by maximizing the strength of the struts. This is possible by locating the nodes outside at the corners of the column as demonstrated in Figure 4.4. Moreover, improvement 9 has developed a model with consideration of three-dimensional nodal zones and is adopted into our proposed model, see Figure 4.3. Because they use a parallelepiped to do this, we consider that this idea could be combined with improvement 11 that uses similar grid elements. In current codes no specifications for this issue are proposed, unless the two-dimensional provisions. According to us, the authors have performed an understandable and improved way to consider this matter. We would suggest future researchers to adopt their principles for further investigation.

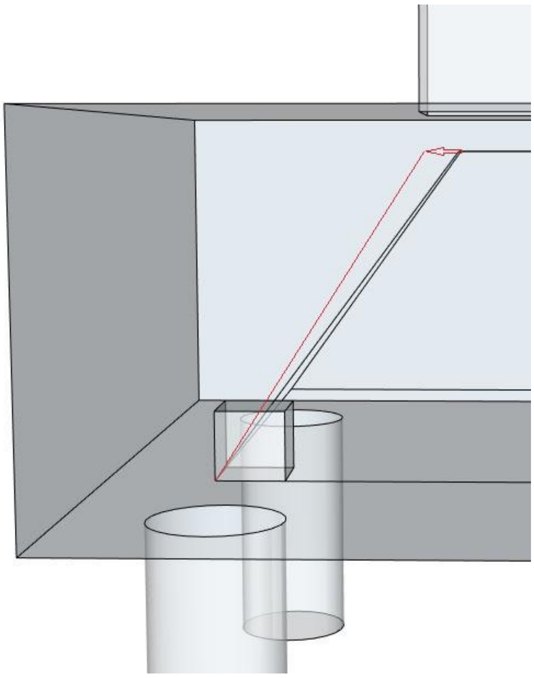

Figure 4.4: Refined node location

Overall, our entire table of improvements represents a complete list of the most recent and significant studies on three-dimensional strut-and-tie modelling for pile caps. All the ideas are explained and discussed and this document can be used for future researchers as a basis to adopt ideas for their own research. We suggest researchers to keep developing these models to minimize discrepancy and to become to a solid and conservative method to calculate pile caps using the strut-and-tie method.

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