Parametric Study for Behavior of the Punching Shear Capacity of Flat Slab Without Shear Reinforcement by The Selected Design Codes

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Abstract: The system of the flat slab is widely used for the construction of structures because it has economic and functional advantages. However, a flat slab has some disadvantages as punching shear problem occurs by shear force concentrations around the column, the transfer of moments between slabs and columns creates both twisting and high local moments, and make larger deflections. Punching shear failure is one of the failure mode types on the flat slabs that lead to brittle failure without the yielding of steel with small deflections when shear reinforcement is not added. Punching shear capacity of flat slabs without shear reinforcement could be calculated by some codes. Nowadays, the most widely used codes for punching shear are Eurocode2 2004, ACI 318, and Model code 2010. The strategy of all these design codes toward punching is mainly empirical and makes use of the findings of different experimental studies. Investigation on the code functions and on the impact of individual factors on the expected punching shear capacity has been carried out through a parametric study. The impact of concrete compressive strength, tensile reinforcement ratio and size of member were also observed.

Keywords: Flat Slab, Punching Shear, Compressive Strength of Concrete, And Tensile Reinforcement Ratio

1. Introduction

Several different kinds of construction structures make use of two-way concrete slabs. These slabs are basically divided into two groups on the basis of their support i.e., slabs supported with beams and slabs supported with columns without beams. Slabs without beams are divided into two types, i.e., flat slabs and flat plates (Wight and MacGregor 2009). Constructions of reinforced concrete flat slabs are commonly used because it has economic and functional advantages. To decide which type of slab should be used either flat slab or two-way concrete slabs supported on beams. In the case of buildings with small heights, flat slabs serve to be a good choice though they are influenced by intense deflections. Reduction in story height proves to be financially beneficial for the overall construction of the building since it influences the mechanical characteristics like piping and elevator shafts which become shorter when the height of the building is decreased. However, flat slabs have some disadvantages as punching shear problem occurs by shear forces concentration around the column, moments transfer between slabs and columns create both twisting and high local moments, and make larger deflections (Park and Gamble, 2000). Ruiz and Muttoni (2010) state that, punching shear failure is one of the failure mode types of the flat slab that occurs brittle failure without the yielding of steel with small deflections when not added shear reinforced. Therefore, it is formed a pyramid-shaped surface around the column, or a truncated cone as shown in figure 1.

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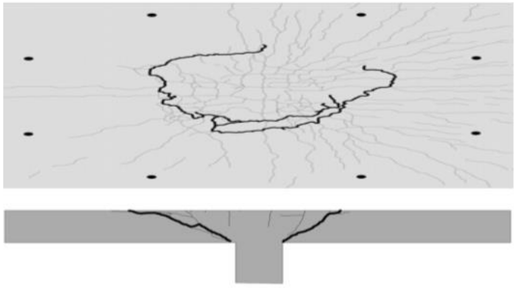


Figure 1: Typical punching shear failure (Guandalini, Burdet, and Muttoni, 2009)

A large number of researchers have studied punching shear; however, it still remains a complex issue and no method is available which can determine punching capacity of the connections between slab and column in the absence of shear reinforcement in an efficient way that can be utilized for design. A number of studies were aimed at studying the effect of different factors on punching shear capacity and different parameters when testing the specimens in laboratories such as low reinforcement ratios studied by (Guandalini, Burdet and Muttoni 2009; high strength of concrete has been studied by (Marzouk and Hussein, 1991; Inacio,M. at.el.2011); and lightweight concrete or normal weigh concrete has been studied analytically by (Theodorakopoulos and Swamy ,2002) and experimentally by (Youm, K. S. at el.2013). The purpose of this study is to investigate the behavior of the punching shear capacity of a flat slab without shear reinforcement by the selected design codes ACI 318-08, Euro-code2, and Model code 2010. According to each code explain which factors affect the punching shear of a flat slab. Every design code for punching shear resistance mostly follows an empirical approach which is based on the observation and study of experimental outcomes.

2. Design Codes

In the present times, the codes used most extensively for punching shear designs include Eurocode2-2004, ACI 318, and Model code 2010. There have not been any considerable variations in the ACI 318 requirements for punching shear in the past thirty years. On the other hand, Eurocode2, which was introduced in 1992, was upgraded in 2004 to incorporate the suggestions of Model Code 90. The Model Code 2010 based on more comprehensive and more physical models with respect to the requirement of punching shear design is based on the performance of the previous version Model Code 90. The punching shear resistance is determined by all the codes on a critical perimeter (u) that is obtained from the column’s face.

2.1 Eurocode2 2004

According to Eurocode2 2004, Punching shear capacity can result from a reaction or concentrated axial force acting on a comparatively small region that is named the loaded area A_{load} of a slab. A suitable confirmation model at the ultimate limit state for checking punching shear failure may affect from reaction or a concentrated load on a loaded area is shown in figure 2. The punching shear resistance must be checked on the loaded area at the edge of the column at the critical perimeter.

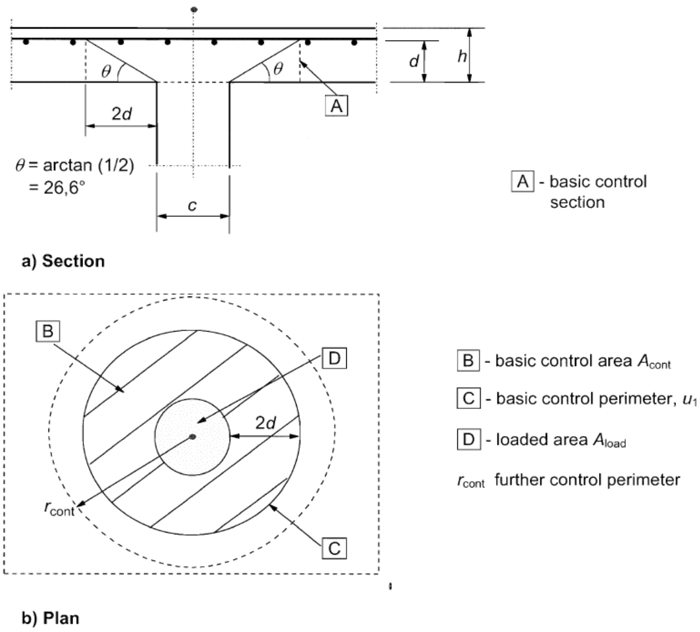


Figure 2: Verification model for punching shear at the ultimate limit state, (EN 1992-1-1:2004).

According to EC2-2004, the resistance of the punching shear of reinforced concrete slab depends on the size effect of the depth of the slab (k), the flexural reinforcement ratio (ρ_l), and the compressive concrete strength (f_{ck}) parameters. The recommended equation for the design punching shear resistance of slab-column connection without shear reinforcement (MPa) is defined as:

$$V_{Rd,c} = \left({}^{0.18}/_{\gamma_c}\right) \cdot k \cdot (100 \cdot \rho \cdot f_{ck})^{\frac{1}{3}} \cdot u_1 \cdot d \geq v_{min} \cdot u_1 \cdot d$$

Where:

f_{ck} is the compressive concrete strength in MPa?

u_1 is a critical control perimeter take (2d) of the edge of the column,?

d is the effective depth of the slab,?

k is a factor accounting for the size effect of the depth of slab that is defined as:?

$$k = 1 + \sqrt{\left(\frac{200}{d}\right)} \leq 2.0 \qquad d\ (mm)$$

ρ is the flexural reinforcement ratio in the slab,?

$$\rho = \sqrt{\rho_{ly} \cdot \rho_{lz}} \leq 0.02$$

ρ_{ly} is reinforcement ratio in the y direction,

ρ_{lz} is reinforcement ratio in the z direction, and

$$v_{min} = 0.035 \cdot k^{\frac{3}{2}} \cdot f_c^{\frac{1}{2}}$$

The basic critical control perimeter $u_1$according to EC2-2004, it is normally make at a distance 2d from the loaded area and should be possible a minimum length as shown in figure 3.

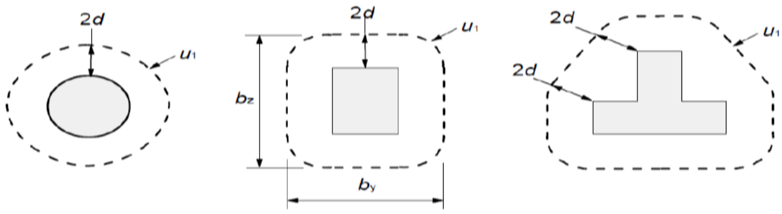


Figure 3: Typical basic control perimeters around loaded areas, (EN 1992-1-1:2004).

The critical perimeters (u_1) at a less distance than 2d should be considered if the concentrated loading is opposed by a high pressure. In addition, when a loaded area positioned close an edge or a corner should be take the critical perimeter as shown in figure 4.

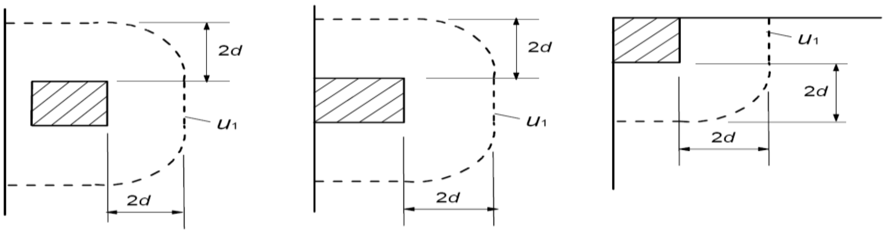


Figure 4: Basic control perimeters for loaded areas close to or at edge or corner (EN 1992-1-1:2004).

2.2 ACI 318-08

The resistance of the punching shear of American code (ACI 318-08), widely depends on the control perimeter length, the concrete strength, and the column geometry of the slab, however, the common dependence of the punching shear capacity on the flexural reinforcement ratio is not taken into account. According to ACI code, for the slab - column connection, punching shear strength without shear reinforcement is calculated by the three equations as follows and takes the smallest value of shear concrete capacity.

$$V_c = \min \left\{ \begin{array}{c} 0.33 \cdot \lambda \cdot \sqrt{f_{c'}} \cdot b_o.d \\ 0.17(1 + \frac{2}{\beta_c})\lambda \cdot \sqrt{f_{c'}} \cdot b_o.d \\ 0.83(0.2 + \frac{\alpha_s d}{b_o})\lambda \cdot \sqrt{f_{c'}} \cdot b_o.d \end{array} \right\} \quad \text{(In SI units, N, mm)}$$

Where:

f_{ck} is the compressive concrete cylinder strength?

b_o is the critical perimeter,

d is the effective depth of the slab?

β_c is the ratio of the long side of the column to short side?

λ is the reduction factor, and?

α_s is equal to 40,30, and 20 for interior, edge, and corner column respectively.

Shear in the flat slab, the critical section is exposed to bending and follows the perimeter at the boundary of the loaded region. The shear stress placed on this part at factored loads depends on the function ($\sqrt{f_{c'}}$) and the ratio of the column's side length to the effective depth of the slab. A simpler equation of the design can be obtained by assuming the presence of a pseudo-critical section at a distance d/2 from the edges of the concentrated load. When this happens, almost independent the shear strength on the ratio of column size to depth of the slab.

For general shape loaded area, the critical perimeter sections $b_o$according to ACI 318-08 should be located at (0.5d) to:

- Corners or edges of columns, reaction areas, or concentrated loads; and
- Variations in slab depths such as drop panels, edges of capitals, or shear caps.
- According to the ASCE-ACI Committee 426 report, which is illustrated critical perimeter sections for different shape column as shown in figure 5 (Park & Gamble, 2000).

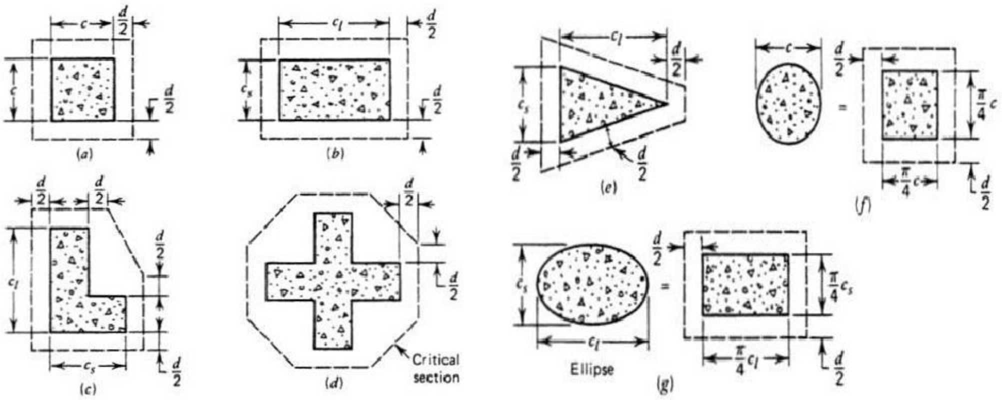


Figure 5: Section of Critical control perimeter for shear in slabs (Park & Gamble, 2000).

Due to ultimate loads, the shear stress in slabs exposed to bending is restricted to ($0.33\lambda\sqrt{f_{c'}}$) limited for square columns. However, ($0.33\lambda\sqrt{f_{c'}}$) has an un-conservative value when the ratio β of the dimensions of the short and long sides of a rectangular column or loaded region is more than 2.0. In these scenarios, there is variation in the actual sheer stress on the critical part at punching shear failure, from the highest value of around ($0.33\lambda\sqrt{f_{c'}}$) at the loaded area or the edges of the column, to the lowest value of ($0.66\lambda\sqrt{f_{c'}}$), or lower at the long sides between the two boundary sections. When the ratio bo/d increases, the value V_c decreases.

Away from the rectangular shape, the value of β for other shapes is considered to be the ratio of the largest overall perpendicular dimension of the effective loaded region to the greatest overall perpendicular dimension of the effective loaded region. The effective loaded region refers to that region which fully surrounds the real loaded region which has the minimum perimeter. This is demonstrated in Figure 6 for an L-shaped reaction region.

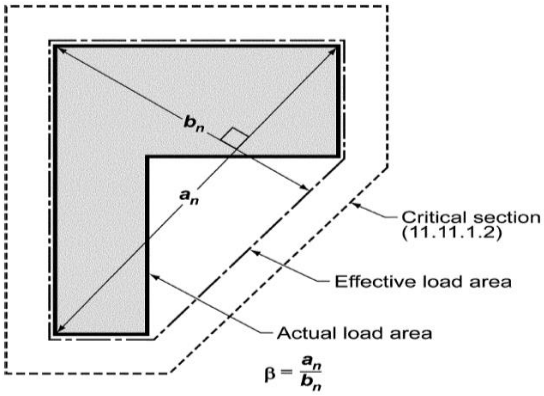


Figure 6: Value of β for a nonrectangular loaded area (ACI 318-08).

2.3 Model Code (MC 2010)

Established the Model Code 2010 as a constant basis that is founded on physical models, which is using for the design of beams as well as (one-way and two-way) slabs. Punching takes place as a result of concentrated load put on a comparatively small region of the slab. The punching design shear force of the slab is determined by using the total design shear forces that are applied on a basic control perimeter (b1). Punching shear failures in flat slabs usually forms around supported regions (columns, walls, capitals).

The Critical Shear Crack Theory (CSCT) is the basis of the Model Code (MC 2010). The slab rotation determines the punching strength, and this rotation occurs due to the applied load and the slab stiffness determined by the flexural strength. The MC 2010 also asserts that there are various approximation levels which allow for a rapid pre-dimensioning. In this study, there is used Level 1 approximation to compare previous test results. The punching strength of the slabs without shear reinforced can be determined as follows:

$$V_{R,C} = k_{\psi}/\gamma_c \cdot \sqrt{f_c} \cdot b_o \cdot d_v$$

Where:

f_c is the compressive concrete cylinder strength in (MPa),?

b_1 is a shear-resisting control perimeter,?

d_v is the shear-resisting effective depth of the slab,?

k_{ψ} is the parameter factor depends on the slab deformations (rotations) and follow from:

$$k_{\psi} = \frac{1}{1.5 + 0.9 \cdot k_{dg} \cdot \psi \cdot d} \leq 0.6$$

d is flexural effective depth of the slab in (mm),

It has been shown that the resistance of punching shear is affected by the aggregate maximum size (dg). When concrete whose maximum size of aggregate is less than dg = 16 mm is employed, then (k_{dg}) value in Equation (7.3-63) is obtained as:

$$k_{dg} = \frac{32}{16 + d_g} \geq 0.75$$

Where (d_g) is measured in (mm).

When size of aggregate is greater than 16 mm, may also be used the (k_{dg}) equation. Lightweight and high strength concrete may cause the aggregate particles to break which would lead to decreased contribution of aggregate interlock. In this scenario, the (d_g) value should be considered to be zero. Providing that the maximum aggregate size particles, the value (d_g), is not below 16 mm, can be considered ($k_{dg} = 1.0$). The parameter (ψ) refers to the rotation of the slab around the supported area, this is illustrated in figure 7.

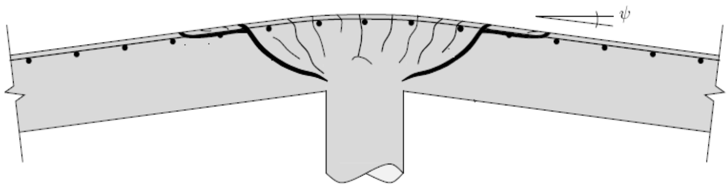


Figure 7: Rotation (**ψ**) of a slab (Model Code 2010)

To calculate rotations (ψ) around the supported area of flat slabs according to approximation level 1, for a normal flat slab that has been developed in line with the elastic evaluation without substantial redistribution of inner forces, the rotation at failure can be safely calculated as follows:

$$\psi = 1.5 \cdot \frac{r_s}{d} \frac{f_{yd}}{E_S}$$

Where r_s indicates the situation where the radial bending moment is become. Can be approximated the values of r_s can be taken as $0.22L_x$ for x-direction or $0.22L_y$ for y-direction. In approximation level I, Where the ratio of the spans ($\frac{L_x}{L_y}$) for regular flat slabs is between 0.5 and 2.0, the (ψ) equation considered for the maximum value of r_s .

The shear-resisting effective depth of the slab (d_v) is the distance from the supported area to the centroid of the reinforcement layers as shown in figure 8.

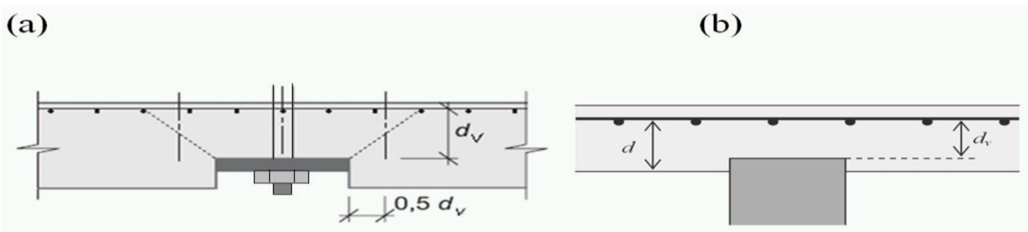


Figure 8: Effective depth of the slab considering support penetration (dv) and effective depth for bending calculations (d) (Model Code 2010)

Flat slabs have a design shear force that is equivalent to the value of the reaction of the support generated through actions that have been put on within the basic control perimeter (like pressure of soil at footings, gravity loads and deviation pressure of pre-stressing cables). The basic control perimeter b_1 according to Model Code 2010 should be located at a distance $0.5d_v$ to the supported area as shown in figures 9 and 10 and must be the minimum possible length (Figure 9c). There is limited the length of the control perimeter by the slab edges (Figure 9d).

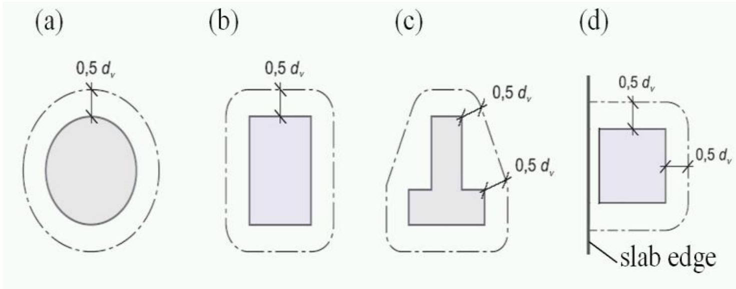


Figure 9: Basic control perimeters around supported areas(Model Code 2010)

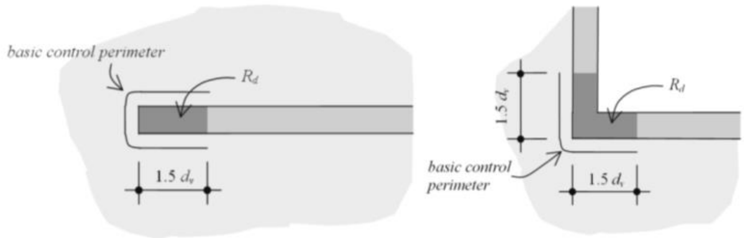


Figure 10: Basic control perimeter around walls (Model Code 2010)

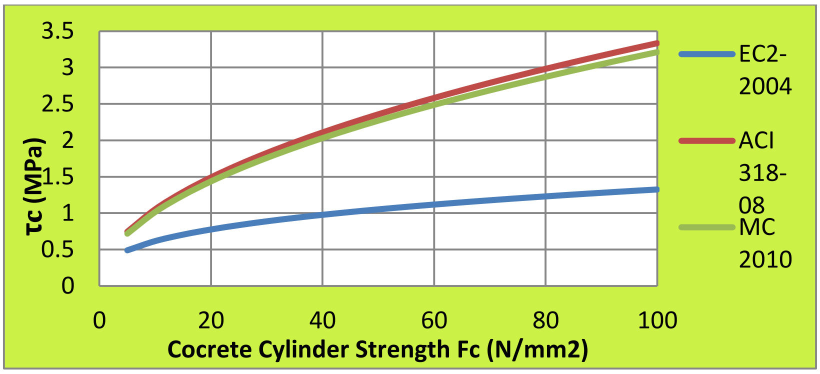
3. Comparative Code Examination

The differences between three codes (EC2 2004, ACI 318, and Model Code 2010) can be understood through the use of graphs which also explain how these variations can indicate the estimated values. The impact of the control perimeters on the outcomes needs to be understood. When contrasting the outcomes that are presented as forces, the variance between control perimeters of every code is high which means that the impact of some parameters such as (f_c) might be unclear as it is going to be included in the considerably greater effect of the control perimeter. It needs to be stated here that design codes for calculating accurate punching shear resistance values maintain a balance between the control perimeter and the shear stress.

3.1 The Concrete Cylinder Strength (f_{ck}) Effect To The Punching Shear Stress (τ_c)

According to the codes ACI 318-08, Model Code 2010 and EC2 2004, the punching shear stress of the element that is ($\tau_c = V_c/u.d$) is determined by using concrete cylinder strength. The factor that has an impact on majority of the outcomes of the design equations is defined by concrete strength. In addition, the punching shear stress is affected to a large extent by flexural reinforcement ratio as the increase in reinforcement ratio causes the punching shear stress to increase as well. There are two different ratios used for the calculation of shear stress as shown below:

3.1.1 When Low Reinforcement Ratio Is Used, $\rho = 0.5\ \%$

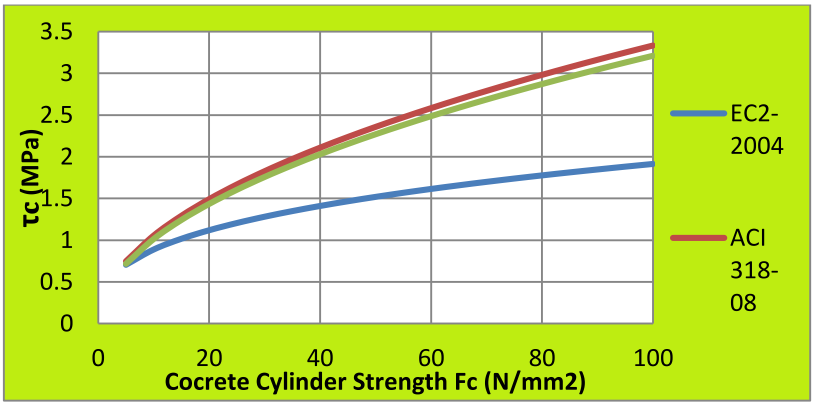


Graph 1: Punching shear stress without shear reinforcement versus concrete strength

$$(\rho_l = 0.5\%\ ,\ f_y = 500MPa\ ,\ d_g = 16mm\ ,\ {}^{C}/_{d} = 1,\ d = 200mm\ ,\ L = 2500mm)$$

As can be seen from graph 1, the impact of increasing concrete strength on the punching shear stress with low reinforcement ratio is demonstrated. The shear stress of the three codes will increase whenever the concrete cylinder strength increases. However, the EC2-2004 is slightly increased when concrete strength increases, also the shear stress according to ACI 318-08 and Model Code 2010 becomes higher than the EC2-2004 when the concrete strength increases. Accordingly, the punching shear stress can be estimated more appropriately using the assumption of cubic root of concrete strength of EC2 2004 instead of the square root of the concrete strength presented in ACI 318-08 and MC 2010. In addition, as the critical section u_1 according to EC2-2004 at a distance of (2d) from the loaded area is bigger than (0.5d) for both cdes ACI 318-08 and MC 2010, so the shear stress in the flat slab according to EC2 -2004 becomes lower than shear stress by other two codes which have lower critical section. Hence, when the critical section of the loaded area is bigger, the shear stress becomes lower.

3.1.2 When High Reinforcement Ratio Is Used, $\rho = 1.5\ \%$



Graph 2: Punching shear stress without shear reinforcement versus concrete strength

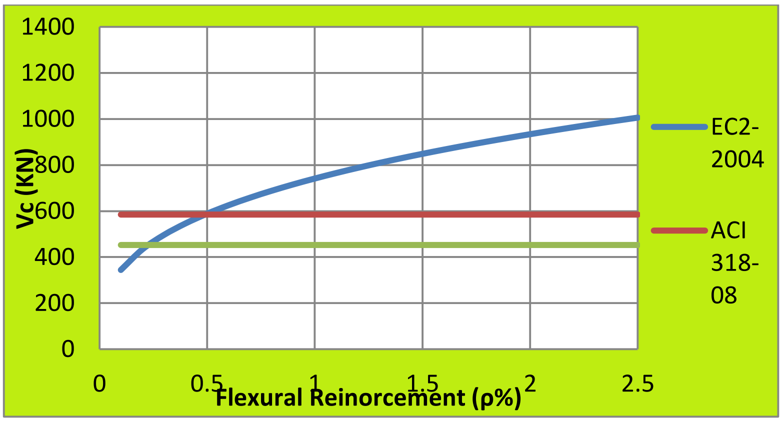
$$(\rho_l = 1.5\%\ ,\ f_y = 500MPa\ ,\ d_g = 16mm\ ,\ {}^{C}/_{d} = 1,\ d = 200mm\ ,\ L = 2500mm)$$

Graph 2 illustrates that when increasing flexural steel ratios, the shear stress in the EC2 increases yet it is still lower than both other codes. In contrast, the ACI 318 and Model Code 2010 still have larger punching strength values and are not viable to change when the flexural reinforcement ratio increases, because the shear stress equation for both codes is not considered for the flexural reinforced ratio.

3.2 The Flexural Reinforcement (ρ%) Effect To The Punching Shear Strength ($V_{R,C}$)

The calculation of the punching shear strength by EC2-2004 is affected by the flexural reinforcement ratio. However, the steel ratio is not considered on the punching shear equation of the ACI 318-08 and MC 2010 codes, so these two codes are not affecting the flexural reinforcement. There are two types of the compressive strength used to show how increasing the flexural reinforcement affects the punching shear strength as shown in graphs (3 and 4) below:

3.2.1 Normal Concrete Compressive Strength (f_{ck} = 30MPa) Used



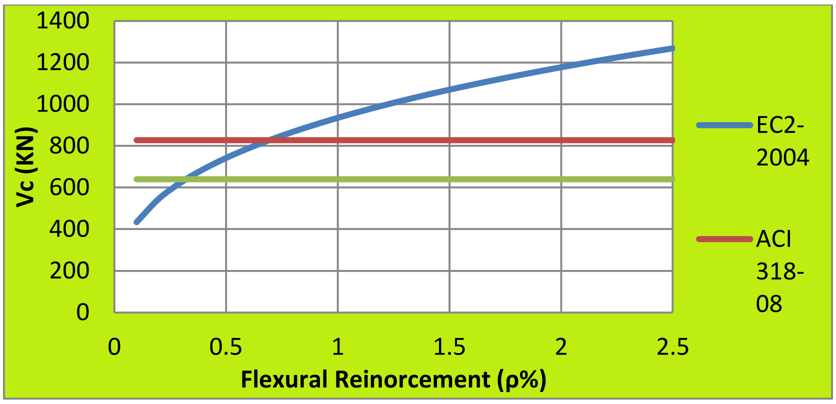
Graph 3: punching shear strength without shear reinforcement versus flexural reinforcement ratio

$$(f_{ck} = 30\ \mathrm{MPa}\ f_y = 500\mathrm{MPa}\ \ d_g = 16\mathrm{mm}\ \ {}^{C}/_{d} = 1\ d = 200\mathrm{mm}\ \ L = 2500\mathrm{mm})$$

As graph 3 presents, using the codes for shear strength critical perimeters should be considered and it is assumed that for normal strength concrete that has increasing flexural reinforcement ratio, EC2-2004 normally gives higher punching resistance values compared to the ACI 318-08 and MC 2010 when flexural reinforcement ratio is up to ($\rho = 0.5\%$). Therefore, the trend of punching shear resistance of ACI 318-14 and MC 2010 is still showing a straight line. The reason is because the flexural reinforcement ratio is not considered on the equations of ACI 318-08 and MC 2010 for punching shear resistance, so these two codes have no effect on the flexural reinforcement.

To sum up, the lower the reinforcement ratios, the more reductions in punching shear strength. In contrary, for higher reinforcement ratios, it is going to give conservative values as illustrated in the EC2-2004 outcomes. in addition to this is due to the flexural reinforcement ratio is not considered on the punching shear strength equation of both codes, so when assuming zero reinforcement ratios, the tension reinforcement term in ACI 318-08 and MC 2010 not allowing zero values for the resistance of punching shear strength. EC2-2004 equation, nevertheless, calculates zero punching shear resistance when flexural reinforcement is not given, and this is not logical, even without flexural steel ratio, can resist punching shear capacity through the compression zone of the concrete.

3.2.2 High Concrete Compressive Strength ($f_{ck} = 60MPa$)



Graph 4: punching shear strength without shear reinforcement versus flexural reinforcement ratio

$$(f_{ck} = 60 \text{ MPa } f_y = 500\text{MPa } \ d_g = 16\text{mm } \ {}^{C}\!/_{d} = 1 \text{ d} = 200\text{mm } \ L = 2500\text{mm})$$

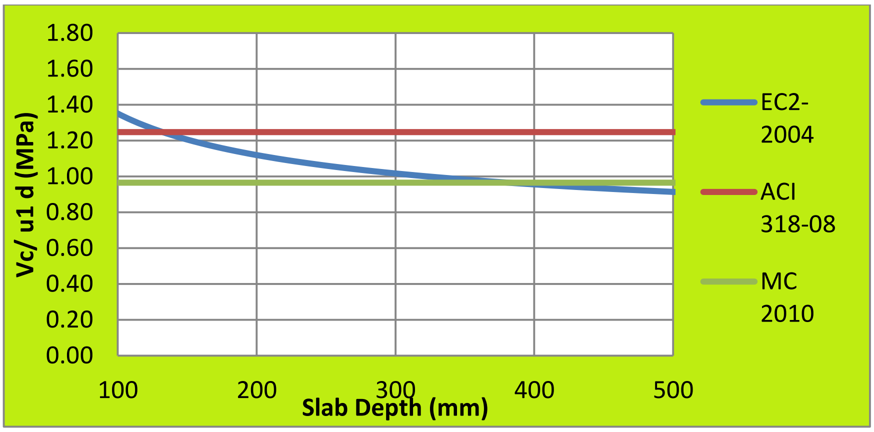
Graph 4 shows that the EC2-2004 estimates the highest values at high concrete compressive strength for increases steel ratio. For reinforcement ratio up to ($\rho = 0.7\%$), the Eurocode2-2004 calculates higher values than the value from both codes Model Code 2010 and ACI 318-08. Therefore, the punching shear resistance of ACI 318 and MC 2010 is still a straight-line graph.

3.3 Size Effect to The Punching Shear

The three codes have other different parameters that affect the punching shear strength of the slab. To show how effective depth of the slab, the parameter factor K and the slab length are affected by each code, these are described as follows:

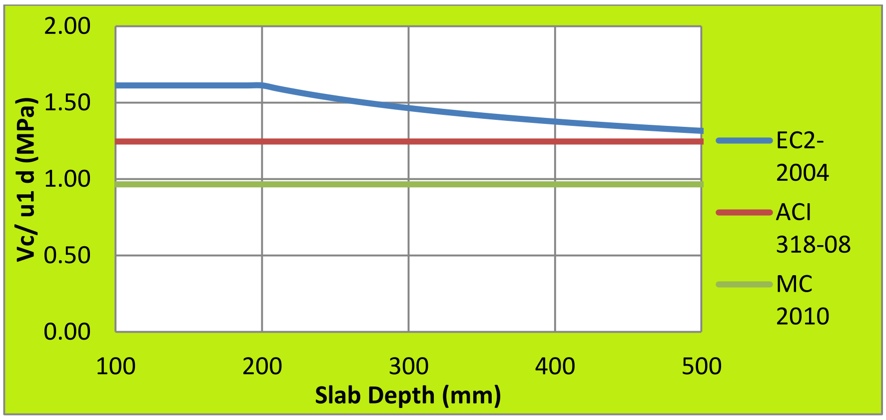
3.3.1 Effective Depth Of The Slab (d)

The normalized shear stress is plotted against different effective depths of the slab in which it shows how codes react to increasing slab depth with increasing flexural reinforcement ratio in the following graphs 5 and 6:



Graph 5: Normalised shear stress versus different slab depth with ($\rho_l = 0.5\%$)

$(\rho_l = 0.5\%\ f_{ck} = 60\text{ MPa } f_y = 500\text{MPa } d_g = 16\text{mm } {}^{C}/_{d} = 1\ L = 2500\text{mm})$



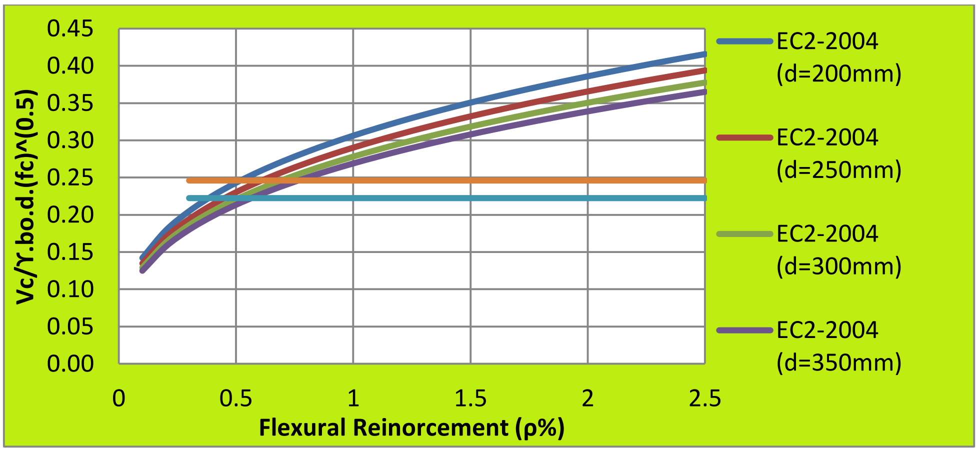
Graph 6: Normalised shear stress versus different slab depth with $(\boldsymbol{\rho_l = 1.5}\%)$

$(\rho_l = 1.5\%\ f_{ck} = 60\text{ MPa } f_y = 500\text{MPa } d_g = 16\text{mm } {}^{C}/_{d} = 1\ L = 2500\text{mm})$

Graphs 5 and 6 show that the ACI 318 and MC 2010 normalized shear stress are not affected by the change in dimensions of the column and slab depth as ACI 318 equation does not include any element that explains the impact of the size. These two graphs also allow one to deduce that MC 2010 turns less conservative at thick slabs in comparison to the other two codes. Furthermore, when it is assumed that the decrease in reinforcement ratio is going to further lower than the curve of the EC2 2004, then MC 2010 is going to become even less conservative at thick slabs with less reinforcement ratios.

3.3.2 The Parameter Factor (k)

This factor (k) is considered in the EC2 2004 and MC 2010 into the equations as a different parameter that affected the punching shear strength of the slab. To illustrate the impact of (k) factor that explains the size effect; there is a curve that relates to four different column and slab depths as sketched in graph 6. The ratio of the column side size to effective depth stayed constant at $1(C/d = 1)$.

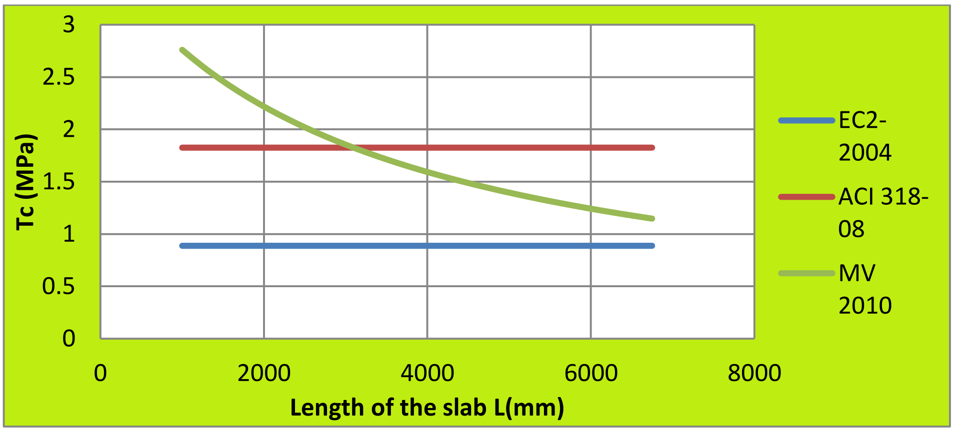


Graph 7: allowable punching shear strength versus flexural reinforcement ratio

$(f_{ck} = 60\text{ MPa } f_y = 500\text{MPa } d_g = 16\text{mm } {}^{C}/_{d} = 1\ L = 2500\text{mm})$

The normalized punching shear strength along with the flexural reinforcement ratio for the ACI 318, MC 2010 and EC2 2004 is shown in Graph 7. A known fact is that the EC2-2004 when the sizes of the column and slab depth are increased; there is a decrease in the nominal shear strength (Guandalini, Burdet, and Muttoni, 2009). In contrast, ACI 318-08 and MC 2010 have not affected by (k) factor during the changing of the size of the column and effective depth of the slab. Therefore, the graph of punching shear resistance for these two codes is a straight line.

3.3.3 Length of the Slab (L)



Graph 8: Punching shear stress without shear reinforcement versus length of the slab

$(f_{ck} = 30\text{ MPa } f_y = 500\text{MPa } \rho = 0.5\% \ d_g = 16\text{mm } {}^{C}/_{d} = 1 \ d = 200\text{mm})$

As can be seen from graph 8, the MC 2010 outcomes for normal strength concrete when increasing the length of slab, decreases the value of MC 2010 however it normally gives higher shear stress values compared to EC2-2004 and ACI 318-08 when slab length is up to 3m. In addition, the graph of shear stress of ACI 318-08 and EC2-2004 is still a straight line. Because of the slab length is not considered on the equations of ACI 318 and EC2-2004 for punching shear resistance, so these two codes will not effect on the slab length. However, the parameter factor (k_{ψ}) depends on the deformation and rotation ψ of the slab around the supported area included in the MC 2010 to calculate punching shear according to approximation level 1. So, increasing length of slab improves the rotation factor and decreases the parameter factor.

4. Conclusion

Different design codes move towards the phenomenon on the basis of experiential studies. Punching shear strength is a function of flexural steel ratio and the concrete compressive strength as these parameters have been defined and included by the EC2-2004. On the other hand, the ACI 318 and MC 2010 employ the concrete compressive strength for its predictions. There are two main reasons why establishing parametric studies surrounded by the punching shear equations of design codes is a complicated process:

- As every parameter have a distinct influence that is assigned by the code, for instance, the EC2 2004 employs the cubic root of the concrete compressive strength (f_c), and ACI 318 MC 2010 employ the square root of the concrete compressive strength (f_c).
- A different control perimeter is used for the loaded area by each individual design code for the purpose of distributing the shear stresses as (0.5d) is used as a control perimeter by both codes

ACI 318 and MC 2010, whereas (2d) is used as control perimeter for EC2 2004. Therefore, the area of the critical section using the EC2-2004 is more than that found by other codes. As a result, the shear stress using both ACI 318-08 and MC 2010 codes is more than that by EC2-2004.

As described by the parametric study, there is increase in the outcomes of the ACI 318 and MC 2010 in connection with the increasing concrete compressive strength. However, no change is made to the tension reinforcement ratio and the effective depth of the slab. Comparing EC2-2004 with the ACI 318 and MC 2010, the EC2-2004 outcomes have a slowest increase and estimate the lowest values with regards to the increasing concrete compressive strength. With regards to increasing reinforcement ratio, EC2 2004 estimates the greatest punching shear capacity. But adaptation to changing flexural reinforcement ratios in case of the ACI 318 and MC 2010 were found to be absent and the size effect is not explained by the ACI 318. Conversely, factor k is included by the EC2 2004 and MC 2010, and when increasing sizes of slab and column reduces the nominal shear stress in EC2-2004 and still it is constant in MC 2010.

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