Concerning flat slab to column connection

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Abstract

A brief review of flat slabs and the flat slab-column connection concentrating on parameters influencing connection behavior including punching shear, shear and flexural reinforcement, connection ductility and so on is presented below.

1 Flat plates or flat slabs

Since early 1950s the trend toward lighter and more flexible construction configurations led to the rise of flat plates, particularly for medium- to high-rise office and residential buildings(FEMA 274, 1997) although it was originally patented years earlier(Gasparini, 2002; Kahn, 1909; Hardison, 1925). Due to their beamless nature and being free of obstructions, flat slabs provide many advantages such as lower story height, better lighting and ventilation, arrangement of piping and wiring, more clear space in addition to architectural flexibility and easier formwork in construction.

Despite the architectural charm this system doesn't have a well established seismic reputation and due to it's vulnerability to seismic loads it's behavior is normally a matter of concern(Hosahalli and Aktan, 1994; Derecho and Kianoush, 2001) which doesn't make one wonder why ACI 318 (2019) applies the lowest inertia moment factor (0.25) to flat slabs and flat plates for elastic analysis at factored load levels. Research on two-way flat slabs with drop panels indicates that in this particular case they have similar behavior to flat plates(Odello and Mehta, 1967). It's notable that detailing requirements for two-way slabs without beams as mentioned earlier for flat plates in ACI 318 (1994, Chapter 21) appear only in the section relating to areas of moderated seismic risk which in turn suggests that ACI 318 (1994) considers the use of flat plates as acceptable components of the lateral-load-resisting system only for areas of moderate seismicity(Derecho and Kianoush, 2001). Punching shear failure at slab-column connection for this system because of its brittle nature and potential in triggering progressive collapse may lead to catastrophic losses as evidenced in the past(King and Delatte, 2004; Park, 2012).

Ductile detailing of all structural connections, including also for those with only gravity load has become a key concept that was learned as a result of failures observed during the 1971 San Fernando Valley Earthquake. Slab-column connections in a flat plate structure though only gravity loaded must maintain load bearing capacity at maximum lateral displacement allowed by the lateral load bearing system during which there is chance of brittle failure modes like slab punching shear taking place if this displacement is not absorbed by slab-column interfaces(UFC 4-023-03, 2016). This mode of failure could occur with little or no warning signs that has resulted in the progressive collapse of this type of structures as in the 1985 Mexico City earthquake in which 91 flat plate buildings collapsed and 44 others were severely damaged due to punching failure(Ghali and Megally, 2000). Notably flat slab performance remained poor in the 2017 Mexico City earthquake also(**?**). Although punching failure occurs during the earlier stages of construction too, when concrete has yet to harness reliable strength to resist shear effects(Gardner, 2011).

The Iranian code of standard practice for seismic design of buildings(BHRC 2800, 2014, Section 3-3-5-5) restricts the use of flat slabs either with or without drop panels to buildings at maximum 3 stories or 10m tall. Whereas in ACI 318 (2019) slab-column frames without beams are at most accounted as intermediate moment frames and per ACI 318 (2019, Section R18.2) are not permitted as part of seismic-force-resisting systems for structures assigned to Seismic design classification (SDC) D, E, or F and are only allowed as a gravity or secondary system where shear walls form the seismic or lateral force resisting system (SFRS or LFRS). The reasoning for this simply put by ACI/ASCE 421.3 (2015, Chapter 7) is that these frames cannot meet the detail requirements for the level of energy dissipation and ductility demanded for special moment frames. Even when an independent lateral-force-resisting system is provided ACI/ASCE 421.3 (2015) suggests that flat plate-column connections should be designed to accommodate the moments and shear forces associated with the displacements or story drifts during earthquakes. Thus all members in slab-column frames not designated as part of the SFRS should be designed to support gravity loads while subjected to design displacements.

During the recent decades various laboratory tests concerning flat slabs have been carried out which are distinguishable in three specimen size classes as

- isolated slab-connections,
- complete single floors,
- and complete frames least two stories tall.

Numerous research has been done as presented herein studying isolated slab-column connections under combined gravity and lateral loading(Dovich and Wight, 2005; Drakatos et al., 2016; Hawkins and Mitchell, 1979; Hueste et al., 2007; Kang and Wallace, 2006; Megally and Ghali, 2000a; Pan and Moehle, 1989; Robertson et al., 2002a; Setiawan et al., 2019; Tian et al., 2008a; Almeida et al., 2016) while fewer have been in the form of size scaled floors tested under cyclic loads(Hwang and Moehle, 1993, 2000; Rha et al., 2014) and only four consisted of complete buildings(Coronelli et al., 2021; Fick et al., 2017; Moehle and Diebold, 1984; Kang and Wallace, 2004). Coronelli et al. (2020) provides an state of the art review of these larger scale tests and an introduction to the "Slab STRESS" research program reported by Coronelli et al. (2021) in which a full-scale flat slab specimen is tested. Coronelli et al. (2021) carried out seismic tests for service and ultimate actions using pseudodynamic technique with virtual walls in which the flat slab frame was the secondary element as suggested by ASCE/SEI 41 (2017); FEMA 356 (2000)[1].

[1]ASCE/SEI 41 (2017) classifies building components as either primary or secondary in respect to their role in resisting the seismic forces and whether their failure or degradation would compromise building lateral stability, thus secondary elements would be any building structural element whose purpose is to support gravity loads. ASCE/SEI 41 (2017) requires that both primary and secondary components be evaluated to confirm their seismic force and deformations do not behave inconsistently with the selected performance level therefore the standard also states that where a component intended in the original building design as primary is deformed beyond the point where it can be relied to resist earthquake effects, the secondary designation may be used.

Moment and crack distribution and redistribution in multi bay slab column frames due to nonlinear behavior and progressive deterioration of joints are major draw backs for isolated connection subassemblies which might fail at providing accurate boundary conditions(Einpaul et al., 2015, 2016). Einpaul et al. (2015) investigated moment redistribution between hogging and sagging moments and compressive membrane action influence on conventional isolated connection experiments suggesting that the punching capacity of continuous slabs with low amounts of flexural reinforcement on the interior column regions may be underestimated in the codes of practice. Note that there are plenty of tests on internal slab-column connection specimens while results for edge or corner columns are less frequent.

1.1 Punching shear

Early foundations for punching shear design was laid in Talbot (1913) though through examination was not carried out until much later (Elstner and Hognestad, 1956; Moe, 1961) without moment application that are reviewed in detail by Ghoreishi et al. (2013); Yang et al. (2011); Hamada et al. (2008). On the other hand Kinnunen and Nylander (1960) proposed that flat slab punching strength is related to slab flexural deformations in the column vicinity that was further improved by Shehata and Regan (1989); Broms (1990).

Column connection region behavior in reinforced concrete elements is characterized by flexural crack developments at incipient loading stages (1a and 2c).

Typically with low reinforcement ratios at an ultimate state these cracks may govern connection behavior leading to a potential yield of longitudinal reinforcement (3c) (Hallgren, 1996). If flexural behavior doesn't govern and with reinforcement stresses nigh on bar yield stress, flexural cracks would propagate into shear cracks leading to a failure mode defined as flexural punching(fib, 2001).

In case of high reinforcement ratios the slab behaves stiffer with reinforcements bearing lower stresses and higher stresses concentrated in the inclined concrete compression zone developed near the column (3a)(Hallgren, 1996). Punching shear failure is described as the development of a diagonal crack with variable inclination starting from the column face on the slab compression side and ending at the slab tension face resulting in the dislocation of a conical body of concrete slab (1) (Regan, 1986).

Typically in concrete members shear is carried by

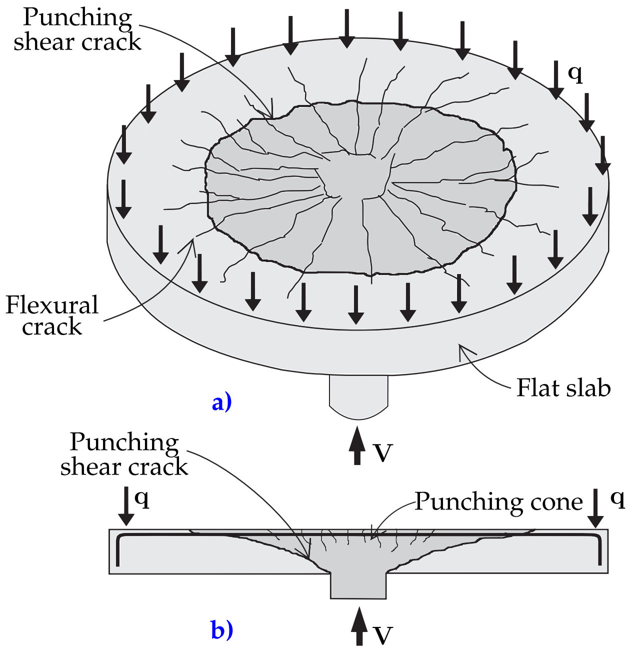


Figure 1: Isolated interior flat slab-column connection region and typical punching shear failure surface, adapted from Bompa and Onet (2015): a) Isometric view; b) Section view.

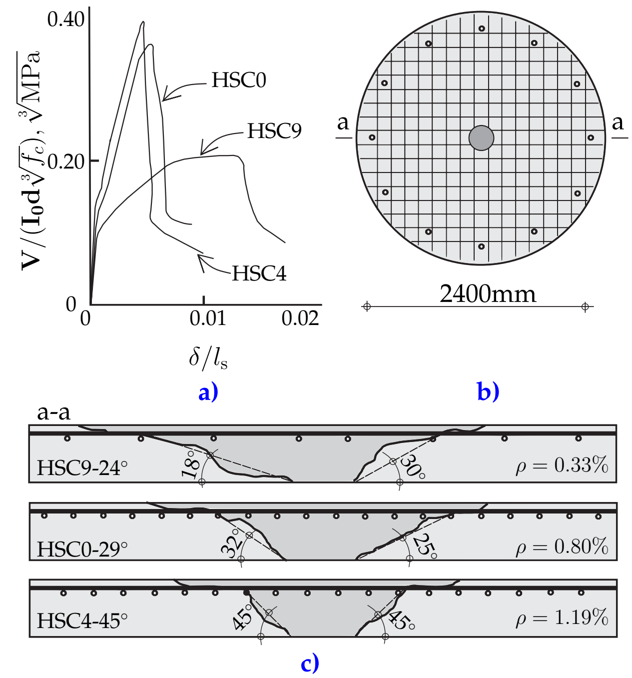


Figure 3: Test results (HSC0, HSC4, HSC9) carried out by Hallgren (1996), adapted from Bompa and Onet (2015): a) Structural response; b) In-plane geometrical configuration; c) Sectional view of saw cuts.

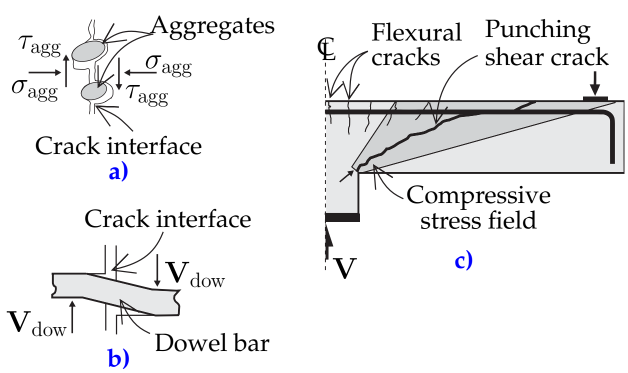


Figure 2: Slab-column connection cracks, adapted from Bompa and Onet (2015): a) Aggregate interlock; b) Dowel action; c) Compression field.

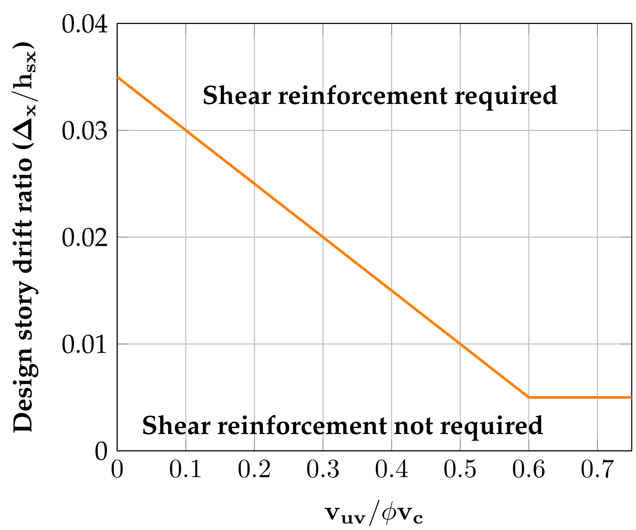


Figure 4: Slab shear reinforcement requirement criteria, recreated from ACI 318 (2019).

- the interlock and frictional resistance of the cracked interface aggregates against crack slip and growth(Walraven and Reinhardt, 1981),
- dowel bar shearing(Dei Poli et al., 1987, 1992; Ince et al., 2007; Paulay and Loeber, 1974; Taylor, 1970),
- transmission through the concrete section compression zone(Chana, 1987),
- and residual stress transfer through the crack tip(2 a,b)

Stress distribution in the column-slab connection region governs punching shear crack inclination angle and the slab punching shear strength is governed by the amount of shear carried or resisted by the cracked interface. The inclination angle and punching shear capacity depend on member geometry[2] and structural parameters[3]. As the inclination angle reduces and the cracked interface parallels the slab faces further, the interlocking surface widens, also more bars get involved in dowel action thus larger amounts of shear are transferred. Inacio et al. (2015) carried out an experimental study(5) over high strength concrete (HSC) flat slabs without shear reinforcement testing four specimens, one of which was built with normal strength concrete (NSC). Significant load capacity increase compared to the reference NSC specimen was observed and the results furthermore indicate that longitudinal reinforcement increase had a positive influence on punching shear capacity(Inacio et al., 2015). Emam et al. (1997); Marzouk et al. (2001) studied high strength and light weight concrete application in flat slabs. Kadhim et al. (2021) showed punching shear capacity improvement with ultra-high performance concrete (UHPC) use in flat slabs without shear reinforcement through a nonlinear finite element procedure implementing DSS (year) verified against Saleem et al. (2011); Zohrevand et al. (2015).

Qi et al. (2021) carried out concentrated load tests on eight similar flat slab specimens in three groups to study UHPC depth and area influence on flat slab punching shear behavior the application of which at full depth(6) over the critical section area transformed the brittle punching shear failure mode into a ductile punching shear-flexure one while limited depth UHPC application over this area ended in brittle failure.

[2]Depth, slenderness, column dimensions to slab thickness ratio.
[3]Material strengths, aggregate properties, reinforcement layout and so on.

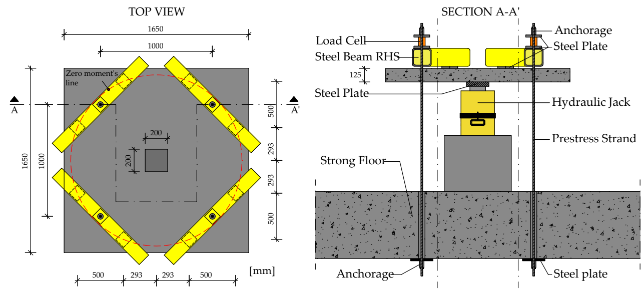


Figure 5: Test setup(Inacio et al., 2015).

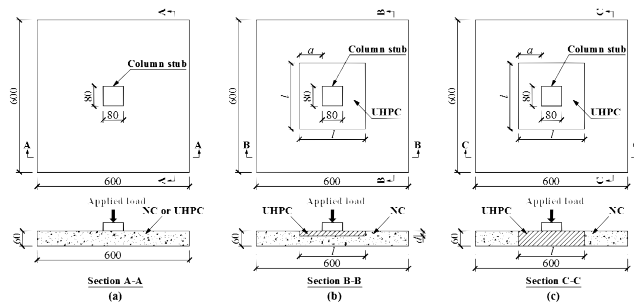


Figure 6: Test setup(Qi et al., 2021).

Ramos et al. (2022) tried high-performance fiber reinforced concrete (HPFRC) application in slab-column connection vicinity as a substitute for shear reinforcement with four specimens under combined gravity and horizontal reversed cyclic loading in which connection drift capacity substantially improved compared to reference specimens and the HPFRC reinforced specimens performed better than specimens with HSC. Ricker et al. (2017) carried out ten punching tests on slab-column connections with double-headed studs as shear reinforcement nine of which had fiber reinforced UHPC units in the slab-column connection compression zone(7) that reached significantly higher failure loads compared to normal specimens.

Another concrete behavior improvement approach for slab-column joints has been introduced through the use of steel fiber additives in the concrete mix as studied by Gouveia et al. (2018); Abdel-Rahman et al. (2018); Ju et al. (2015); Gouveia et al. (2014) that improves a plethora of structural characteristics in addition to punching shear strength including load bearing and flexural capacities along with structural stiffness and connection ductility.

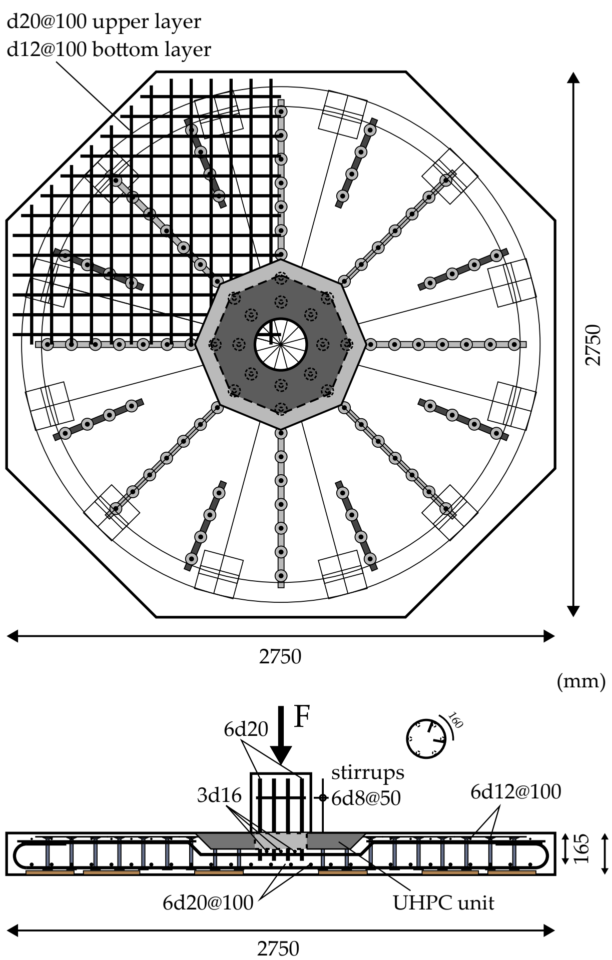


Figure 7: Layout of flexural reinforcement and double-headed studs for test specimen DUHPC2(Ricker et al., 2017).

2 Slab-column connection

Tests on slab-column connections subjected to reversed cyclic loading(Carpenter et al., 1973; Symonds, 1976) indicate that ductility of flat slab-column connections can be significantly increased through the use of stirrups enclosing bands of flexural slab reinforcement passing through the columns[4]. (Robertson and Johnson, 2006) observed that an increase in the flexural stiffness of a slab-column connection induces a higher shear demand. Also (Robertson and Johnson, 2006) concluded that connections with increased slab flexural reinforcement will support greater lateral loads, but the increased eccentric shear transfer may result in premature punching shear failure. However tests on interior column-to-slab connections with lightly reinforced slabs with and without shear reinforcement (Peiris and Ghali, 2012; Hawkins and Ospina, 2017; Bayrak et al., 2009; Muttoni, 2008; Dam et al., 2017a; Muttoni, 2008) have shown that yielding of slab flexural tension reinforcement in the vicinity of the column or loaded area leads to increased local rotations and opening of any inclined crack existing within the slab. In such cases, sliding along the inclined crack can cause a flexure-driven punching failure at a shear force less than the strength calculated by the two-way shear equations of ACI 318 (2019, Table 22.6.5.2) (2) for slabs without shear reinforcement and less than the strength calculated in accordance with ACI 318 (2019, Section 22.6.6.3) for slabs with shear reinforcement.

Tests of slabs with flexural reinforcement less than $A_{s,\min}$ have shown that shear reinforcement does not increase the punching shear strength(ACI 318, 2019). However, shear reinforcement may increase plastic rotations prior to the flexure-driven punching failure(Peiris and Ghali, 2012). Whereas Megally and Ghali (2000b); Kang and Wallace (2006); Robertson et al. (2002b) argue that connection ductility will increase with shear reinforcement application in slab-column connections. This is sound mainly because slab connection will yield or fail in flexure prior to punching shear. Kang and Wallace (2006); Megally and Ghali (2000b); Robertson and Durrani (1991, 1993); Anggadjaja and Teng (2008) showed that gravity load strongly influences flat slab-column connection lateral ductility and with gravity load increase connection lateral displacement capacity decreases that was also observed in reviewed test data

[4]Such shear-reinforced bands would essentially function as shallow beams connecting the columns.

(a) DUHPC2 (b) DUHPC3 (c) DUHPC4 (d) DUHPC9

Figure 8: UHPC units(Ricker et al., 2017).

by ACI/ASCE 421 (2010) to assess gravity load effect on lateral drift capacity for interior flat plate-column connection specimens with and without shear reinforcement. Also tests have shown that beam-column joints laterally supported on four sides by beams of approximately equal depth exhibit superior behavior compared to joints without all four faces confined by beams under reversed cyclic loading(Hanson and Connor, 1967).

2.1 Shear reinforcement

Provisions for shear reinforcement at slab-column connections for non-SFRS members were adapted in ACI 318 (2005) to reduce slab punching shear failure likelihood. ACI 318 (2019, Section 18.14.5) states for nonprestressed slabs that design story drift ratio is limited to the following

$$\frac{\Delta_x}{\mathbf{h_{sx}}} \geq \mathbf{0.035} - \mathbf{0.05}\frac{\mathbf{v_{uv}}}{\phi \mathbf{v_c}} \tag{1}$$

presented in 4. If the drift ratio exceeds this amount(1) thus slab shear reinforcements either stirrups or headed studs should be provided at any slab critical section considering that $\mathbf{v_{uv}}$ is evaluated with load combinations including earthquake effect $\mathbf{E}$ and $\Delta_x/\mathbf{h_{sx}}$[5] is the greater of values for adjacent stories above and below the slab-column connection while $\mathbf{v_c}$ is evaluated as below.

$$\mathbf{v_c} = \min \begin{bmatrix} 1 \\ 1/2 + 1/\beta \\ 1/2 + \alpha_s d/4b_o \end{bmatrix} \frac{\lambda_s \lambda \sqrt{f'_c}}{3} \tag{2}$$

where β is the ratio of long to short column sides or supporting element, $\alpha_\mathbf{s}$ is the factor accounting for connection location, $\mathbf{d}$ is the slab effective depth, $\lambda_\mathbf{s}$ is the size effect factor, and regardless of shear reinforcement the critical section perimeter $\mathbf{b_o}$ is at least $\mathbf{d}/\mathbf{2}$ away from column edges or changes in slab thickness minimizing $\mathbf{b_o}$ while with shear reinforcement an extra critical section $\mathbf{d}/\mathbf{2}$ away from the outermost peripheral line of shear reinforcement considering the shape should be a polygon minimizing $\mathbf{b_o}$ is added[6],

$$\begin{aligned} \lambda_s &= \sqrt{2/(1+0.004d)} \geq 1, \lambda = 1 \\ \alpha_s &= \begin{cases} 40 & \text{interior} \\ 30 & \text{edge} \qquad \text{columns} \\ 20 & \text{corner} \end{cases} \end{aligned} \tag{3}$$

Development of 2 is reviewed in Bayrak et al. (2009). Also note that most of the existing research on punching shear are based on test results from reinforced concrete column and slab specimens.

Megally and Ghali (2002); Moehle (1996); Kang and Wallace (2006); Kang et al. (2007) identified the likelihood of punching shear failure about the slab critical section without moment transfer considering story drift ratio and shear stress $\mathbf{v_{uv}}$ due to gravity loads and the vertical component of earthquake loads therefore no induced moment calculations would be necessary. Megally and Ghali (2000a); Kang et al. (2009); Moreno and Bastos (2008); Song et al. (2012); Krüger et al. (1998b, 1999) found that moment presence significantly decreased slab punching shear resistance that is in line with earlier findings (Hawkins and Corley, 1973; Islam and Park, 1976). The above mentioned requirement (4) can also be satisfied by increasing slab thickness, changing the design to reduce story drift ratio, or a combination of all of the above.

The shear reinforcement in the slab critical section should provide $\mathbf{v_s} \geq \mathbf{0.29}\sqrt{\mathbf{f'_c}}$ extending at least four times the slab thickness from the support face adjacent to it. In case the above requirements are not met for

[5] $\mathbf{v_{uv}}/(\phi \mathbf{v_c})$ is otherwise known as the ratio between the acting vertical shear force and the punching shear resistance or Gravity shear ratio (GSR) in the literature. Larger GSRs imply lower frame or structure horizontal deformation or drift capacity and are possible with shear reinforcement(Gouveia et al., 2019).

[6] There will be two critical sections for shear reinforced sections.

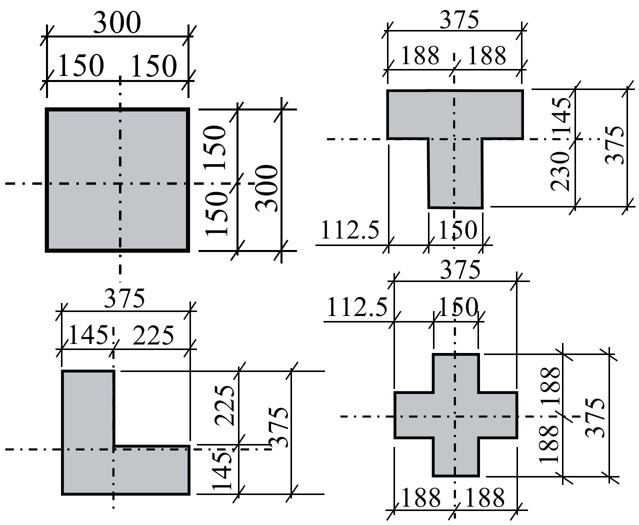


Figure 9: Model column geometries(Akinpelu et al., 2023).

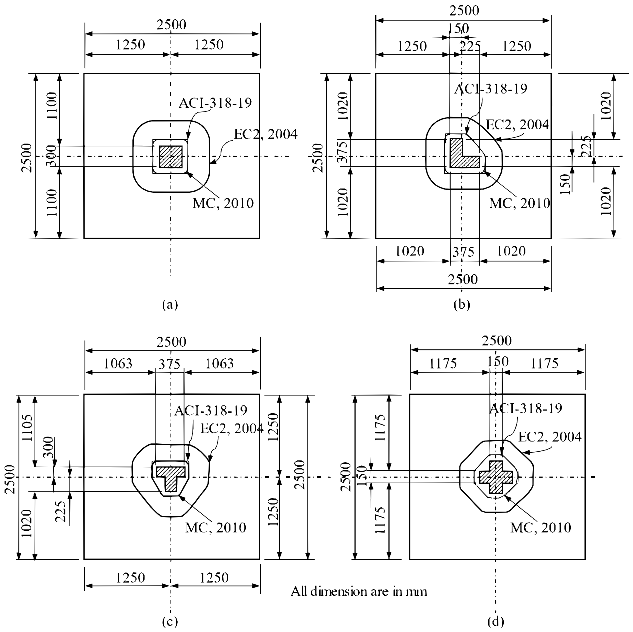


Figure 10: Layouts of modelled flat slabs showing critical sections as per various code provisions, adopted from (Akinpelu et al., 2023); a) Square column; b) L; c) T; d) +.

two-way slabs that are designated as part of SFRS ACI 318 (2019, Section 18.4.5.8) states that two-way shear stress caused by factored gravity loads without moment transfer should not exceed $0.4\phi \mathbf{v_c}$ beyond which nonprestressed slab-column connections in laboratory tests by Pan and Moehle (1989) exhibited reduced lateral displacement ductility.

Variety in shear reinforcement lies partly on a practical point, when various construction limitations arise such as difficulties in proper bar placement or improper stirrup configurations after slab reinforcement is done or even shear head sections barring essential reinforcement in the column and slab considering the ACI 318 (2014) requirements such as uninterrupted inserts.

Moe (1961) tried to increase column effective size placing steel plates with a certain overhang as illustrated in 11(a) over the column and was successful at achieving increased capacity noting that shear was concentrated on plate corners. Akinpelu et al. (2023) carried out a numerical study of column shape effect on flat slab punching shear behavior implementing the finite element package ABAQUS(DSS, year) in which the authors tried L, T and + column geometries maintaining the column area and perimeter also introducing damage plasticity into the model(9). The simulation results of Akinpelu et al. (2023) showed that the + geometry renders a larger column effective size and thus behaves better than others in comparison to rectangular sections(Akinpelu et al., 2023). Hawkins (1968) studied the bearing strength of these steel plates and proposed a plate thickness formula to provide maximum increase in column effective size without disadvantageous corner effects.

2.1.1 Shearheads

Shearheads are another type of shear reinforcement that despite being originally introduced in Wheeler (1936) are seldom used in current practice so their design provisions have been omitted in ACI 318 (2019) and may be designed following ACI 318 (2014) provisions. Design criteria in ACI 318 (1971) were developed based on shear forces in Hawkins and Corley (1973); Corley and Hawkins (1968) who performed 21 tests, 16 of which were with cruciform grillages of I and C steel sections complementing Corley and Hawkins (1968); Hawkins and Corley (1973)(11 b,c) and the approach has been maintained up to ACI 318 (2014). Results from Hawkins (1974) contrasted with ACI 318 (1971) provisions and Hawkins and Corley (1973) also showed that after cracking in the slab around the connection the subsequent shear forces applied are carried by the shearhead module while failure initiates either through punching along the shearhead perimeter or reaching the shearhead flexural capacity. Commentary on ACI 318 (2014, Section 22.6.9)

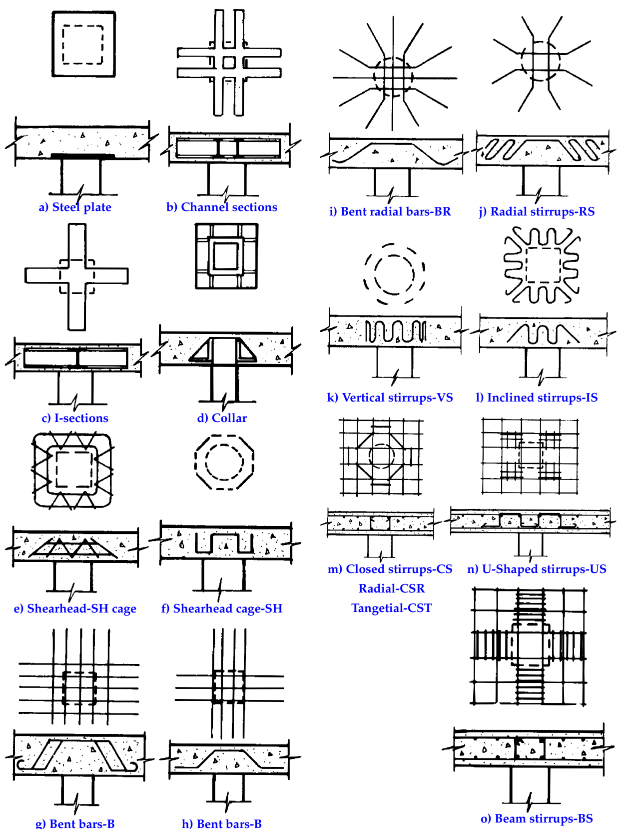


Figure 11: Shear reinforcements(Hawkins, 1974); a) Steel plate(Moe, 1961); b) Channel sections(Hawkins and Corley, 1973); c) I sections(Hawkins and Corley, 1973); d) Collar(Tasker and Wyatt, 1963); e) Shearhead-SH, cage; f) Shearhead cage-SH; g,h) Bent bars-B; i) Bent radial bars-BR; j) Radial stirrups-RS; k) Vertical stirrups-VS; l) Inclined stirrups-IS; m) Close stirrups-CS, Radial-CSR, Tangential-CST; n) U-shaped stirrups-US; o) Beam stirrups-BS.

suggests that a minimum flexural strength should be provided to ensure slab shear strength is reached before shearhead flexural strength is exceeded. The above mentioned shearheads (11 b,c) increased slab shear capacity as shearhead arm length grew thus Corley and Hawkins (1968); Hawkins and Corley (1973) introduced the critical sections and equations for column face moment and shear illustrated in 12. These results were adopted by Al-hamd et al. (2018) who proposed two new shearhead designs (13) based on nine laboratory tests with eccentrically loaded specimens with an offset corbel (14) similar to Krüger et al. (1998a) followed by a numerical study of connection behavior. Load and moment in the manner described by Krüger et al. (1998a); Al-hamd et al. (2018) is quite advantageous compared to Ghali and Megally (2000); Kang et al. (2009); Moreno and Bastos (2008); Song et al. (2012); Hawkins and Corley (1973); Islam and Park (1976) who produced moments by applying a lateral load to one of the column sides. While reporting results from testing shearhead systems to improve steel column-flat slab connection ductility under cyclic loads Eder et al. (2012) made a similar argument to the above pointing out that slab behavior is controlled by shearhead stiffness and high connection strength and ductility are realizable when using partially integrated shearheads if dissipative elements are designed to yield in shear first. Steel columns and flat slabs are generally connected with steel inserts welded to the column and then integrated into the slab[7].

Shearhead response in accord with 12 is ensured by anchoring the shearhead steel section flanges within the slab compression zone based of which ACI 318 (1971) requires having the compressions flange within 0.3d of the slab compression surface. Godycki and J. (1984) showed embedment length on ultimate connection capacity for flat slabs with cruciform or + shearheads under eccentric loading.

Bompa and Elghazouli (2015) proposed a method for punching shear evaluation of reinforced concrete flat slab-column connections without shear reinforcement. Having studied shear behavior of beam to steel column assemblages(Bompa and Onet, 2015) with a series of 5 large scale tests with shearheads or arms welded to the steel column and embeded in reinforced concrete beams, Bompa and Elghazouli (2016) carried out six large scale tests and developed an analytical model for steel column-flat slab connection with shearheads(15) that showcased the positive influence

[7] More on this in 4.

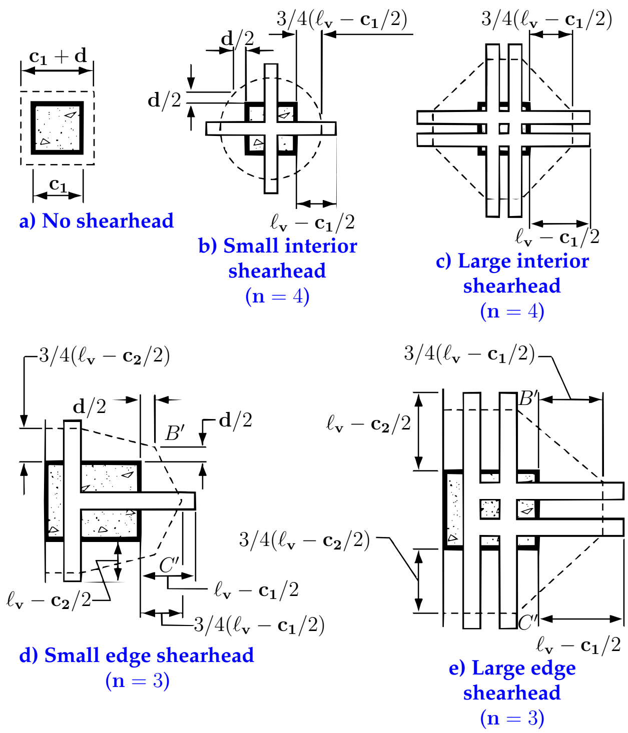


Figure 12: Flat slab shear critical sections with and without shearhead adapted from ACI 318 (2014); a,b,c) Interior columns same as Hawkins (1974); d,e) Edge columns.

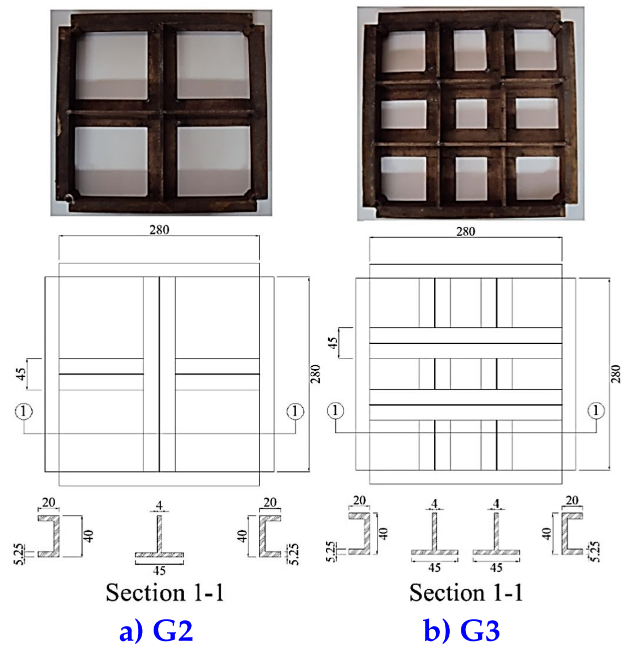


Figure 13: Proposed shearhead layouts by Al-hamd et al. (2018); a) G2; b) G3.

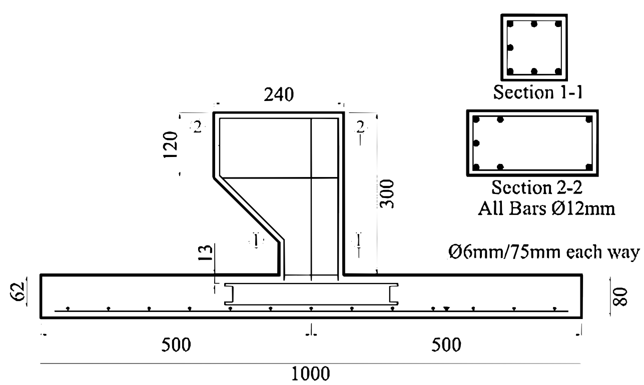


Figure 14: Tested slab geometry with offset corbel by Al-hamd et al. (2018) (all dimensions in mm).

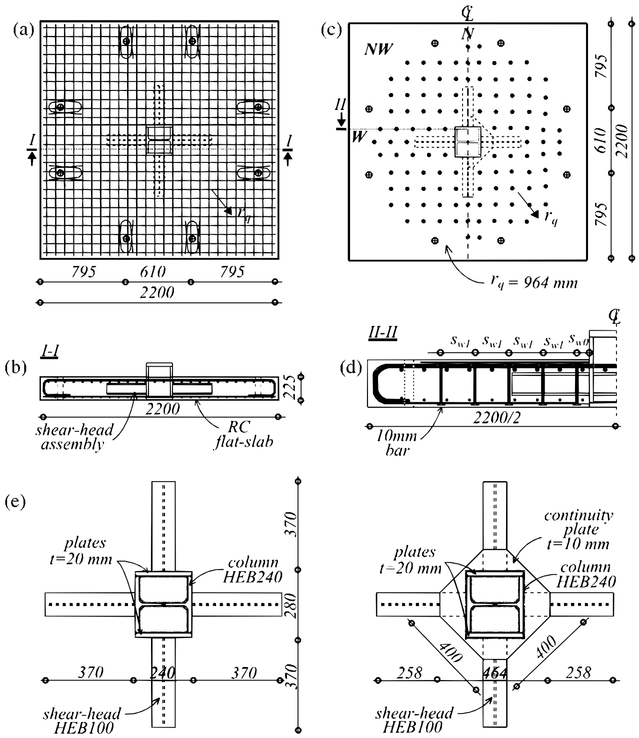


Figure 15: Specimen arrangement Bompa and Elghazouli (2016); a) Longitudinal reinforcement layout; b) I-I cross-sectional view; c) transverse reinforcement layout; d) II-II cross-sectional view from transverse reinforcement; e) Shearhead details, without continuity plate on the left and with continuity plate on the right.

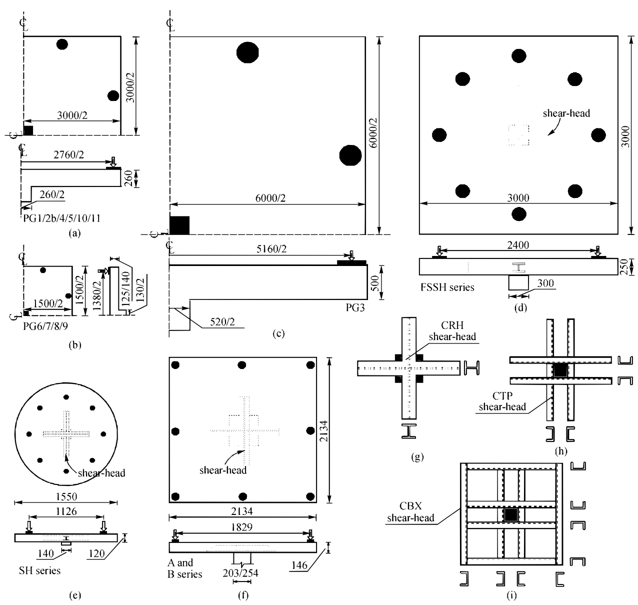


Figure 16: Schematic representation of specimens numerically studied in Bompa and Elghazouli (2020); a) Full scale specimens PG1, PG2b, PG5, PG10, PG11; b) Half scale specimens PG6, PG7, PG8, PG9; c) Double scale specimen PG3 from Guandalini et al. (2009); d) FSSH series; e) SH series from Chana and K. (1996); f) A and B series (Hawkins and Corley, 1973); Cruciform shear-heads made of g) Welded back-to-back channel or I sections (CRH); h) Two pairs of channels at the support region (CTP) shearheads; i) Closed-box shearheads (CBX).

of continuity plates connecting the shearheads. The shearheads were welded to the steel column and fully embedded in the slab while in the 6 test specimens the embedment length, slab thickness and shearhead cross section were maintained whereas shearhead assemblage configuration, flexural reinforcement ratio and transverse reinforcement contributions varied. Bompa and Elghazouli (2020) Carried out a a thorough three-dimensional nonlinear numerical and parametric analysis implementing the concrete damage plasticity models validated against experimental results from three test series(Guandalini et al., 2009; Chana and K., 1996; Hawkins and Corley, 1973) and proposed analytical models for connection rotational response as well as the ultimate strength of reinforced concrete slab systems provided with fully embedded shearheads(16). Eder et al. (2010) validated a nonlinear finite element model against a large-scale reinforced concrete flat slab without shear reinforcement that failed in punching and subsequently carried out

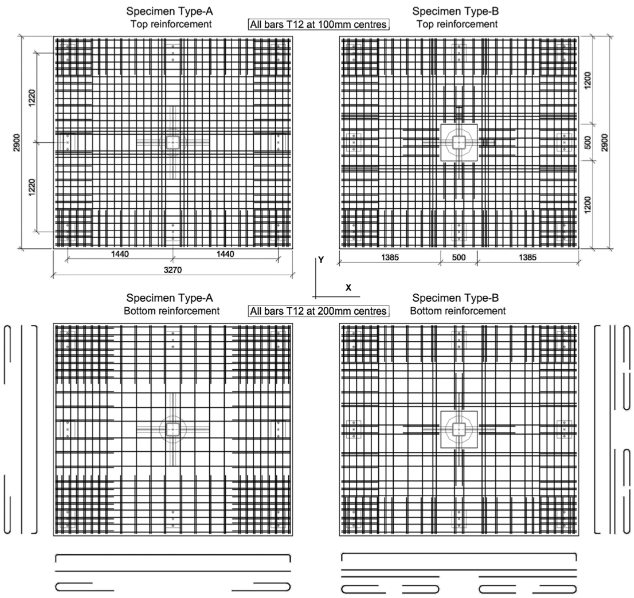


Figure 17: Specimen reinforcement layouts in Eder et al. (2011).

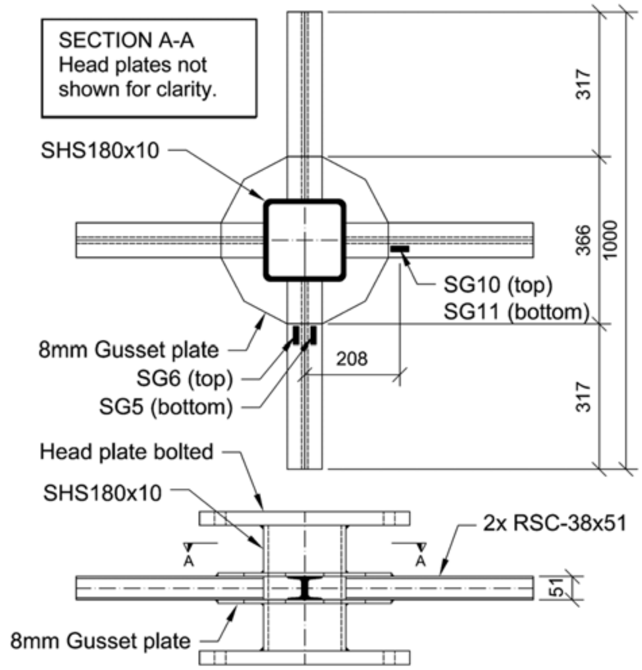


Figure 18: Shearhead detail proposed in Eder et al. (2011).

a parametric analysis of key parameters. These authors modelled a large scale hybrid reinforced concrete flat slab specimen tested at the London Imperial College with a steel column and ACI 318 (2008) type steel shearhead as well. Their analysis suggests that loads are principally transferred into the shearhead arm tips if the failure surface lies outside shearhead arm length while a more uniform load transfer into these arms would occur if the failure surface lies inside. Eder et al. (2011) presented a novel partially integrated shearhead detail(17) with a gap around the tubular steel column(18) to enable shearhead yield in the shearhead prior to slab punching shear and compared it with the typical ACI 318 (2008) shearhead presenting test results from four large-scale specimens and numerical analysis under gravity and cyclic lateral loadings following which Eder et al. (2012) further studied the inelastic performance and design of this novel ductile steel shearhead with additional u-bars under these loads. Eder et al. (2012) notes that I section shear arms perform better than closed box sections due to improved composite action with the slab and also suggests anchoring of the shear arms to the slab through end plates or otherwise to further resist the siginificant axial forces which arise as a result of geometric nonlinearity.

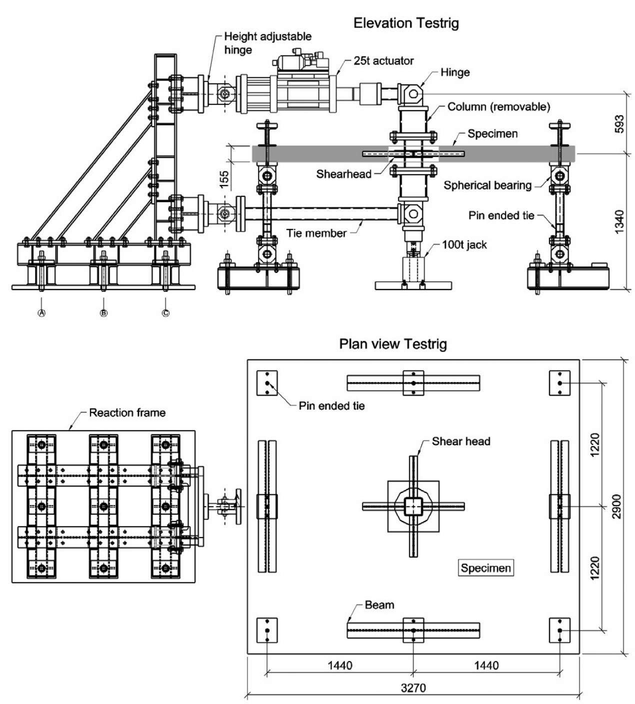


Figure 19: Test rig used for large-scale tests(Eder et al., 2012).

2.1.2 Bent bars and stirrups

Bažant and Cao (1987) improved understanding of member size effect on slab behavior with large scale specimens and proposal of a design formula. Following an experimental program Shehata and Regan (1989) presented a mechanical model to estimate punching shear resistance of axisymmetric slabs under concentric loads. In order to analyse punching shear resistance of flat slabs with shear reinforcement Gomes and Regan (1999) proposed an analytical model based on Kinnunen and Nylander (1960); Shehata and Regan (1989) validated on 12 full scale test specimens. Birkle and Digler (2008) focused on the influence of slab thickness while Beutel and Hegger (2002) studied anchorage effect on shear reinforcement effectiveness over the shear punching critical section. Detailed investigations by Guandalini et al. (2009); Muttoni (2008) resulted in a analytical model of punching shear strength as a function of slab rotation that formed the basis for fib (2010) as well. Fernandez Ruiz and Muttoni (2009) delved deeper into shear reinforcement contribution to punching shear strength based on slab rotation applying the critical shear crack theory. Several transverse reinforcement configurations were tested through 16 flat slab specimens with various depth by Lips et al. (2012) to verify design effectiveness of methods presented in both the United States and Europe ACI 318M (2014); EN 1992-1-2 (2004) in addition to the mechanical models presented in Muttoni (2008); Fernandez Ruiz and Muttoni (2009). Note that EN 1992-1-2 (2004) does not provide any design guidance for members with shearheads.

2.1.3 Headed studs

Bent bars and stirrups make for rather tiresome to install shear reinforcement and this problem has caused the widespread use of shear studs (stud rails) during the recent decades. Several horizontal cyclic loading tests Dilger and Cao (1994); Robertson et al. (2002a); Brown (2003); Broms (2007); Tan and Teng (2005); Kang and Wallace (2008); Hong-gun et al. (2007); Matzke et al. (2015); Isufi et al. (2019) have investigated headed stud efficiency, placing it among the most effective and practical solutions for flat slab shear reinforcement(20). Ghali and Youakim (2005) attributes shear stud enhanced behavior to proper anchorage on both stud ends compared to other shear reinforcement. Dam and Wight (2016) presented the results of three full-scale slab-column connection test carried out to evaluate stud layout effectiveness in flat slabs with low flexural reinforcement(21). Dam et al. (2017b) carried out seventeen large-scale tests of interior slab-column connections with different shear stud layouts and slab flexural reinforcements(22).

Isufi et al. (2019) tested four reinforced concrete slabs with shear studs and a control specimen without any shear reinforcement under constant gravity loads and reversed horizontal cyclic displacements that differed in gravity load and stud perimeters.

2.2 Flexural reinforcement

Isufi et al. (2020) preformed combined horizontal reversed cyclic and gravity loading tests on two flat slab-column specimens with and without shear reinforcement and later continued with three more specimens in Isufi et al. (2021) to better study flexural reinforcement(23) influence on flat slab-column connection seismic performance which was aligned with previous results(Muttoni, 2008; Guandalini et al., 2009; Ghali and Gayed, 2019; Torabian et al., 2019) indicating that flexure rather than punching shear governs slab load carrying capacity with flexural reinforcement decrease. Low reinforcement ratios lead to flexural reinforcement yield onset and a more ductile behavior from the slab until shear crack widening at relatively large displacements ends in punching(Muttoni, 2008; Torabian et al., 2020, 2019). The importance of flexural reinforcement was noted earlier tests as well(Hawkins et al., 1974; Symonds et al., 1976). Morrison et al. (1983) tested five relatively thin slab-column connection specimens, three of which had different flexural reinforcement ratios and their response was dominated by flexure since these specimens were tested without gravity load allowing high ultimate drift capacity. Morrison et al. (1983) Notes theoretical yield onset for the lowest flexural reinforcement specimen. Emam et al. (1997); Marzouk et al. (2001) investigated flexural reinforcement influence on slab-column connection seismic behavior. Detailing of top and bottom reinforcement of slab-column connection under lateral cyclic loading was studied by Robertson and Johnson (2006) with six specimens that showed horizontal load bearing capacity improvement by flexural reinforcement increase while deformation capacity could be hindered by premature punching shear. Five specimens with different flexural reinforcement were tested by Tian et al. (2008b) one with reversed horizontal cyclic loading to failure and two with gravity loading to failure after undergoing a cyclic loading protocol, that exhib-

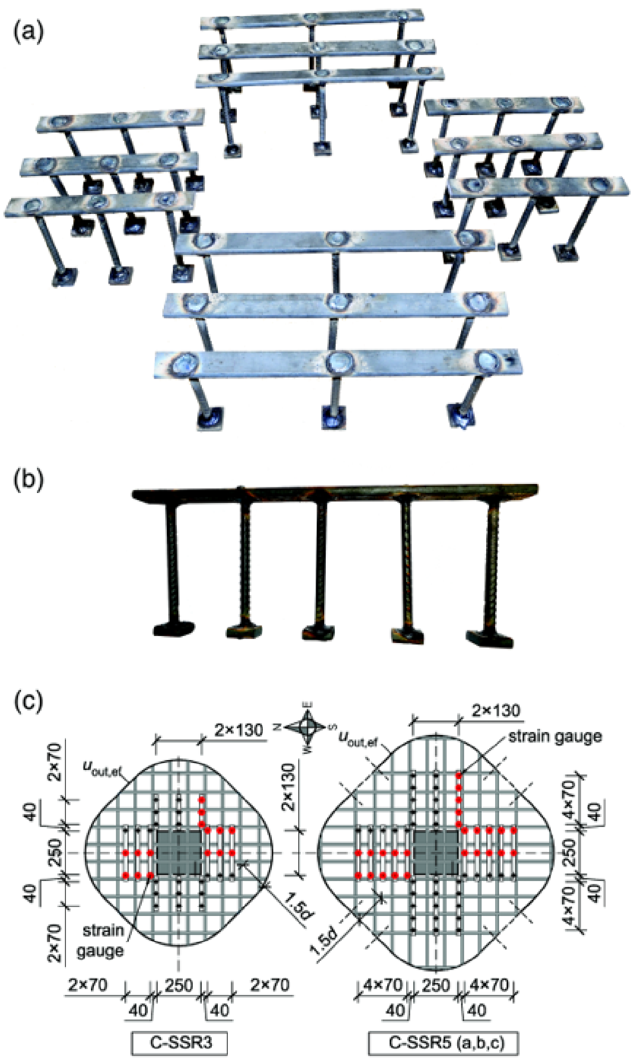


Figure 20: Shear Studs(Isufi et al., 2019): a) Complete set of studs for specimen C-SSR3; b) Stud rail along the longitudinal (N-S) direction for specimens with five rows of studs; c) Layout and instrumentation.

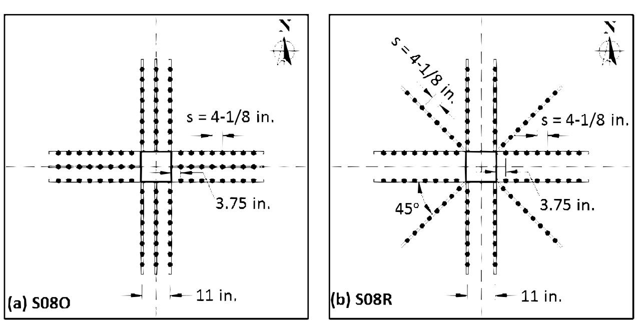


Figure 21: Shear stud layouts in S08O and S08R specimens(Dam and Wight, 2016).

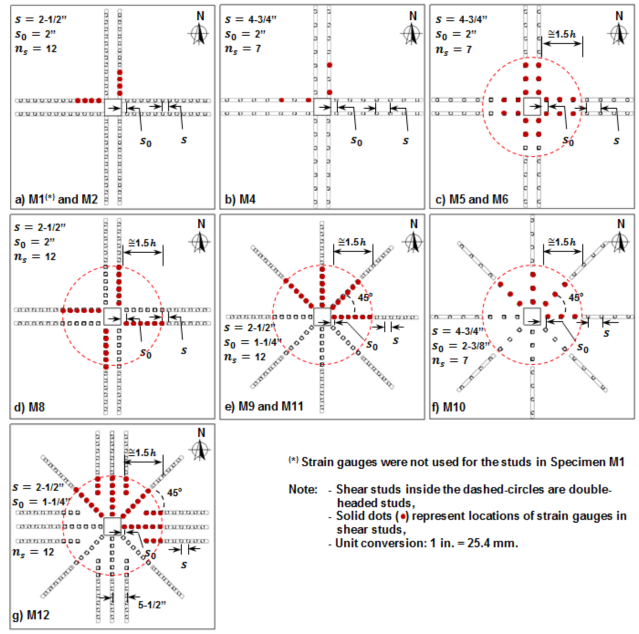


Figure 22: Shear stud configurations and strain gauge locations for the M specimen series(Dam et al., 2017b).

ited punching shear capacity increase under gravity load and lateral stiffness increase under horizontal load with flexural reinforcement increase. A total of 13 internal slab-column connection specimen test were conducted by Drakatos et al. (2016) in which monotonic and reversed cyclic loading, gravity load and flexural reinforcement varied. Slab-column connection lateral stiffness improved with flexural reinforcement increase while its influence on unbalanced moment and deformation capacities remained heavily dependent on applied gravity load levels for monotonic lateral loading, also cyclic loading affected slabs with higher flexural reinforcement ratios less than those with lower ratios.

3 Progressive collapse

Considering progressive collapse and column removal it could be stated that in flat slabs the pressure absorbed formerly by the removed columns cannot in the absence of beams be transferred thus these slabs are more prone to collapse than slab-beam-column systems(Singh et al., 2023).

On paragraph 3-3-5-4 (BHRC 2800, 2014) emphasizes use of moment frames or dual systems for buildings above 15 stories or 50m in height stating that in this class of buildings the designer should not rely merely on shear walls or braced frames for lateral seismic loads.

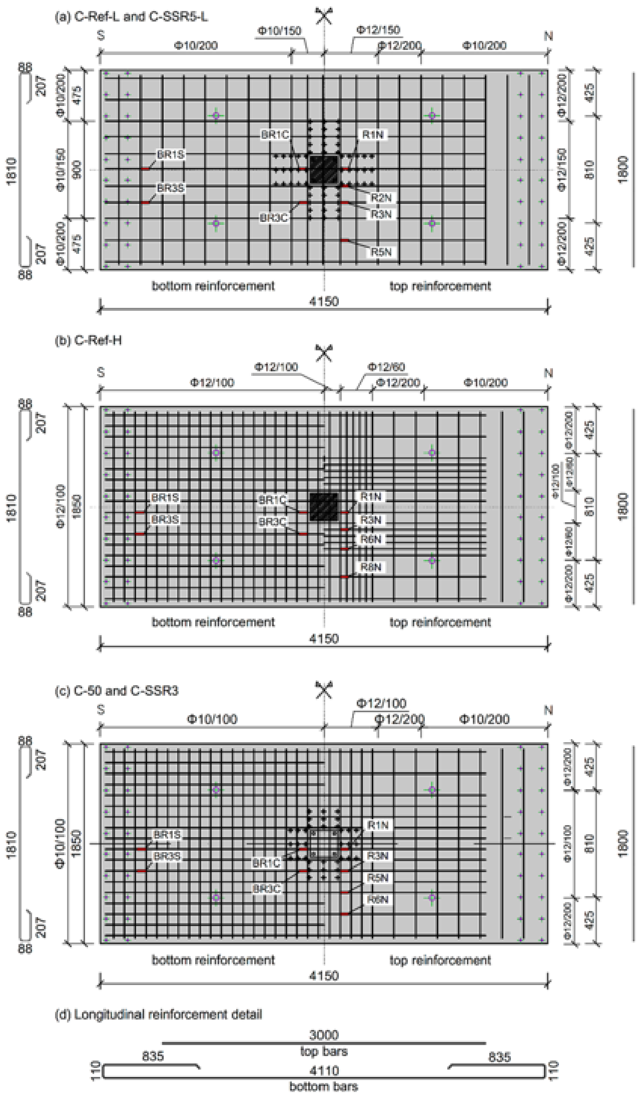


Figure 23: Flexural reinforcement details(Isufi et al., 2021).

4 Steel column-flat slab systems

Concrete filled steel tube (CFT) columns have become more prevalent in construction practice during the last couple of decades providing relatively higher strength and ductility(Morino, 1998), reduced labor costs, less formwork, lower reinforcement requirements and ease of concrete pour compared to conventional reinforced concrete columns. Although the main disadvantage lies in the discontinuity due to the smooth interface between the reinforced concrete slab the CFT column which is quite troubling when considering punching shear(Yu and Wang, 2018).

This gave a rise in research for providing proper connection detail between the slab and CFT column to alleviate shear transfer most of which focused on using shearheads(Satoh and Shimazaki, 2004; Lee et al., 2008; Yamaguchi et al., 2008; Eder et al., 2011, 2012; **?**; Kim et al., 2014; Yan and Wang, 2014; Bompa and Elghazouli, 2016; Lee et al., 2019) in the form of either I, T, rectangular steel tube sections or shear studs welded on the column connection perimeter extending into the slab section(Yu and Wang, 2018), while a combination of these was also reported(Lee et al., 2008). In Yan and Wang (2014); Lee et al. (2008) top and bottom reinforcement were passed through the steel columns via provided holes which improves connection post-punching behavior and load bearing capacity. Following an experimental program Chen et al. (2020) carried out six full-scaled tests of interior slab-column sub-assemblages under monotonically increased gravity loading with negligible eccentricity to study differences between reinforced concrete and CFT columns(25) and also shear enhancement around the CFT column(24). Test results showed that composite connections exhibited comparable punching shear strengths and failure modes while shear enhancement around the CFT column effectively shifted the failure plane away from the column faces hence increasing connection punching shear capacity(Chen et al., 2020).

Rafiee et al. (2021) conducted a reversed cyclic loading test on a half-scale post-tensioned (PT) flat slab-steel column specimen to examine the seismic details of their proposed exterior connection(26). Yan and Wang (2016) ran an extensive parametric numerical study investigating punching shear resistance of hybrid steel tubular column to reinforced concrete flat

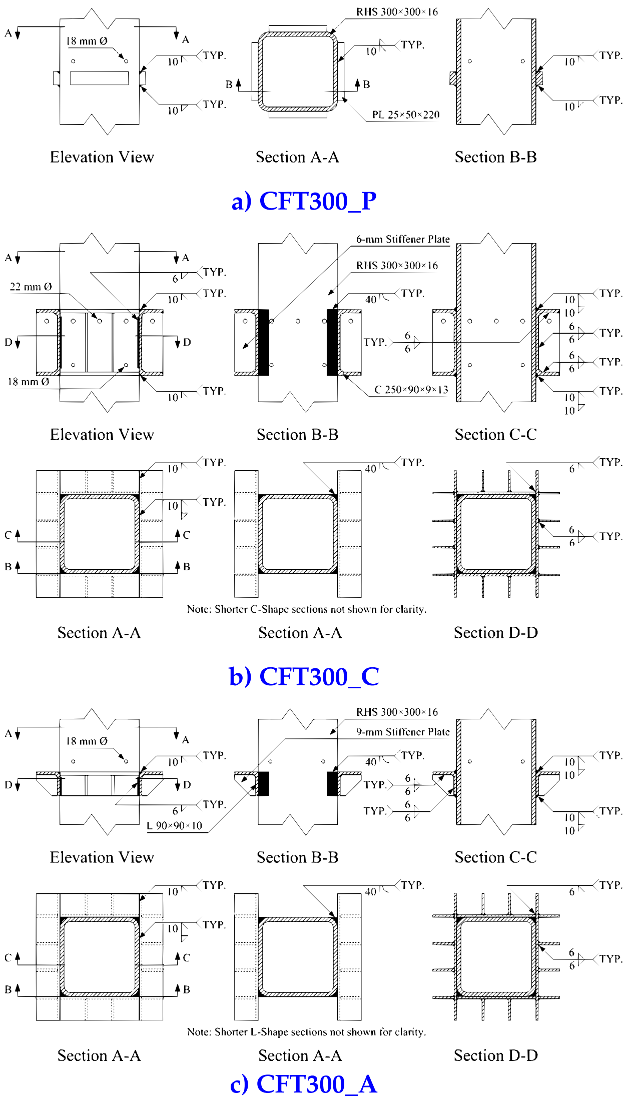


Figure 24: Shear enhancement detail(Chen et al., 2020).

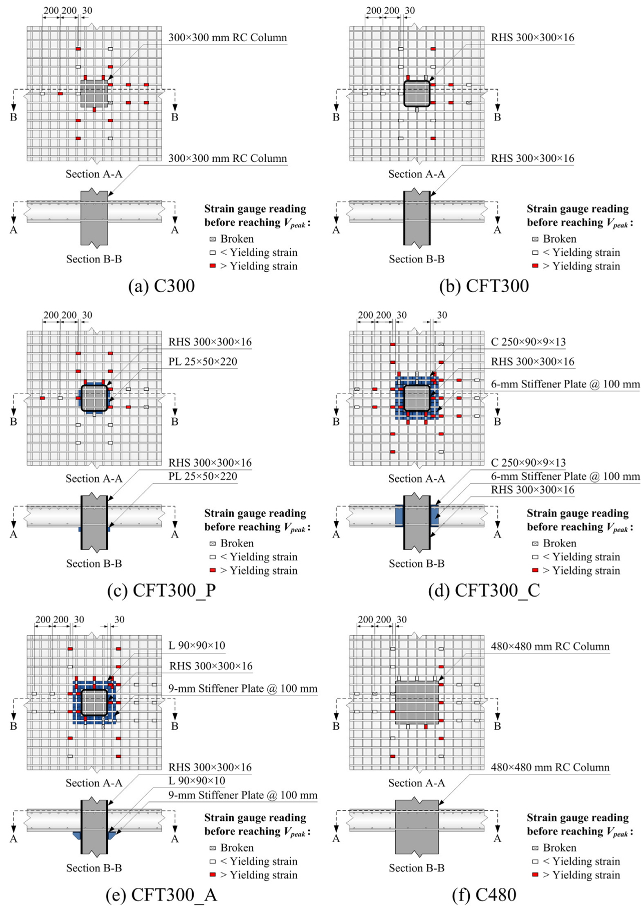


Figure 25: Slab-column connection detail(Chen et al., 2020).

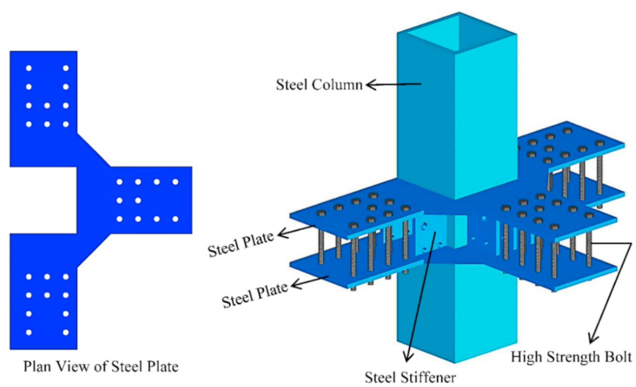
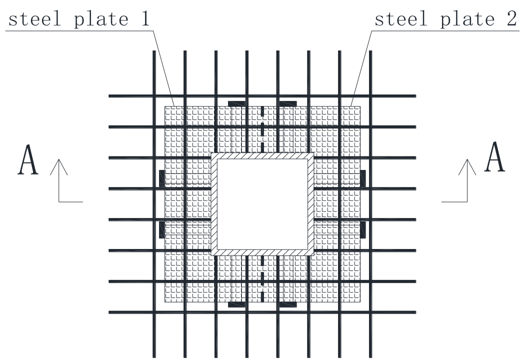
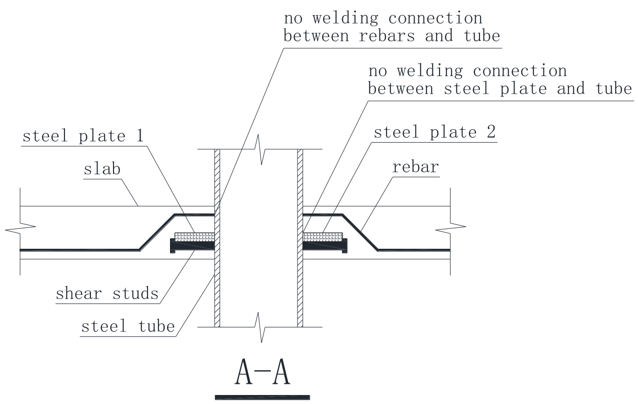


Figure 26: Propose flat slab to steel column connection configuration(Rafiee et al., 2021).



(a) Plan layout around connection



(b) Detailed elevation view

Figure 27: Proposed steel tube to concrete flat slab connection detail(Yu and Wang, 2020).

slab connection using shearhead arms in which study parameters included column shape, shearhead arm properties and slab reinforcement. These authors in a subsequent numerical and analytic investigation(Yu and Wang, 2020) proposed an innovative shear connection for flat slab to steel column connections using welded shear studs, steel plates and bent-up bars(27). Zhang et al. (2018) proposed a new type of connection mechanism between a prefabricated reinforced concrete flat slab and square steel tube column(28) through an experimental test followed by a complementary numerical simulation. A total of nine simply supported slab-column connection specimens were subjected to vertical load by Zhou et al. (2021) in addition to a finite element study to investigate punching shear behavior of slab-column connections embedded with steel skeletons which changed the mode of failure from punching shear into flexural punching(29).

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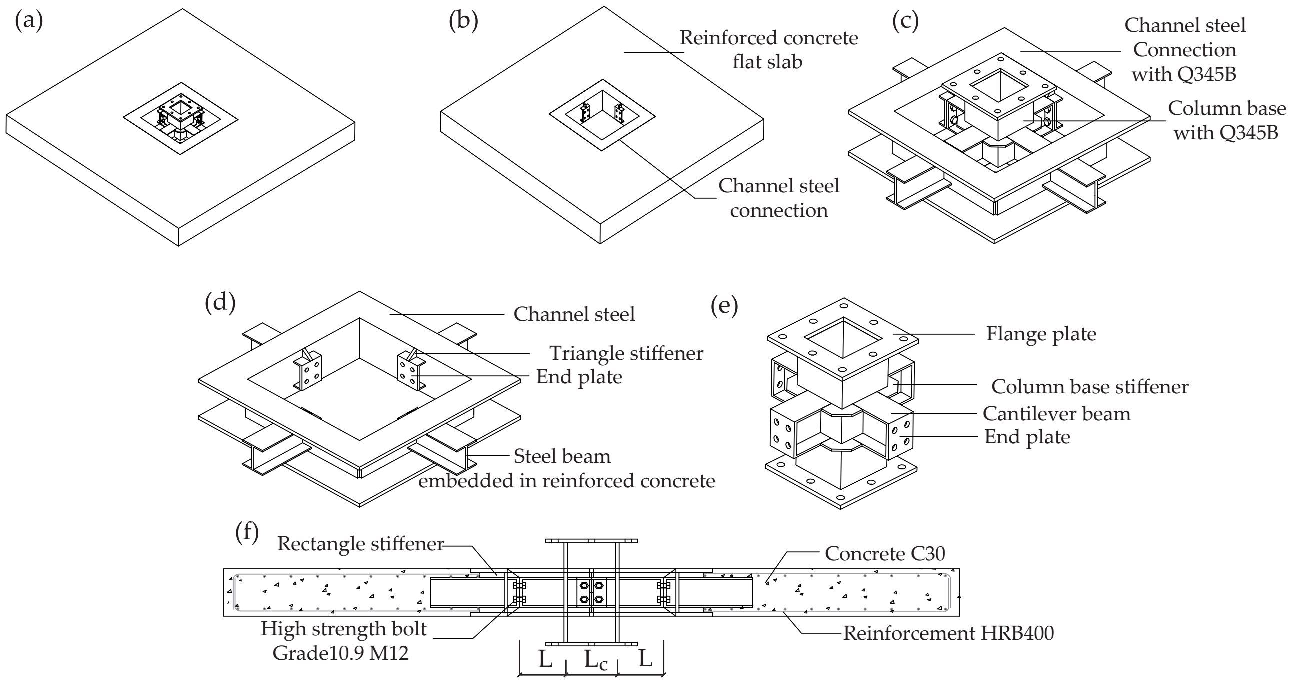

Figure 28: Configuration of prefabricated reinforced concrete flat slab to square steel tube column connection(Zhang et al., 2018): a) Slab-column connection; b) Reinforced concrete flat slab and channel steel section; c) Column base connection; d) Channel steel connection; e) Column base; f) Connection cross-section.

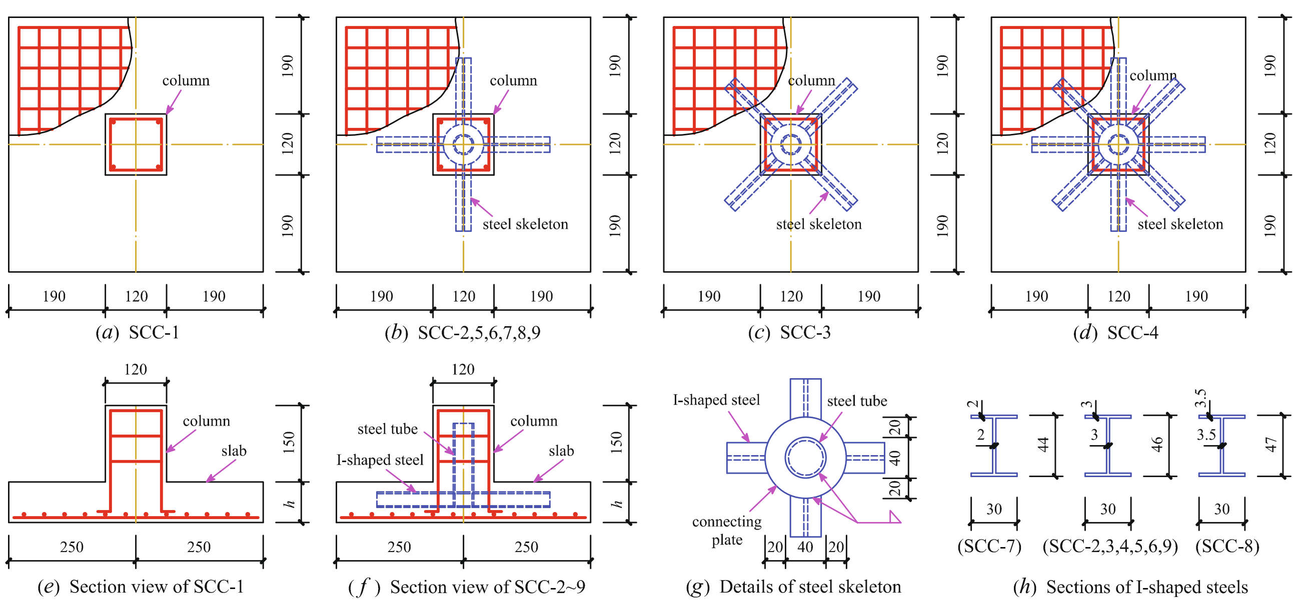

Figure 29: Specimen details(Zhou et al., 2021).

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