FINITE ELEMENT ANALYSIS OF R.C. INTERIOR SLAB-COLUMN JOINTS WITH RIGID REINFORCEMENT

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ABSTRACT

Finite element modeling of reinforced concrete interior column-slab joints with and without shear reinforcement, as well as analysis of shear stress fields in the middle layer of the concrete plate were performed using STAAD.ProV8i. For proper calibration between the models, a definite finite solid model without shear reinforcement is analyzed and considered a reference sample. This study examines a new form of rigid shear reinforcement that eliminates steel congestion at joints and provides a simple approach to determining the rigid shear reinforcement arm.

Keywords: Punching shear, slab-column joint, rigid shear reinforcement, finite element modeling, staad pro

Cite this Article: Eid I.Al-Sahawneh, Finite Element Analysis of R.C. Interior Slab-Column Joints with Rigid Reinforcement, *International Journal of Civil Engineering and Technology*, 11(8), 2020, pp. 14-23.

<https://iaeme.com/Home/issue/IJCET?Volume=11&Issue=8>

1. INTRODUCTION

The first attempt to incorporate various types of transverse reinforcement into flat plates was performed by Graf and Wheeler by using bent-up bars. It was observed that bent-up bars increased the punching shear resistance. Islam and the Park [1], Hannah et al. [2], Pillai et al. [3] and Robertson et al. [4] used vertical stirrups with additional horizontal bars extending outward from the support in two perpendicular directions as another type of punch shear increase as in Figure 1. The ACI 318-19M section 22.6.7.1 [5] requires, when using vertical stirrups for increasing punching shear, that the slab effective depth d to be at least 15cm, but not less than 16 times the diameter of the shear reinforcement. Regan and Samadian [6], Oliveira et al. [7] noted that only inclined stirrups, with a 60° inclination, have shown to be efficient in increasing the punching resistance.

Figure 1 Flat Closed stirrups shear reinforcement for slabs at slab-column connections.

The most popular punching shear reinforcement in the civil construction industry today, is headed shear studs welded to a steel plate as in Figure 2. Shear studs and stud rails were used in slabs and tested by Dilger and Ghali [8], Elgabry and Ghali [9], Megally and Ghali [10].

Figure 2 Shear studs (taken from Polak et al., [11]).

El-Salakawy et al. [12] presented a new type of shear reinforcement installed later in flat plates, called shear bolts as in Figure 3. Adetifa and Polak [13] observed that shear bolts prevent the propagation of the shear cracks in strengthened slabs and improve the performance of the slabs with openings.

Figure 3 Shear bolt and its installation (Polak and Adetifa, [13]).

Corley and Hawkins [14] and Dilger and Gali [15] tested a new type of shear reinforcement called shearheads, shown in Figure 4. Shearheads are cross-shaped elements constructed by welding structural-steel members, such as channels, or I-beam, into a rigid unit. Shearheads are usually used to increase the effective perimeter b_o of the critical section through which shear can be transferred into an interior column from the surrounding slab.

Figure 4 Shearheads shear reinforcement

2. REVIEW OF ACI318-19M DESIGN PROVISIONS FOR PUNCHING SHEAR

The research done by Moe in 1961 led to the ACI design provisions for punching shear. The ACI Committee 326 adopted a pseudo-critical section located at a distance $d/2$ from the face of the column (or the perimeter of the loaded area), as in Figure 5. In this critical region, for slabs of uniform thickness, the shear strength is checked on one section. For slabs with variable thickness, it is required to check the shear at many sections.

Figure 5 Perimeter Critical section according to ACI for typical flat plate (without capitals or drop panels)

The ACI Code, Section 22.6.1.3, requires that the factored shear force V_f on the critical section should be less than $\phi(V_c + V_s)$ where V_c is the shear resistance provided by concrete, V_s is the shear resistance provided by shear reinforcement, and ϕ is a reduction factor. The nominal punching shear resistance of concrete for slabs without shear reinforcement according to ACI Code, Section 22.6.5.2, is:

$$V_c = \min \begin{cases} 0.33\lambda\lambda_s b_o d \sqrt{f'_c} \\ 0.17\lambda\lambda_s b_o d \sqrt{f'_c} \left(1 + \frac{2}{\beta}\right) \\ 0.083\lambda\lambda_s b_o d \sqrt{f'_c} \left(2 + \frac{\alpha_s d}{b_o}\right) \end{cases} \quad (\text{SI units}) \quad (1)$$

Where, λ is the concrete density factor equal to 1 for a normal-weight concrete, λ_s is the size effect factor given in 22.5.5.1.3, b_o is the perimeter along the critical section located $d/2$ from the face of the column, and, d is the effective depth of the slab, f'_c is the specified compressive strength of concrete (f'_c is limited to 68 MPa, and no effect of flexural

reinforcement is considered according to ACI code, section 22.6.3.1), β is ratio of long side of column or loaded area to short side $\alpha_s = 40$ for an interior column, 30 for edge columns, and 20 for corner columns, where the words interior, edge, and corner refer to critical sections with 4, 3, or 2 sides, respectively. The nominal punching shear resistance of slabs with shear reinforcement (stirrups) is calculated according to ACI code, section 22.6.6.1

3. RESEARCH SIGNIFICANCE

The study examines the distribution of shear stresses around the interior slab-column connection without rigid reinforcement using (STAAD.ProV8i) relying solely on ACI318 recommendations. The Flat plate is modeled by a 3D solid element; flexural reinforcements are modeled with a one-dimensional element, and rigid shear reinforcements are modeled with a two-dimensional element or a plate element. STAAD.ProV8i software helps to model any shape of rigid shear reinforcement and the embedment condition of rigid reinforcement into concrete is done using the interaction module of meshing, which provides bonding between steel and concrete.

4. STRUCTURAL FINITE ELEMENT MODEL

The interior column and the flat plate are replaced with three-dimensional solid elements. Flexural reinforcement bars have a line shape that represents the actual size and location and is embedded in the solid finite element by meshing. The refined fragment model is shown in Figure 6. The geometric characteristics of the refined fragment model were as follows: the length of the column was taken equal to 1 m below the concrete plate and 1 m above the plate; it is assumed that the area of the concrete plate is 5 m x 6 m; the thickness of the concrete plate is 20 cm; the concrete column section is 40cm x 40cm; flexural reinforcement bars have a diameter of 16 mm and are spaced 15cm center to center in two perpendicular directions. Types of finite elements of the refined fragment model were: concrete column and concrete flat plate - eight-node isoparametric solid element; reinforcement bar - spatial bar; steel plate is a meshed plate element. The partitioning step of the solid elements is 0.05 m. The characteristic compressive strength f'_c of concrete, given in the model, is 28 MPa with Young's modulus value of 24.87GPa and Poisson's ratio value of 0.17. Similarly, Young's modulus value of steel is given as 200GPa with Poisson's ratio of 0.3. Steel reinforcement behaves elastically up to the Von Mises yield stress of 420 MPa for round deformed bars and 248 MPa for steel plates.

Figure 6 .The refined fragment model of the interior column–plate connection with embedded flexural reinforcement

5. SHEAR STRESS FIELDS

5.1. Reference Shear Stress Diagram, Model №1

The first case, model N_{0.1} includes the computation of the axial compressive force P_u generated in the interior column due to the shear stress initiated in the critical section at distance $d/2$ from the column faces, which have a value computed according to ACI 318-19M. Table 22.6.5.2. Model N_{0.1} has flexural reinforcement, but does not have rigid shear reinforcement and is considered a calibrated model. The design maximum load P_u , that can be applied to the interior column and does not lead to punching failure of the concrete plate, is computed by ACI 318-19M. Table 22.6.5.2, formulae and the values of the generated shear stress' fields in the plate are considered as the maximum allowable shear stress that can be resisted by concrete.

$$P_u = V_u = \min of \begin{cases} 0.75 \cdot 0.33 \cdot 1 \cdot \sqrt{28} \cdot 174 \cdot 4 \cdot 574 \cdot 10^{-3} = 523.21 kN \\ 0.75 \cdot 0.17 \cdot \left(1 + \frac{2}{1}\right) \cdot 1 \cdot \sqrt{28} \cdot 174 \cdot 4 \cdot 574 \cdot 10^{-3} = 808.59 kN \\ 0.75 \cdot 0.083 \cdot \left(2 + \frac{40 \cdot 174}{574}\right) \cdot 1 \cdot \sqrt{28} \cdot 174 \cdot 4 \cdot 574 \cdot 10^{-3} = 1858.84 kN \end{cases}$$

The shear stress fields in the column-plate joint region due to the applied axial force $P_u = 523.21 kN$ in the interior column, as shown in the Figure 7 and Figure 8.

Figure 7 Distribution of absolute values of shear stress fields τ_{xy} in the middle layer of plate thickness, which can be resisted individually by concrete plate

Figure 8 Absolute values of shear stress fields' τ_{xy} in the middle layer of plate thickness at different distances from critical section perimeter.

Shear stress values, as a rule, should not exceed 1.3 MPa over the area of the critical section for model N_{0.1}, but locally; narrowly limited zones are observed in the middle layer of plate thickness where the shear stress values exceed 30-45%, as shown in Figure 8. The maximum shear stress values obtained are assumed to be permissible, and when they occur in the concrete, the joint should not fail by punching. The reference diagram of shear stress in the control model N_{0.1}, as shown in Figure 9, is constructed for the most stressed section based on STAAD.ProV8i results

Figure 9 Reference diagrams of shear stresses τ_{xy} and τ_{zy} in the middle layer in the control model N_{0.1}.

5.2. Shear Stress Diagram with Rigid Reinforcement, Model № 2.

By increasing the applied axial load up to $P_u=796\text{kN}$, the shear stresses values here, in the model No.2, are increased by 37-40%. This means that the concrete in the design critical section cannot resist alone the new P_u ; therefore, additional shear reinforcement is required for punching shear resistance. Additional reinforcement, consisting of separated four plates bent at 90° , is located at the corners of the interior column, starting with an initial thickness of 5 mm and a constant height of 160 mm, and arranged in pairs in two mutually perpendicular directions, as illustrated in Figure 10. The axes of the plates are displaced symmetrically from the axes of the column at a distance of 150 mm, noting that stresses in the model № 2 concrete with the introduction of rigid reinforcement should not exceed the maximum values of the reference diagram and the shear stress fields to ensure that the punching failure does not occur.

The effective perimeter for shear of the critical section b_0 shown in Figure 10 is determined from geometry, with b_0 expressed in terms of l_v as in equation 2.

$$b_0 = 4 \cdot \sqrt{2 \left(\frac{l_v - c}{2} \right)^2} + 4c \quad (2)$$

Where:

l_v = the overall distance between the outermost edges of shearheads arms in one direction; c = side size of the square column. The ultimate allowable load at new section as per ACI318-19M section 22.6.9.8 recommendations

$$P_u = V_u = \phi \cdot 0.33 \cdot \lambda \cdot \lambda_s \cdot \sqrt{f'_c} \cdot b_0 \cdot d$$

$$P_u = 0.75 \cdot 0.33 \cdot 1.1 \cdot \sqrt{f'_c} \cdot \left(4 \cdot \sqrt{2 \left(\frac{l_v - c}{2} \right)^2 + 4c} \right) \cdot d \quad (3)$$

Then, the required length l_v is found by transposing the preceding equation 3; we get the dependence of l_v on P_u :

$$l_v = \frac{P_u}{0.7 \cdot d \cdot \sqrt{f'_c}} - 0.4c \quad (4)$$

Figure 10 (a) The proposed four steel plates A36 (b) Steel plates embedded into the joint, model No.2 (c) Plan of plate-column concrete joint reinforced with rigid shear reinforcement.

This theoretical minimum extension length of shearheads arms l_v in one direction ensures the exclusion of punching. However, in fact, this value should be increased by the anchorage length. Since the introduction of steel plates of obviously longer length is not rational from an economic point of view, it is necessary to check the minimum length of the plates, which eliminates the punching failure of the joint. For this, several calculations with different lengths of elements of rigid reinforcement are carried out using STAAD.ProV8i program. Substituting the value of $P_u = 796 \text{ kN}$ into the formula (3), we obtain $l_v = 756.7 \text{ mm}$, say $l_v = 80 \text{ cm}$. The first model considered to have a length element of rigid reinforcement equal to the theoretical minimum value $l_v = 80 \text{ cm}$. In all models, the length of the elements in both directions was the same. After that, this length is increased by an amount equal to the plate thickness $h = 20 \text{ cm}$. The stress fields arising in the region of the plate-column connection are obtained and in accordance with these stress fields, the stress diagrams τ_{xy} and τ_{zy} were constructed in sections in which plates extend along the larger and smaller spans. Plots are presented in Figure 11.

Figure 11 Plot of stresses τ_{xy} and τ_{zy} in the most stressed section with $l_v = 80\text{cm}$

Concentrations of shear stresses, as in Fig. 9.2 and Fig.9.3, appear in places where the plate breaks. The shear stress values τ_{xy} and τ_{zy} are 6% and 11% higher than the allowable values, respectively. With an increase in length l_v by $2h$, that is, with a length $l_v = 1.2\text{ m}$, the shear stresses do not exceed the permissible values. This is illustrated in Fig.9.8 and Fig.9.9, as well as on the graphs (Figure 12 and Figure 13).

Figure 12 Plot of stress τ_{xy} in the most loaded section with $l_v=120\text{cm}$

Figure 13 Plot of stresses τ_{xy} and τ_{zy} in the most stressed section with $l_v=120\text{cm}$

A brief summary of the main provisions of the state of stresses arising in the steel plate with $l_v = 120$ cm can be summarized as shown in Table 1.

Table 1 Magnitude of stresses acting on the steel plate

Type of stress	Shear		Membrane			Von Mises yield criterion stress	Tresca Stress Criterion
	τ_{xz} (local) N/mm ²	τ_{yz} (local) N/mm ²	σ_x (local) N/mm ²	σ_y (local) N/mm ²	τ_{xy} (local) N/mm ²	Top or bottom of a plate surface N/mm ²	Top or bottom of a plate surface N/mm ²
Max τ_{xz}	5.153						
Min τ_{xz}	-5.119						
Max τ_{zy}		3.907					
Min τ_{zy}		-3.907					
Max σ_x			75.271				
Min σ_x			-78.602				
Max σ_y				4.963			
Min σ_y				-10.172			
Min center τ_{xy}					-13.020		
Max Von Mises						85.040	
Min Von Mises						9.876	
Max Tresca							90.550
Min Tresca							10.173

The stresses, as illustrated in Table 1 above, do not exceed 38% of the design resistance of the metal $F_y = 248$ MPa. However, as the thickness of the plate decreases, the rigidity of the structure decreases, which leads to a significant increase in stresses in the concrete. Consequently, such an option of installing rigid reinforcement at the column slab connection ensures the elimination of punching.

6. CONCLUSIONS

According to the data presented in the research, the following conclusions can be drawn due to the Introduction of rigid reinforcement in the form of four plates arranged in pairs in two perpendicular directions.

- The shear stresses decrease significantly with increasing plate thickness, maintaining a constant plate height.
- A simple formula has been defined for calculating the minimum length of rigid reinforcement elements to withstand the punching stress at plate column connection in the form of:

$$l_v = \frac{P_u}{0.7 \cdot d \cdot \sqrt{f'_c}} - 0.4c + 2h$$

- A new formula for the effective perimeter of the critical section for shear is determined in the form of:

$$b_0 = 4 \cdot \sqrt{2 \left(\frac{l_v - c}{2} \right)^2 + 4c}$$

ACKNOWLEDGEMENT

This work has been performed during sabbatical leave granted to the author from Al-Balqa' Applied University (BAU) during the academic year 2017/2018. The author would like to express gratitude for the support provided by the University at all stages of this work.

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