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Practical, Efficient and Sustainable bridge pile cap design

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Abstract

In the design and analysis of bridge pile caps the requirements of AS51000.5-2017 can be interpreted by the designer to achieve an array of design solutions that may be compliant and fit for purpose but may not always be the most efficient solution.

Whilst there is ample guidance provided for bridge superstructure and substructure pile design the design of bridge pile caps is generally quite bespoke and open to designer interpretation.

This paper proposes a review of the options for pile cap design via different strut and tie models or flexural design for strength and serviceability. In trialling these approaches as applied to typical pile cap design examples the paper will provide an assessment of design, construction, cost and sustainability outcomes.

With a secondary focus on sustainability the underlying material cost and carbon footprint outcomes that result from designer choices should not be underestimated. It is hoped that the discussion on the benefits/differences between design approaches will provide a simplified guide to inform emerging designers to better understand best practice and the opportunity that we all have to work towards a sustainable future.

Keywords: Bridge, Foundation, Pile Cap, Strut and Tie

1. Introduction

Pile caps are an essential component of bridge design and construction that are often taken for granted. Whilst the superstructure provides the obvious visual headline with distinct types such as Super-T, composite steel girder, steel truss, segmental precast, balanced cantilever or cable stayed that have ample detailed design guidance for modelling and analysis the humble pile cap is often overlooked as a simple conventional reinforced concrete element.

Some bridge sites are blessed with dense sands for spread footings or founding rock sockets for mono-piles, whilst others may have demand for more substantially engineered complex foundations for longer span crossings. In the context of this paper, it is intended to focus on piled foundations as used for intermediate span length bridges in the range from 30-50m as is typically required on urban infrastructure overpass structures or viaducts where ground conditions are suited.

In these applications when presented with a pile arrangement the designer is given relative free license to manage the interface between pier column and piles as they see fit. In practice AS5100 provides minimal guidance in Part 3: Foundation design deferring to Part 5: Concrete where provisions written for standard beam, slab and column components require some design interpretation to be applied to pile cap design.

In interpreting these requirements there is limited potential for pile caps to be inadequately designed however there is an opportunity for pile caps to be more efficiently designed as will be explored in the following sections of this paper.

The paper will explore AS5100 design requirements as applied to pile caps and look at the outcomes that can result from the designers' choices. The information presented is intended to be more practical/common sense than State of the Art. The author has found from industry experience that there is generally a missing link between the Standards as written and the available commentary for this element of design and that it has been a more than worthwhile exercise to consolidate and provide guidance for developing engineers to shed light on and demystify this aspect of design.

2. General Design Considerations

For typical bridge foundations subject to vertical and out of plane loading and bending moments piled raft foundations provide an efficient means of transferring design loads to ground. Vertical loading is shared across the pile group with the overall spacing between the outermost piles in the group providing a lever arm to assist in resisting overturning moments. Depending on the specific load combination critical piles may be in differing positions within the pile group but are usually an outer edge or corner pile.

Depending on the complexity of the bridge superstructure to be accommodated the designer should determine pile loading through a combined structural/geotechnical model. This may be simplified where possible but should account for the intended span articulation and relative stiffness of the foundation as opposed to rigid supports. For example, in some structures there is the potential for pile displacement at ultimate limit state to lead to a rotation in the pile cap and a displacement at the top of the pier contributing to second order effects.

The potential for pile toe displacement at ultimate also allows for redistribution of pile loads once a limiting ULS capacity has been achieved. The use of suitably sophisticated software with provision for non-linear analysis can assist in this regard.

Piles

Piles are typically either reinforced concrete bored pile or precast driven pile. For the purposes of this review precast driven piles have been adopted as the most common and cost-effective solution based on Victorian industry practice. A typical 400mm square precast concrete pile the limiting ultimate limit state capacity of the pile can be in the order of 2400-2900kN.

This capacity is achieved through a combination of skin friction and end bearing and is subject to an additional geotechnical risk rating factor determined in accordance with AS2159-2009 based on a number of factors including the form of construction, ground investigation and testing proposed to verify the capacity of the pile on completion. Within a pile group to avoid any confusion it is most common to quote only the critical load and to apply this to the entire group. Whilst the required geotechnical capacity is often quoted on design drawings for clarity it is not necessary for the pile cap to be designed for any load greater than the ultimate structural loading.

Pile group configuration

AS5100.3-2017 limits minimum centre to centre pile spacing to 2.5 pile diameters where the capacity of the pile is developed by skin friction. For 400mm square precast driven piles this would theoretically allow spacings as low as 1.0m. In driving through founding materials, the supporting ground can become compacted and impact on driveability of any subsequent piles. Accordingly minimum pile spacings are generally set at a more generous 1.2 to 1.5m

In the absence of other interface restrictions, the pile group configuration is then set by the design loading arrangement, if there is a bias to lateral loading or overturning in one direction or the other a rectangular pile group may be preferred alternately if the loading is more balanced a square pile group should be considered. Often for bridges being constructed in brown fields sites there will be boundary constraints on the available construction footprint or utility clashes that can dictate the configuration of the pile group.

Pile cap depth

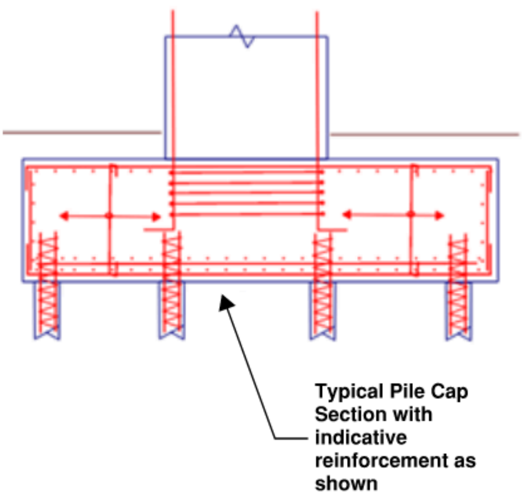
The depth of the pile cap is the most significant variable that the designer has control over. To minimise overall material quantities the temptation may be to minimise this dimension as much as possible. In setting this dimension the designer should account for pier column reinforcement and pile reinforcement anchorage requirements and any other inclusions.

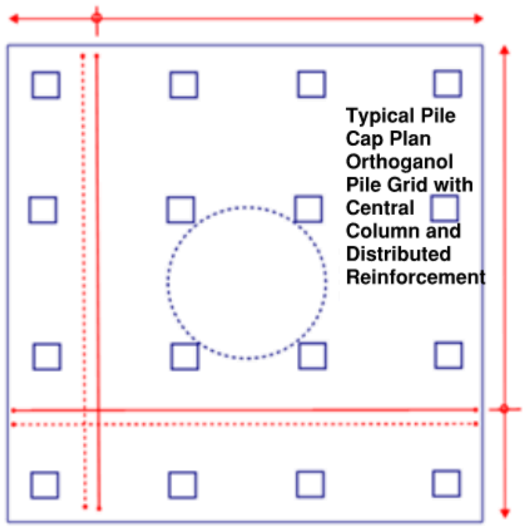
Conventionally reinforced columns may have large diameter reinforcing bars to be anchored within the pile cap, alternatively where precast construction is used for the pier column there will be complex post tensioning anchorages to be accommodated. Similarly driven piles will have reinforcing bars to be anchored within the pile cap.

It is also common for other inclusions to be required in the pile cap, modern bridges may need to accommodate concealed drainage pipes and utility conduits for ITS communications and lighting. Where these items are needed a clear zone should be provided for them to enter and exit the pile cap and pile group at the desired orientation.

Based on the above constraints pile cap depths in the order of 1.5 to 2.5m can be typically adopted. In conjunction with pile spacing above the span to depth ratio of a pile grid is typically less than 1.

Figure 1 – Typical Pile Cap arrangement





3. Structural Design Requirements

Non-Flexural Design

AS5100.5-2017 Section 12 allows for design of members including deep beams and pile caps where the ratio of clear span to overall depth is less than 3 for simply supported members and 4 for continuous members. For the pile cap instance, we would generally consider moment restraint at pile and column locations and could accept the span to depth ratio up to 4 which would cover most pile cap foundations as noted above.

Non-flexural design for strength then allows the pile cap to be designed using one of 3 approaches:

- Linear Elastic Stress Analysis
- Strut and Tie
- Non-Linear Stress Analysis

Of these approaches Strut and Tie is the most appropriate for pile cap design at the expected span to depth ratios. By comparison the Linear Elastic approach may be simpler but can yield more conservative design outcomes whilst Non-Linear Finite Element Modelling may be overly complex without significant benefit for typical design applications.

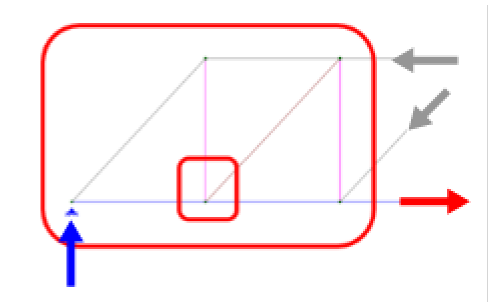
Non-Linear Stress Analysis for pile cap design is not specifically discussed in this paper, from experience the author has found benefit in this approach for structures subject to extreme catastrophic loading or for assessment of existing structures that are subject to increased loads for which they were not originally designed.

Strut and tie design basics

The requirements for Strut and Tie Design are discussed in AS5100.5-2017 Section 2.3 and 7.1. To avoid unnecessary repetition, they are not restated here in full other than to highlight relevant discussion points to assist in understanding their design impact.

- Strut and tie analysis is based on a truss analogy with members carrying axial loading only acting in tension or compression without any bending.
- The overall model and all nodes within the truss need to be in equilibrium, this can be assessed overall and by free body boundary on any section of the theoretical truss.

Figure 2 – Strut and tie truss model with free body boundary

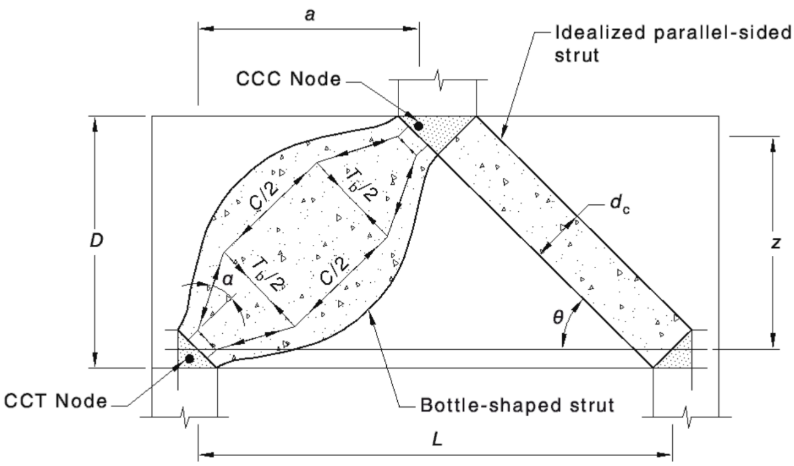


- There are no unique solutions, the designer assumes a model and it becomes valid when detailed accordingly. In reality the element may undergo some redistribution to arrive at the assumed model particularly for ULS. As a result, it is best to adopt simple models and complete secondary checks for SLS if cracking is a concern.
- Truss geometry needs to consider strut, tie and nodal dimensions. This means that the truss alignment should be set out with adequate space for the required strut depth/width and for ties that there should be allowance for cover and consideration of layering or detailing required to accommodate the required volume of reinforcement.

- Ties may cross each other however struts can only intersect at node locations. Noting these requirements in developing a model may lead to some trial and error, and the designer should not be afraid to change model geometry or overall dimensions if the model is not working or to get a more practical solution.
- Tension Tie reinforcement shall be anchored past the **centre** of any node with a minimum 50% anchorage length beyond the node. Tie reinforcement representing an idealised single element can be detailed as many elements with concrete responding to form a series of struts to each individual tie.
- Compression struts can be simple uniform prismatic struts or bottle shaped but may require additional bursting reinforcement to allow the compression load to spread out.

The minimum angle between a strut and tie is 30^{o}.

Figure 3 – AS5100.5 2017 Fig 7.2.4(A) Model of bursting forces in bottle-shaped struts (part)



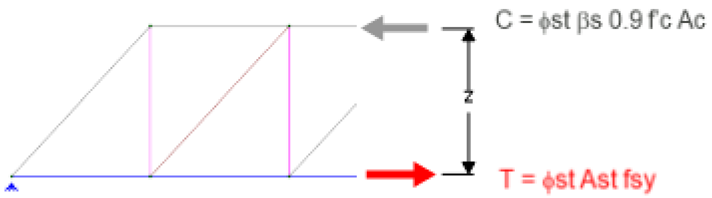
- The code requires an additional strut efficiency factor to be applied to bottle-shaped strut capacity based on the dispersion angle in the strut. This factor accounts for a potential weaking of the compression strut due to transverse splitting although the code also includes checks for provision of additional bursting reinforcement to compensate for this effect. The efficiency factor can be taken as 1.0 for prismatic struts but may be as low as 0.6 for the maximum allowable dispersion angle of 45^{o} at ULS. This significantly impacts on the efficiency of bottle-shaped struts.
- Similar to the strut efficiency factor the code also requires assessment of the principal stress on a node face with a reduction factor applied to account for tensile splitting of unconfined nodes subject to tension. As nodes typically experience load in 3 directions with either Compression or Tension they are defined as CCC, CCT, CTT (as shown in Figure 3) and have a reduction factor of 1, 0.8 or 0.6 accordingly. The code allows this reduction factor to be designed out with provision of confinement reinforcement at the critical node locations. In practice pile and column connections are provided with confinement reinforcement for potential plastic hinge formation which can be used to negate this issue.

Strut and tie vs linear elastic design for strength

- Strength Reduction factors for strut and tie analysis are generally consistent with flexural design at φst = 0.6 for concrete in compression and φst = 0.8 for steel in tension. These reflect the potential failure implications of brittle non-ductile failure for concrete and ductile failure for reinforcement yielding.

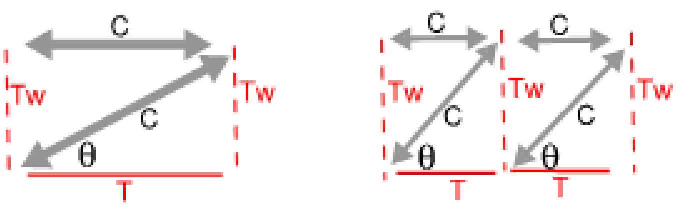
- For an equivalent section in bending the resulting tensile reinforcement is generally similar under both design approaches as the strength reduction factor for flexural bending is also $\varphi st = 0.8$. The resolution of bending moments via a tension compression couple with a lever arm z is similar to the rectangular stress block approach and for relatively deep pile cap design the section remains ductile and the depth of the rectangular stress block is similar to that of a compression strut. With compression set equal to tension the resulting capacity under either method is effectively the same.

Figure 4 – Strut and tie model Truss Bending Capacity



- The key difference between the design approaches is the method for resolution of shear forces. The Linear Elastic shear strength reduction factor of φ st = 0.7 is higher than φ st = 0.6 for a typical strut and tie concrete diagonal strut but reflects the presence of shear reinforcement throughout a typical flexural member. As shown in the figure below as a strut and tie model is adjusted to include vertical ties to improve the angle of the diagonal strut there is a reduction on the load demand on the diagonal strut the combined system can achieve a similar strength result with limited impact from the strength reduction factor.

Figure 5 – Strut and tie model shear capacity comparison

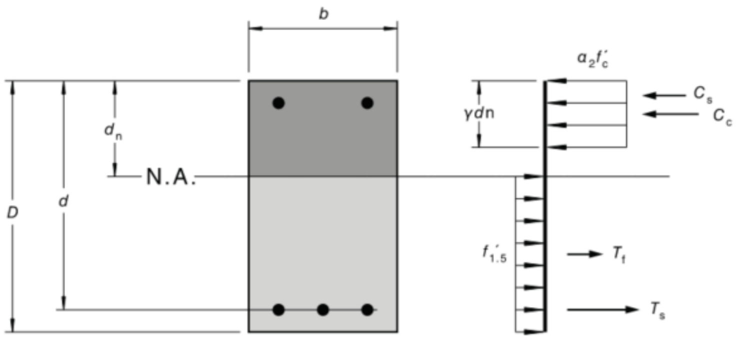


- A key point of difference between the two is the provision of the minimum shear reinforcement requirements under AS5100.5-2017 Clause 8.2.1.7. These requirements are not relevant to Strut and Tie design which allows the designer to provide vertical ties where necessary to suit their assumed model but does not mandate them through out. There are also minimum strength and reinforcement requirements in AS5100.5-2017 Clause 8.1.6 for flexural members but these are not strictly applicable and in the event that they are considered by the designer are typically readily achievable.

Steel Fibre Reinforced Concrete

AS5100.5-2017 now includes guidance for Steel Fibre Reinforced Concrete (SFRC) however it does not yet cover SFRC in strut and tie design. AS3600-2018 provides some additional commentary on SFRC for strut and tie design but is not considered transferable to Bridge design. With these provisions yet to make it into AS5100.5 the benefit of SFRC for foundation design comes from linear elastic analysis via the refined rectangular stress block with steel fibres providing additional tensile capacity below the neutral axis. Due to the large surface area of the concrete cross section this can be a substantial force and contribution to overall capacity although the depth of the resulting compression block is increased to achieve equilibrium. Shear capacity is similarly enhanced due to the depth of pile caps this contribution can be significant and can offset shear reinforcement requirements substantially.

Figure 6 – AS5100.5 2017 Fig 16.4.2 – Stress blocks and forces on reinforced SFRC section.



(a) Single tensile reinforcement layer

SFRC also provides a benefit in shear resistance with and additional Vuf component added to the Vu calculation for linear elastic beam shear.

Design for serviceability

As a result of the potential for redistribution to be required to achieve an intended model the bridge code reverts to AS5100.5-2017 T8.6.1(A) for strut and tie serviceability stress limits.

Table 1 – AS5100.5-2017 Table 8.6.1(A)

TABLE 8.6.1(A)

MAXIMUM STEEL STRESS FOR TENSION OR FLEXURE

Nominal bar diameter (d_b) mm	Loading case specified in Item (c)(i)	Loading case specified in Item (c)(ii)
	Maximum steel stress (f_{scr}) MPa	
10	360	275
12	330	250
16	280	215
20	240	185
24	210	160
28	185	140
32	160	125
36	140	110
40	120	95

This differs from AS3600-2018 and can be more onerous than the service limits otherwise presented in AS5100.5-2017 Section 12 leading to two key considerations for pile cap design:

- Pile cap strut and tie design generally requires larger diameter bars to achieve the significant reinforcement amounts required. As noted in the table there is a reducing benefit in allowable service stress with increasing bar diameter. By comparison increasing a bar diameter from N36 to N40 increases the area of steel by 23% but allowing for service stress limits may only provide a 6% increase in service capacity. With consideration of handling and detailing requirements to anchor such large diameter bars they may not present the best design solution.
- Load case (c)(ii) as noted in Table 1 is applicable to permanent effects load combinations in exposure class B2, C, U. For foundations in aggressive ground conditions with a high proportion of dead load to live load this can be an issue.

Minimum reinforcement

Subject to checking governing ULS or SLS design actions minimum reinforcement for pile cap top and side faces should be provisioned in accordance with AS5100.5-2017 Clause 4.12 to prevent early-age thermal cracking that may occur in large, restrained members.

In the absence of specific guidance for non-flexural members or more sophisticated analysis Clause 9.4.3 for slabs and 11.7.2 for walls provide the best available direction for pile caps. Using an effective depth of 500mm for walls which is further increased for slabs the resulting minimum side face reinforcement is more onerous than standard minimum crack control requirements 500mm^2/m, typically 2000mm^2/m for side faces and for top faces may be increased to 2800mm^2/m for a 1500mm deep section or 3600mm^2/m for a 2500mm deep section.

Materials

Concrete

Pile caps generally use moderately high strength concrete mixes such as f'c = 50 Mpa for durability and cover requirements. Due to their significant depth and potential for elevated concrete temperatures to occur during placement and curing, pile cap mix designs should adopt minimum supplementary cementitious materials proportions as noted in AS5100.5-2017 Clause 4.11 to assist in controlling the risk of delayed ettringite formation. Specialist durability advice or mix designs may also be required where pile caps are located in aggressive ground conditions or groundwater.

Reinforcing Steel

AS5100.5-2017 allows strut and tie tension forces to be accommodated by reinforcing steel or post tensioning tendons. Typically for pile caps standard D500N reinforcement is used as there is limited benefit in the use of tendons. AS5100.5-2017 T3.2.1 has been updated to include D600N reinforcement, subject to availability this alternative high strength steel would provide direct savings at ULS but would be subject to the same SLS stress limitations. In the Australian market, Infrabuild SENSE 600^R provides available Grade 600 reinforcing bar alternative with custom bar sizes and a maximum bar diameter of 37mm.

Strut and tie design models

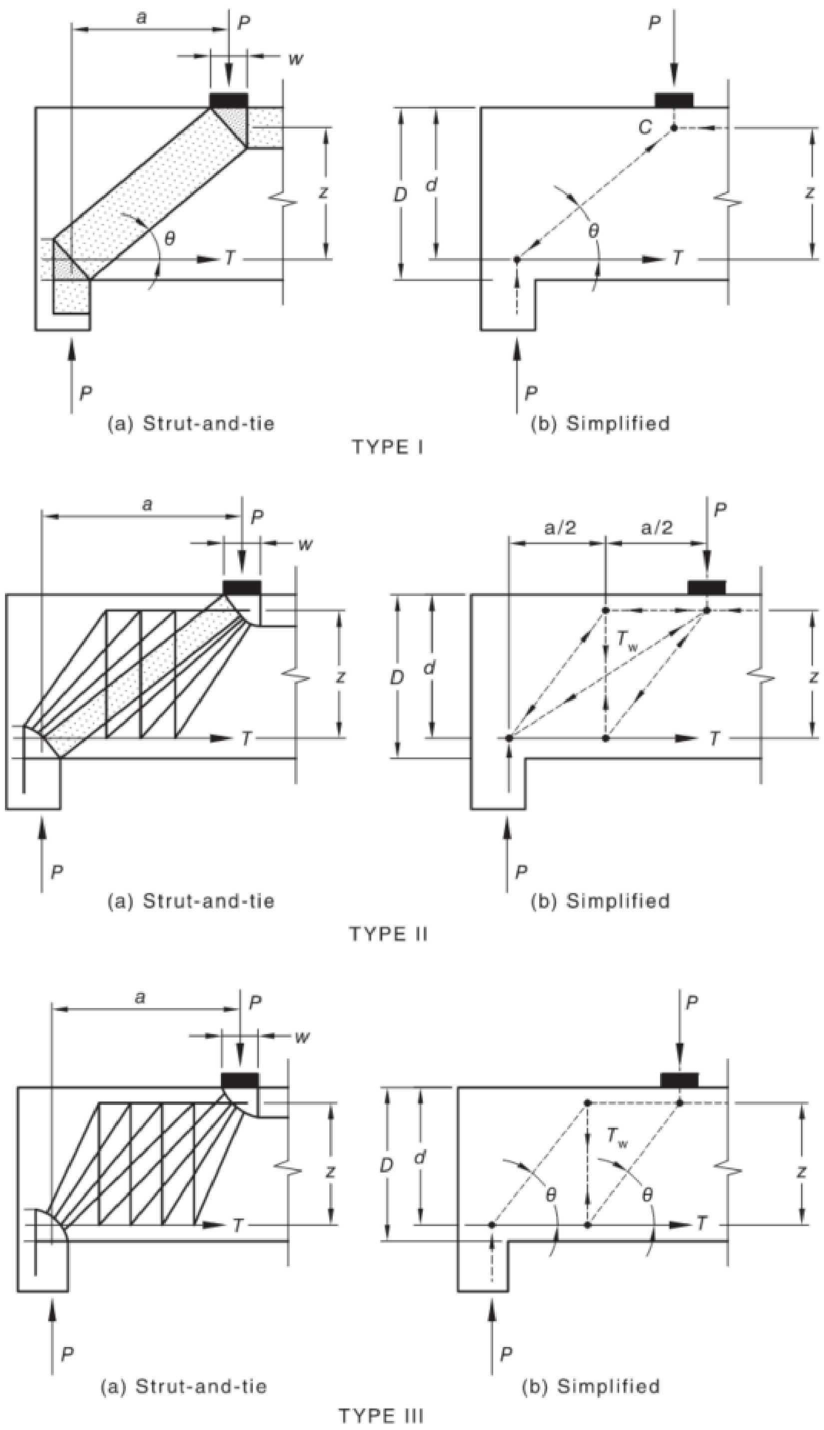
AS5100.5-2017 Section 12 provides guidance on strut and tie design via a series of three model types defined by the angle of the compression strut to the horizontal tension tie or the ratio of the lateral dimension a over the depth z.

- Type I – Load transfer directly to support by strut action permitted for a/z < 1 or θ < 45^O
- Type II – Load transfer to support by combination of primary direct strut and secondary indirect struts. Vertical ties are required to return the vertical component of the secondary strut force to the top of the member. Applicable to 1 < a/z < 1.73 or 45^O > θ > 30^O. The proportion of vertical force shared between the direct and indirect struts is taken to vary linearly with the change in angle between these limits.

- Type III – Load transfer by indirect struts and vertical ties only. Applicable to a/z > 1.73 or $\theta < 30^{\circ}$.

The Figure below demonstrates the differences between Model Types I, II and III with the simplified model shown on the right-hand side.

Figure 7 - AS5100.5-2017 Figure 12.3.2 – Strut and tie models and simplified design models



Understanding these models is essential to efficient strut and tie design whilst they can appear complex, they can be readily applied to most pile cap instances as discussed in the following section.

4. Practical strut and tie models for pile cap design

Whilst AS5100.5-2017 Section 12 defines the Design Models the detailed requirements for strut and tie design are set out in AS5100.5-2017 Section 7. When read in conjunction with Commentary from AS5100.5 Supp 1-2008 or AS3600-2018 Supp 1-2022 there is good guidance on general strut and tie design however the missing link remains the interpretation of this guidance to the pile cap scenario.

For the purposes of this demonstration the following arrangements are presented as two-dimensional sections. In practice asymmetrical pile caps would need consideration in each plane or could be expanded to three dimensional models for more complex layouts.

The intent in presenting and discussing these models is that the designer should be able to resolve most of the critical design actions by simple hand calculations and should not be reliant on spreadsheets with angles calculated to multiple decimal places or can be used to sanity check more complex structural models when required.

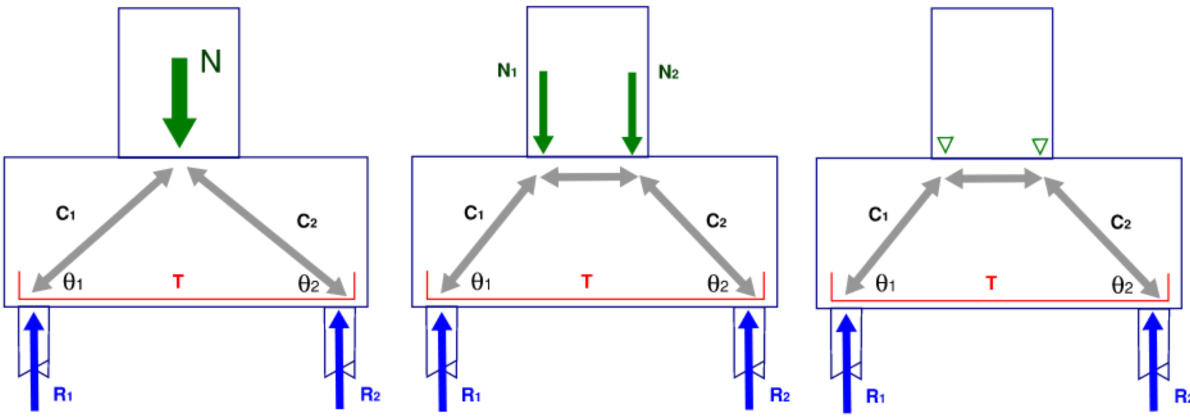
The models then allow the designer to understand what is happening at any location within the pile cap and to detail and allocate reinforcement accordingly to achieve an efficient and practical design solution that can be readily interrogated and adjusted if the initial solutions are impractical.

The sign convention for struts and applied loads and reactions follows the convention from Figure 6 which helps to demonstrate the direction of loads at nodes ie compression loads act towards the node, tension loads act away from the node.

Typical two pile cross section

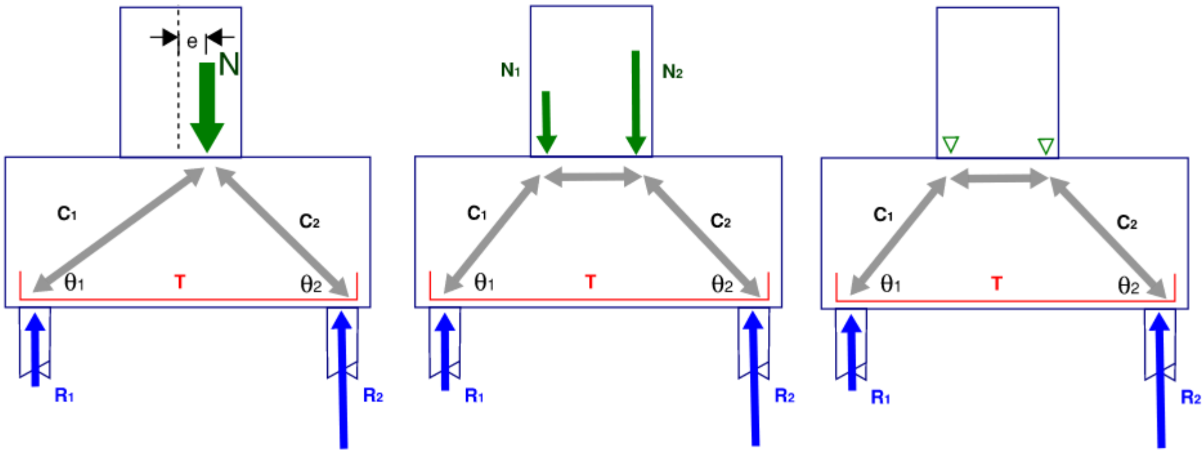
The first series of models below show a simple Type I model applied to a two-pile cross section. The models from left to right demonstrate how a column reaction can be resolved by a simple strut and tie triangulation to the supporting pile locations. In the first model the reaction is represented by a single centrally applied load, if the strut angle for the first model is less than 45° requiring a Type II or III model the designer may elect to idealise the column loads as two separate loads improving the strut angle to retain a Type I model. On the right-hand side the last model shows that the same model could be utilised if known pile reactions from a global geotechnical model were applied to a model with theoretical supports at the column location.

Figure 8 – Two pile cap with concentric vertical load only



Taking the same simple two pile cross section the series of models below demonstrate the resolution of design actions in a pile cap subject to axial load and bending moment represented as the eccentric application of the vertical load. With the load point shifted to one side this reduces the strut angle on one side but empasizes the benefit of the adjusted model with either two applied loads or theoretical supports to improve the strut angle and maintain a Type I model.

Figure 9 – Two pile cap with vertical load and bending moment (eccentric loading)

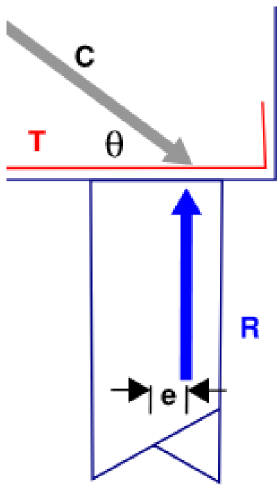


In each of the models above the individual struts and ties would need to be checked for strength and serviceability and sized and detailed accordingly. Regardless of the intended Type I model a highly loaded strut may require tie reinforcement if it needs to be increased in cross section to a bottle-shaped strut. For a typical pile arrangement with struts sized similar to the pile cross section this should not be an issue as the average stress in a pile cross section should be fairly modest even at ULS and should readily be accommodated by a prismatic strut when resolved at a 45° angle or greater.

Piles in Bending

In addition to bending in the column reaction there may also be bending in the piles themselves. This is readily resolved in the above models with consideration of a small eccentricity in the pile reaction. For example a 100mm eccentricity in a pile with an axial reaction of 2500kN ~ 250kNm in bending. A sensitivity assessment shows that variation in the horizontal “a” dimension of the “a/z” ratio of +/- 100mm can result in +/- 5-10% change in forces for typical pile cap depths and strut angles of 45-60°. Resolution of shear forces in piles can be resolved in a similar manner.

Figure 10 – Pile with bending moment (eccentric pile reaction)



Four pile and larger cross sections

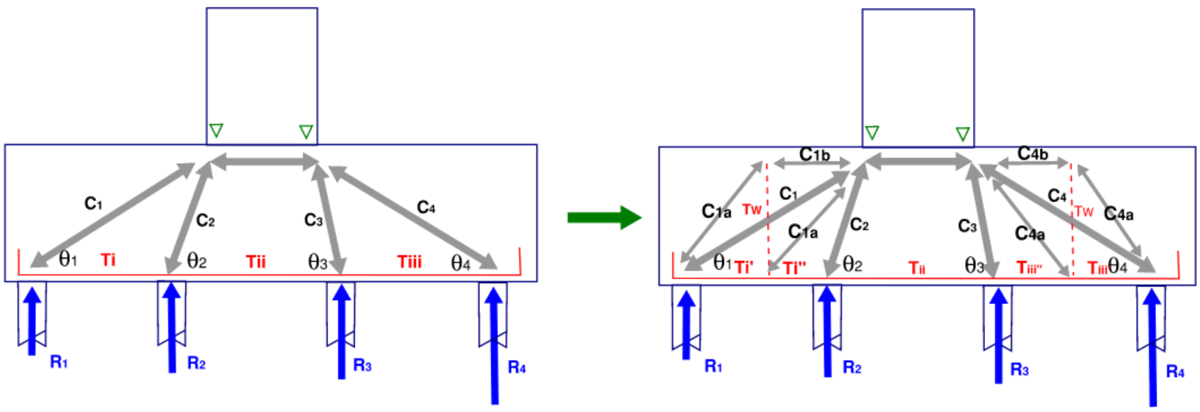
Taking these simple models further to a four-pile cross section the models below show additional considerations as the pile reaction location is moved further from the face of the column increasing the angle of the outer strut to move from a Type I to a Type II model.

With pile cap a/z ratios generally much less than 1 the angle of the outer strut can be maintained at greater than 45° in most cases for a Type I model.

In the event that a Type II model is required it should be understood that the force in the outer primary compression strut is progressively reduced and can never be more than in the Type I model, additionally using the free body boundary working from outer pile back towards the centre of the pile cap the vertical tie force can never be more than the outer pile reaction.

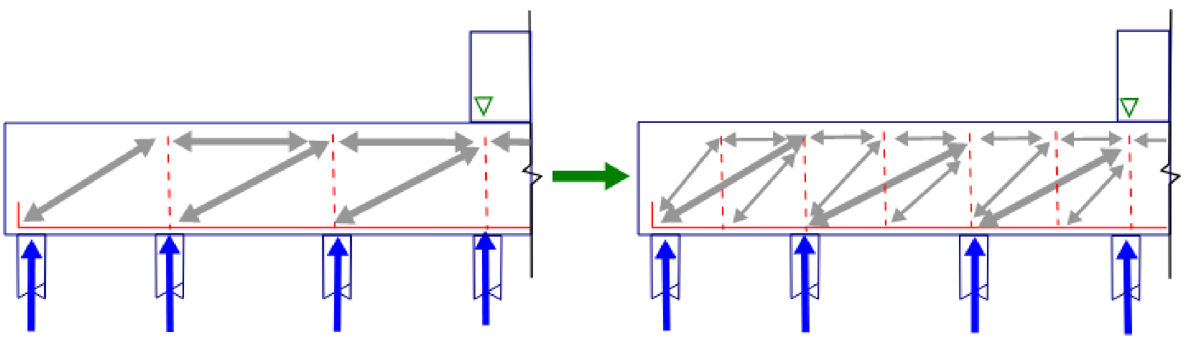
In the Type I model the bottom face tension force is a maximum at the cente and is reduced in the outer bays of the model Ti and Tiii < Tii. In the Type II model the maximum force is same but further reduced in the outer most bays again such that Ti’ < Ti” etc. This allows bottom face tensile reinforcement to be more efficiently detailed and terminated when it is no longer required. If reinforcement is provided in layers it should be possible to terminate a layer before reaching the outer most pile location to avoid congestion.

Figure 11 – Four pile cap Type I to Type II model



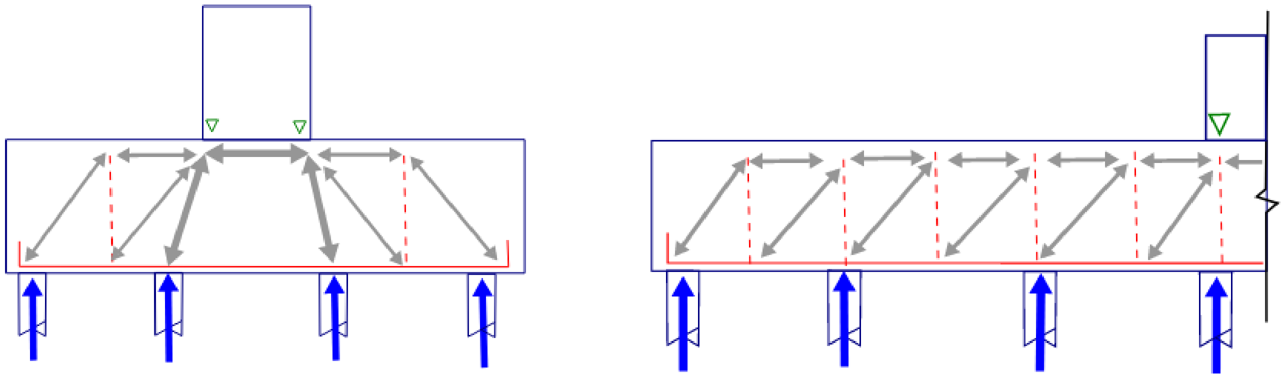
This methodology can be expanded further to a large pile cap with any number of bays. When dealing with larger pile caps the free body boundary demonstrates that the force in any diagonal compression strut or vertical tie needs to accommodate the sum of the pile reaction forces applied up to that point working back to the column support. In the Type II model this means that vertical tie forces and the volume of reinforcement required in the inner bays closer to the column will be greater than in the outer most bays away from the column.

Figure 12 – Large pile cap Type I to Type II model



In the above examples the Type II model using a combination of primary and secondary compression struts can become messy. The use of a Type III model can simplify this as shown below. Using these simplified arrangements in conjunction with the free body boundary the designer can readily check critical sections and detail pile caps in a much simpler manner.

Figure 13 – Type III model examples

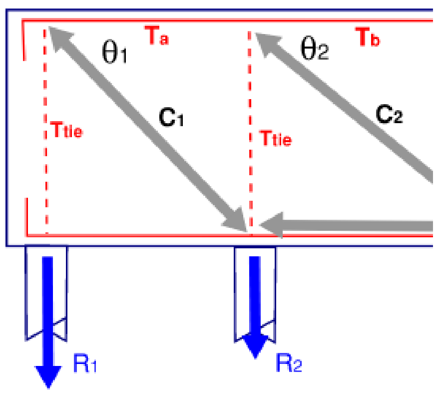


Piles in tension

In any of the above models it is possible that piles may be required to act in tension under certain load combinations. If this is the case the theoretical model is inverted with main tension reinforcement moving to the top face and compression on the bottom face of the pile cap. This will require tension ties to bring to pile force to the top of the pile cap for diagonal compression struts to work.

Pile tension forces are typically relatively small as compared to compression loads and the minimum reinforcement required for the top face of the pile cap can generally accommodate this actions. At tension pile locations it is not essential for the pile cap reinforcement itself to extend to the top face as this may be impractical (particularly for driven piles) as such a supplementary group of two or four ties around the pile will be sufficient for most cases.

Figure 14 – Pile in tension



5. Sustainability

AS5100.1-2017 recommends “consideration” of sustainability via whole of life impacts. Unfortunately, unless specified by the Relevant Authority for a project there are generally no specific mandatory requirements for Sustainability. Under different project delivery models there may be some performance incentive or KPI bonus attached to achieving an overall sustainability outcome or ISCA rating but in practice this may only be pursued where there is a parallel cost saving.

Notwithstanding the above the designer should feel empowered to understand and take ownership of their impact as much as they would do for functional performance, structural integrity or serviceability.

As discussed in the proceeding sections the designer has some influence on the impact of their overall design and thus it's carbon footprint. It should not be too onerous but in self-checking their design the engineer should do a sanity check on basic quantities and can then complete simple sensitivity checks to understand the efficiency of the overall design.

Carbon Footprint

The largest primary whole of life impact on sustainability arises from the embodied carbon emissions in the construction materials and their global warming potential. Based on published data typical emissions per tonne have been considered as follows:

Table 2 Carbon Emissions per material type

Material	Emission per tonne	Reference
Cast insitu 50 MPa concrete (30% SCM)	0.181 t (CO2-eq)/t	(ISC 2023)
Cast insitu 50 MPa 1-2% steel fibres	0.195 – 0.208 t (CO2-eq)/t	(ISC Materials
D500N Grade reinforcement	1.58 t (CO2-eq)/t	(Infrabuild 2022)
D600N Grade reinforcement	1.24 t (CO2-eq)/t	(Infrabuild 2022)

Steel Tonnage review of past projects

The review has considered a sample of eighteen pile caps across five past projects that were designed to AS5100-2004. Whilst pile caps can vary significantly from bridge to bridge due to their specific design loading requirements or other site constraints a comparison point for assessment arises in the steel reinforcement weight provided per cubic metre of concrete kg/m3.

This review identified tonnage varying significantly from as low as 120kg/m3 up to 300 kg/m^3 although the average tonnage was generally in the range 150kg/m^3 to 175kg/m^3.

Using the above values the resulting CO2 emissions quantities and contribution of steel per m3 concrete (density of 2400kg/m^3) can be derived demonstrating logically that with other variables such as pile layout and pile cap plan dimensions remaining equal, that less steel reinforcement = less CO2. To improve their design the engineer can interrogate their models and look to reduce reinforcement requirements at critical locations.

The overall reinforcement tonnage is made up of top face, bottom face, side faces and internal tie reinforcement with differing proportions from the sample reviewed it was found that the average distribution of reinforcement across these regions is as follows:

- Top face – 16%
- Bottom face – 54%
- Side faces – 20 %
- Ties – 10%

Using these ratios and the different overall tonnage rates it is then possible to test the theoretical sustainability benefit of design optimisation decisions. For comparison purposes the data is presented with a single variable change for each instance. For simplicity data is presented for 150, 200 and 250 kg/m^3 tonnage points. In each instance with the resulting impact is discussed in terms of adjusted Steel Tonnage kg/m^3 and the resulting CO2-eq t per m^3.

Steel Reinforcement Optimisation

From the sample of pile caps reviewed it was found that there was opportunity for optimisation through more efficient detailing of reinforcement. In almost all instances there was an opportunity for a 5-10% reduction in critical bottom face reinforcement and optimisation of the use and distribution of tie reinforcement. For comparison purposes the 10% tonnage reduction has been applied to this scenario with no changes to the concrete mix contribution.

Reduction in Pile Cap Depth

As discussed in the design commentary above the area of tensile steel required on the bottom face is directly influenced by pile cap depth in any of the models considered. If we test the benefit of reducing pile cap depth by 10%, in principle whilst this would reduce the concrete volume overall this would be countered by increasing the volume of tensile steel required on the bottom face, side face and tie reinforcement reduced, top face reinforcement would be unchanged. For comparison purposes the contribution of concrete per m^3 has been applied at 90% for this scenario reflecting the overall reduction in concrete volume.

Whilst there may be other constraints that limit pile cap depth adjustment there are also secondary sustainability impacts incurred by the pile cap construction, spoil removal and temporary works implications that warrant pile cap depth being minimised wherever practical.

D600N Reinforcement

Where ULS design governs the use of Grade D600N reinforcement in place of Grade D500N can provide direct saving to bottom face tensile reinforcement and any vertical tie reinforcement by ratio of the yield strengths 500/600 = 83.3%. Noting that SLS limitations can impact on the benefit of this substitution the suggestion would be to implement the saving by reducing the bar diameter as opposed to the number of bars to gain a similar increase in allowable SLS stress limits.

The added sustainability benefit for the current Infrabuild SENSE 600[R] offering is that the emissions per tonne are significantly reduced by the manufacturing process and energy consumption. No change has been considered in the area of steel required for top and side faces of pile cap as these are assumed to be controlled by SLS stress limits. For comparison purposes no change has been considered in the concrete mix for this scenario.

Steel Fibre Reinforced Concrete

The use of SFRC has been considered with significant benefit in reduction of tensile reinforcement. From spot checks of the impact on flexural capacity the % reduction depends on the level of reinforcement provided initially in that a lightly reinforced section could benefit by a 40-50% reduction in steel whereas a more heavily reinforced section might only see a 20-30% reduction.

Due to the significant depth of pile caps the shear contribution of fibre reinforcement can be substantial but is relative to Tie reinforcement provided by the designer for shear purposes which varies by the adopted model. To avoid overstating the benefit a 30% reduction has been considered for tie reinforcement allowing for minimum shear reinforcement provisions to be maintained for lightly reinforced pile caps. The benefits in reduction of top and side face reinforcement are unclear from the wording in the code and would require further investigation to be determined so have been excluded for this assessment.

These reductions are then significantly offset by an increase in carbon emissions generated by the mix and an increase in cost of concrete mix.

Table 3 CO2-eq Tonnage per m3 impact of design changes

D500N Base Steel Tonnage kg/m3	150	200	250
CO2-eq t per m3	0.67	0.75	0.83

D500N Base Steel Tonnage kg/m3	150	200	250
% Contribution of Steel to Concrete	35%	42%	48%
10% Reduction in Steel Tonnage through design optimisation			
Adjusted Steel Tonnage kg/m3	135	180	225
CO2-eq t per m3	0.65	0.72	0.79
% CO2-eq vs Base line	96%	96%	95%
10% Reduction in pile cap depth			
Adjusted Steel Tonnage kg/m3	154	205	256
CO2-eq t per m3	0.63	0.71	0.80
% CO2-eq vs Base line	94%	95%	96%
D600N Reinforcement			
Adjusted Steel Tonnage kg/m3	134	179	223
CO2-eq t per m3	0.60	0.66	0.71
% CO2-eq vs Base line	89%	87%	86%
SFRC 1.5% Mix			
Adjusted Steel Tonnage kg/m3	105	150	202
CO2-eq t per m3	0.66	0.73	0.81
% CO2-eq vs Base line	98%	97%	98%

From the summary table above, it is apparent that the most efficient means of optimising pile cap CO2-eq contribution would be using Grade D600N reinforcement. In each of the other scenarios considered the benefit of one adjustment was offset or negated to some extent. Of the options tabled SFRC would require further investigation to justify the potential increase in cost against minimal sustainability benefit whereas the options of steel optimisation, pile cap depth refinement and use of Grade D600N would all deliver parallel cost savings to construction and should be explored and implemented wherever possible and could readily be combined for maximum benefit.

6. Conclusion

Pile caps are an essential component of bridge design and construction that is often taken for granted. In the design of pile caps the designer is required to interpret the provisions of AS5100 that were generally written for other applications. The paper has explored these considerations and provided a consolidated commentary to assist designers in this undertaking.

With this broader understanding the designer has the opportunity to design more efficient solutions and should also understand the sustainability impact of their designs and the parallel opportunities to minimise material quantities and cost with the introduction Grade D600N reinforcement providing a significant opportunity to implement change within our industry.

7. References

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