# CHAPTER 2

# LITERATURE REVIEW

## 2.1 General

In this chapter, a review of relevant previous research work is presented. In particular, research related to the concrete filled steel tubes, and the longitudinal shear failure, such as investigations on push-out tests and composite beams are discussed.

## 2.2 Concrete-Filled Steel Tubes

### 2.2.1 Concrete-Filled Steel Tube as Composite Members

Concrete filled steel tubes are commonly used for columns. In Japan composite columns are exclusively used in moment frames Ollgaard et al (1971). In United States very high strength concrete has been used to maximise the stiffness of composite column (Davies 1967). The main advantages of composite columns over pure steel or reinforced concrete columns are,

- The concrete core adds stiffness and compressive load capacity and reduces the potential for local buckling of steel tube.
- The steel tube reinforces the concrete core to resist tension, bending moment, and shear.
- The steel tube acts as formwork for the concrete during casting.



In foundation construction there is a great potential in using composite piles. In this case the steel tube is mainly used as formwork during casting of the concrete core. The reinforcement bars in concrete core are possibly excluded by existing steel tube. Therefore this type of composite piles save material and production time.

Further, the hollow steel sections are strengthened by filling the section with concrete. In this way it is possible to increase the stiffness and load capacity of an existing structure. Finally it increases the durability of the structure (Roddenberry 2002).

### 2.2.2 Confining Effect on Concrete

Confined concrete greatly increases the maximum compressive strength, the stiffness, and extended strain at peak stress Grant et al (1977). Also, concrete can sustain large deformation without substantial reduction of the load-bearing capacity and fails gradually in a ductile way.

In short concrete-filled hollow steel sections, it was found that the columns with circular cross-section have more concrete confinement than columns with rectangular cross section (Roddenberry 2002).

Also, it was found that the influence of confining pressure on the maximum compressive strength of high strength concrete is not pronounced as on that of normal strength concrete.

### 2.2.3 Interaction between the Concrete Core and the Steel Tube

It was identified that the manner in which the load was transferred between the concrete core and the steel tube, consisted of four different mechanisms. The loads transfer mechanisms were defined as,

- Adhesion due to chemical reaction and/or suction forces along the interface, resulting from capillary action during the hydration process.

  It is an elastic brittle load transfer mechanism that is active mainly at the early stage of loading when the relative displacements are small. The shrinkage of the concrete core has an adverse effect on the development of adhesion forces.

- Micro-interlocking between the concrete and the steel due to surface irregularities of the steel tube.

  It is relates to the surface roughness of the steel tube and surface irregularities increased the bond strength.

- Friction between the concrete core and the steel tube due to normal forces.

  The magnitude of the friction force developed in concrete filled steel tubes is connected with the confining effect and depends in the same way on the rigidity of the tube walls against pressure perpendicular to their plane.

- Binding or curvature effect, which results from imposing compatible global deformations.

  Binding mechanism means that the load is transferred between the concrete core and the steel tube because they are bonded together by imposing compatible global deformations. In other words curvature and variations in shape of the steel tube can be defined as binding mechanisms.

These can also be classified into two groups according to scale as follows,

- Micro-effect: adhesion and micro-interlocking

- Macro-effect: friction and binding

It is found that better compaction could enhance both the effect of micro-effects and macro-effects, resulting in higher bond strength.

### 2.2.4 Mechanical Behaviour of Concrete-Filled Steel Tubes

#### 2.2.4.1 Axially Loaded CFST Columns

Columns loaded axially in compression will behave in two different ways. That is as a short column or long column. Straight columns under purely concentric axial loading can be found very rarely. Therefore, a more realistic approach to examine the CFST column behaviour incorporates the bending moment caused by the eccentric loading.

A thin walled short column specimen fails either by elastic or local buckling of the steel tube, or by a shear failure in the concrete followed by local buckling of steel tube, which is in a state of yielding.

In a thick walled short column specimen, with high strength steel, the concrete will reach its compressive strength limit and may crush before the steel yields. For lower strength steels, the failure of thick walled short columns begins with the yielding of the steel.

If the long CFST column is sufficiently slender, stability rather than strength will govern the ultimate load capacity and second order effects become more critical. Overall column buckling will precede strains of sufficient magnitude to allow large volumetric expansion of the concrete to occur. Hence, for overall buckling failures, there is little confinement of the concrete and thus little additional strength gains.



The overall elastic buckling of the member characterizes the method of failure of long concrete filled steel tube columns.

The overall stiffness of the axially loaded CFST is mostly influenced by the steel tube, since the steel has a much higher modulus of elasticity than the concrete.

#### 2.2.4.2 Pure Bending (CFST Beams)

Concrete filled steel tubes subjected to pure bending behave much like hollow tubes. The tensile resistance of a CFST depends primarily on the steel alone. Therefore, moment resistance is highly influenced by the steel tube.

Pure bending tests of CFSTs indicated an increase in moment capacity due to concrete infill for the square and rectangular beams of between 10% and 35% as compared to hollow tubes (Brett et al 2001).

The stiffness of a CFST beam depends to some degree on whether or not bond exists at the interface of the two materials. In the absence of bond, there will be no interaction between the materials, and the composite stiffness will depend heavily on the stiffness of the steel tube.

The failure took place by local buckling in the compression flange of the tube, concrete crushing in the locally buckled area, and often yielding of the tube in tension. Beams containing high strength concrete or beam having high D/t ratios begin failing when the concrete fractures in shear after the steel has begun to yield. The concrete shearing is causes further stretching and then a subsequent rapture of the steel tube. Local buckling in the compression region also occurs near failure (Brett et al 2001).

### 2.2.4.3 Combined Axially Load and Bending (CFST Beam-Columns)

Typically, beam-column tests are different from eccentric loading tests by the magnitude of the induced moment and the type of failure. Beam-columns have moments of significantly larger magnitude. These moments may be introduced transversely, via loading of a connected member, or by a number of other methods.

Several key parameters influence the behaviour of beam-columns such as the D/t ratio, the L/D ratio (or the slenderness ratio), and the axial load ratio ($P/P_o$). With respect to material strength, beam-column tests on square CFSTs with high strength materials, there is reduction of initial stiffness and moment capacity with an increase in D/t ratio. The yield strength of the steel and the axial load ratio does not have any significant influence on the initial stiffness. However, when the yield strength of the steel increased, the moment capacity is enhanced.



Concrete-filled steel tube beam-columns typically perform better under cyclic loading than comparable hollow tubes and reinforced concrete members.

In the cyclic shear tests on square CFST columns with high strength materials, steel yielding in tension and concrete crushing occurred either prior to or at the same time as local buckling of the flanges, which took place near the peak load (Brett et al 2001).

### 2.2.4.4 Shear in CFSTs

The behaviour of a CFST member under a shear-type loading is dependent upon essentially the same parameters as beam-columns, including the D/t ratio, the axial load ratio, and the shear span ratio (a/D ratio). Based on the shear span ratio, shear behaviour can be divided into two types.

- Small shear span ratio (0.83-1.0)

  Diagonal shear cracking indicative of shear failure occurs in specimens that are also subjected to axial load.

- Large shear span ratio (2-3)

Columns exhibit a flexure-type failure with plastic hinges forming at the specimen ends.

Concrete-filled steel tube members subjected to shear forces display a large amount of energy dissipation and ductility. Circular members tend to have more ductility than rectangular tubes (Brett et al 2001).

#### 2.2.4.5 Torsion in CFSTs

As with shear, few tests of CFSTs under torsional loading have been done. The concrete filled steel tubes performed satisfactorily under torsional loading.

Torsional failure in a CFST is not distinct, but is characterized by a large increase in torsional rotation at a fairly constant load. The failure is due to a combination of spiral cracking in the concrete and tensile yielding of the steel.

## 2.3 Behaviour of Shear Strength of Composite Slabs and Development of Shear Strength Prediction Equations

This review will begin with the strength prediction equations developed for welded headed studs in solid slabs and will include more recent research on welded headed studs in slabs with formed metal deck.



Chinn (1965) performed ten solid slab push-out tests and two beam tests using lightweight and normal weight concrete. His tests, which were similar to those by Viest (1956), used 1/2, 5/8, 3/4, and 7/8 in. diameter studs. Stud lengths were approximately four times the diameter. Flanges of the steel beams were greased before the concrete was placed to reduce the effect of friction. It was found that, based on two tests, cycling the load did not affect the slips, i.e., a specimen could be loaded to failure without unloading. Referring to Figure 2.1, a “useful capacity” $Q_{uc}$ was found at the intersection of the straight-line lower part with the straight line projected backward, tangent to the upper part of the curve.

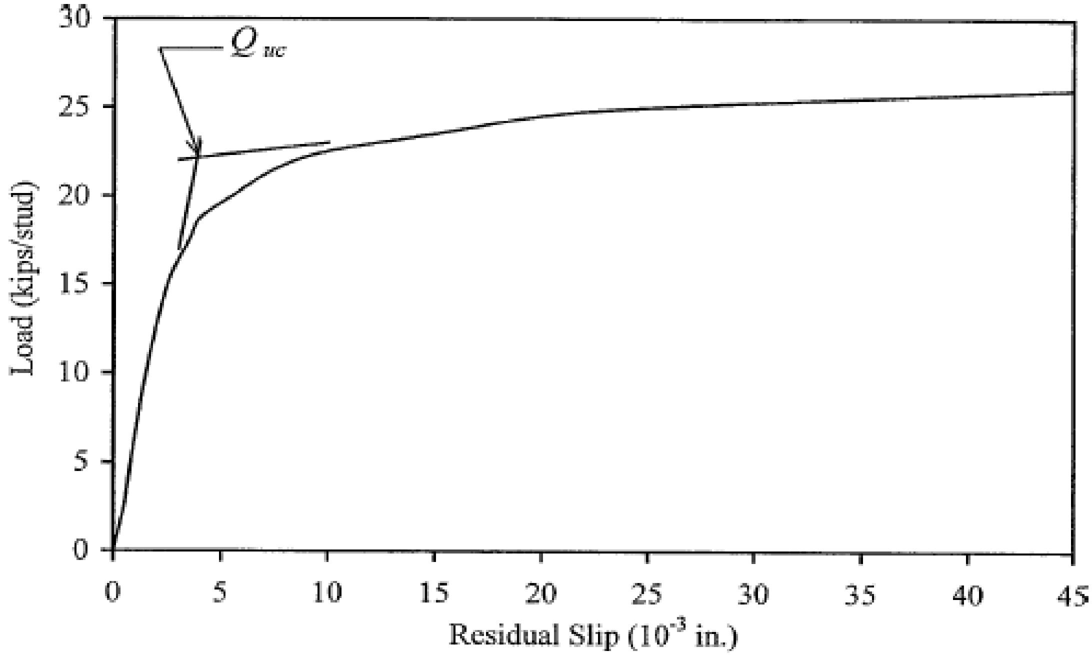


**Figure2.1 "Useful Stud Capacity," $Q_{uc}$, as determined by Chinn (1965)**



The useful capacity, which occurred at a slip of about 0.015 in., followed the relationship

$$Q_{uc} = 6.5d^2 f'c \sqrt{(4000/f'_c)} \qquad (2.1)$$

Where, $Q_u$ = useful stud capacity (kips)
d = diameter of stud (in.)
$f'_c$ = concrete strength (psi)

The failure mode was stud shearing in all specimens except the one with 7/8 in. diameter studs, which failed by slab cracking. The ultimate loads did not appear to be affected by the concrete strength, and followed the relationship,

$$Q_u = 39.22d^{1.766} \qquad (2.2)$$

Where $Q_u$ = ultimate stud capacity (kips)

Slutter and Driscoll (1965) studied the ultimate strength design of composite beams. Their tests were evaluated using simple elastic theory and the theory of incomplete interaction between the concrete slab and the steel beam. Twelve 15 ft simple span composite beams, one two-span continuous beam, and nine push-out specimens were tested. Two of the beams were constructed without shear connectors to investigate the effect of the natural bond between the steel and concrete. One of these two beams and a beam with bent studs were loaded from below the steel beam. According to Chinn (1965), this prevented friction forces at the beam-to-slab interface from being developed.

The push-out and beam test results were combined with results from two beam tests and 11 push-out tests from other sources to form an equation for the ultimate strength of stud shear connectors. This equation, which was developed for both bent and headed studs, is

$$Q_u = 930\, d_s^2 \sqrt{f'_c} \qquad (2.3)$$

Where $Q_u$ = ultimate strength of stud shear connector (kips)
$d_s$ = diameter of shear connector (in.)
$f'_c$ = concrete strength (psi)

This equation was deemed applicable for use with concrete strengths less than 4000 psi. Two beam tests without shear connectors and with identical parameters, except for the method of applying load, were conducted to investigate the role that bond and friction play in transmitting shear forces. Bond was not present in either test because concrete shrinkage caused bond failure. On one of the beams, the load was suspended from the steel beam so that only the concrete weight caused friction forces. The test was discontinued at a load of 20 kips because the member separated. In the other beam test without shear connectors, the load was applied on top. Friction forces were probably developed because the beam carried a load of 41.5 kips, giving it an ultimate moment of about 7% greater than its plastic moment. Tests on beams with shear connectors and loads suspended from the beam gave similar results when loads were applied on top of the beam. Roddenberry (2002) concluded that the ultimate strength of a shear connector is related to the ultimate flexural strength of a beam; and that shear connectors can be uniformly spaced if there are an adequate number of connectors because the shear connection loads are redistributed.



Ollgaard et al (1971) performed 48 solid slab push-out tests. Variables considered were concrete compressive strength, concrete split tensile strength, modulus of elasticity of concrete, density of concrete, stud diameter, type of aggregate, and number of connectors per slab. The stud tensile strength, slab reinforcement, and geometry were constant for all tests. Stud diameters tested were 5/8 in. and 3/4 in. Two types of normal weight concrete and three types of lightweight concrete were tested. The failure modes observed were stud shearing, concrete failure, and a combination of the two.

For the concrete failure mode, the lightweight concrete had more and larger cracks in the slabs than did the normal weight concrete. When one pair of connectors was in each slab, all failed by shearing off the studs. Specimens with one or two rows of studs per slab had the same strength per stud. The lightweight concrete tended to crush in front of the studs, causing the stud to remain straight when it deformed. When lightweight concrete was used, stud strengths decreased 15% to 25%.

The normal weight concrete provided greater restraint of the stud, so more curvature of the stud occurred. Studs in both types of concrete rotated a large amount at the weld. The tests showed that studs in both types of concrete exhibited considerable inelastic deformation before failure as the specimens did not fail suddenly at ultimate load.

However, the stud strength decreases when the concrete strength decreases considerably. The data indicated that the stud strength is more influenced by the concrete compressive strength and modulus of elasticity than by the concrete split tensile strength and density. They also concluded that the concrete properties control at ultimate load, so shear connector tensile strength is not as critical.

Roddenbary (2002) and Ollgard et al (1971) showed that the equation proposed by Slutter and Driscoll (1965) is not valid for different types of concrete. This equation over predicts the stud strengths obtained from tests. Roddenberry (2002) performed multiple regression analyses using logarithmic transformations. Fifteen models were tested, containing all possible combinations of the four concrete properties described previously as independent variables. The average shear strength divided by the cross-sectional area, $(Q_u/A_s)$, was the dependent variable. The equation below was shown to adequately represent the stud strength:

$$Q_u = 0.5\, A_s \sqrt{(f'_c\, E_c)} \tag{2.4}$$

Where $Q_u$ = ultimate stud strength (k)

$A_s$ = area of stud ($in^2$)

$f'_c$ = concrete compressive strength (psi)

$E_c$ = concrete modulus of elasticity (psi)



Reloading the specimen during testing caused the same overall load-slip behaviour as continuous loading caused. The load-slip curve for continuously loaded specimens, which includes the initial bond, was expressed as,

$$Q = Q_u\,(1-e^{-18\Delta})^{2/5} \tag{2.5}$$

Where, $Q$ = load (k)

$\Delta$ = slip (in.)

The load-slip equation for specimens that were reloaded is similar to the one suggested by Buttry (1965) and is,

$$Q = Q_u\,\{(80\Delta)/\,(1+80\Delta)\} \tag{2.6}$$

Roddenberry (2002) and Grant et al (1977) concluded that there was ductile shear connection which permitted redistribution of the slab force along the span of the beam. This resulted in a large ductility of the beam, and also concluded that rib height, rib width-to-height ratio, and embedment length should be included in a model to predict connector strength.

A modification to the equation developed by Fisher (1970) was made to include the height of the stud shear connector. The strength of the stud in the ribs of composite beams with formed steel deck can be expressed as

$$Q_{rib} = (0.85/\sqrt{N})\ [(H-h)/h]\ (w/h)\ Q_{SOL} \le Q_{SOL} \qquad (2.7)$$

Where, $Q_{rib}$ = strength of a stud in formed steel deck
$N$ = number of studs in a rib
$H$ = height of stud shear connector
$h$ = height of rib
$w$ = average rib width
$Q_{SOL}$ = strength of the stud shear connector in a flat soffit slab (Eqn. 2.4)

Johnson and Oehlers (1981) analyzed 125 push-out test results from 11 sources, performed 101 new push-out tests and four composite T-beam tests, and performed a parametric study. Based on a finite element model, the authors found that a weld collar less than 5 mm high attracts 70% of the total shear and reduces the bending moment at the base of the stud to one-third of the value found for a stud without a collar. As the height of the weld collar increases, the stud shank failure strength increases. Also, the distance that the resultant force on the stud acts from the base increases as the stiffness of the concrete decreases. This increases the bending moment at the base of the stud and decreases the stud shear strength. Based on the model, the authors found that voids in the concrete significantly reduce the stud strength. Further, stiff inclusions, such as reinforcing bars, significantly increase the stud strength when placed at the soffit of the slab but do not affect the strength when typical bottom cover is provided. They also attributed variations in push-out test results to variations in the degree of compaction of the concrete and position of the aggregate. They stated that in push-out tests the top of the stud shank is subjected to axial tension, but the bottom is subjected to compression due to the downward frictional force on the shank and weld collar. In beam tests, however, uplift may cause the axial forces to be tensile.



Hawkins and Mitchell (1984) performed 10 push-out tests under reversed cyclic loading and 13 tests under monotonic loading to study the seismic response of shear connectors. Thirteen of the tests used metal deck (1 1/2 in. and 3 in.), and ten were solid slabs. Variables included the type of loading, presence of ribbed metal deck, geometry of metal deck, and orientation of metal deck. The authors observed four failure modes: stud shearing, concrete pull-out, rib shearing, and rib punching. The studs that failed in stud shearing had large slips at failure, were very ductile, had stable hysteresis loops, and had large energy absorbing capabilities. The shear strength of the stud when subjected to reversed cyclic loads is 17% lower than when monotonically loaded. Staggering the studs or using large stud spacings increases the stud shear strength. Also, decreasing the concrete strength increases the slip. Studs may also fail in concrete pull-out due to a tensile force in the stud caused by large deformations. This type of failure is very brittle, and has a poor hysteretic response. It can cause a large decrease in strength and ductility compared to stud shearing failure. The reversed cyclic strength is 29% lower than the monotonic strength. The strength of the connection, based on three tests on tension specimens, can be expressed as,

For concrete pull-out failure

$$V_c = 5.4\sqrt{(f'_c\ A_c)} \qquad (2.8)$$

Where, $V_c$ = shear strength due to concrete pull-out (psi)
$f'_c$ = concrete compressive strength (psi)
$A_c$ = area of concrete pull-out failure surface ($in^2$)

Rib shearing failure usually occurs when studs are grouped together or the deck profile has narrow ribs or large rib heights. The strength and ductility are decreased significantly for this failure mode. Reversed cyclic loading causes S-shaped hysteresis loops and little energy absorption. Rib punching failure occurs when the concrete cover over the stud in the direction of the applied shear is limited.

Elkelish and Robinson (1986) studied six parameters that affect longitudinal cracking of composite beams with metal deck using experimental specimens as well as a finite element analysis. The parameters investigated were type of loading, concrete compressive strength, beam span-to-slab width ratio, thickness of the solid part of the slab, percentage of transverse reinforcement, and the existence of the metal deck. Three loading conditions were used: uniformly distributed load, single point load at mid-span, and two point loads at the third points. Twenty-four simply supported beams were used in the analysis. Six experimental beams were used to verify the analysis method.



The results showed that a uniformly distributed load causes the longitudinal crack to start at the top of the slab; single point load and two point loads cause the crack to start at the bottom. The initial longitudinal crack is delayed with an increase of the span-to-width ratio, the steel beam yield strength-to-concrete strength ratio, the thickness of the solid part of the slab, and the transverse reinforcement ratio. Welded wire mesh did not increase the resistance to initial longitudinal cracking. The metal deck helps to resist cracking when point loads are applied, but it does not help for a slab that is uniformly loaded.

Oehlers and Coughlan (1986) analyzed 116 push-out tests to derive the shear stiffness of shear stud connections in composite beams. The authors stated that the flexibility of the shear connection is important because it indirectly affects the flexural strength and fatigue life of the beam. The tests showed that studs in strong concrete are stiffer than studs in weaker concrete. The slip at failure is about one-third of the stud diameter. The tests showed that there is large permanent set even at low loads. The authors explained that this occurs because the weld collar embeds into the concrete; this is helped by the irregular shape of the collar which causes local crushing of the concrete. Near the ultimate load, when the stud fractures, the stud has little permanent deformation and the concrete is crushed next to the bearing surface of the weld collar. The authors concluded that at low loads, the amount that the stud embeds into the concrete is affected by voids or dense aggregate particles and the roughness of the weld collar. At high loads, the embedment depends more on the cube strength of the concrete.

Mottram and Johnson (1990) performed 35 composite slab push-out tests using three types of steel deck, with ribs placed only transverse to the steel beam, and using both lightweight and normal weight concrete. Studs used were 19 mm x 95 mm (0.75 in. x 3.75 in.) or 19 mm x 120 mm (0.75 in. x 4.75 in.).

The tests showed that failure occurred in the concrete ribs, not in the studs, with the strength being proportional to $f_{cu}^{0.27}$. A decrease in transverse spacing from 76 mm (3 in.) to 50 mm (2 in.) resulted in a 6% reduction in strength. The resistance per stud for two studs per rib was less than for one stud per rib. The resistance per stud for two studs placed diagonally was less than for an unfavorable stud; however, the maximum slip was greatly reduced. Two studs in line were stronger than two diagonally placed, even though the diagonal studs were further apart. The authors recommend that off-center studs be placed on the "favorable" side away from the midspan of the beam (referred to as the "strong position in the US"); tests done on unfavorable (referred to as the "weak position in the US") studs were 35% weaker than favorable studs. The "weaker effect" of an "unfavourable" stud was less of shallow deck. One stud per through had a slip capacity of 7mm or more; two studs per through had a smaller slip capacity, half less than 5mm. This is large different in ductility. The authors recommended accounting for this loss of slip capacity with increasing longitudinal shear resistance in the design of long spans with partial shear connection. Increasing the slab thickness was shown to increase the connection resistance. The authors compared the test result with the predicted values from equation below, which was being developed but was latter modified and published by Lawson (1992), and the equation from Grant et al (1977). The strength reduction factor, SRF, is multiplied by the equation for $Q_{SOL}$ (equation 2.4) to obtain the strength of a stud in a composite slab.



$$SRF = (0.75r/\sqrt{N_R})\,[H_s/(H_s + h_R)] \leq 1.0 \qquad (2.13)$$

Where, r = factor to account for position of stud in rib
for central or strong position studs, r is the lesser of $b_o/h_R$ and 2.0
for weak position studs, r is the least of $b_o/h_R$, $e/h_R + 1$, and 2.0
$N_R$ = number of studs per rib
$H_s$ = height of stud
$h_R$ = depth of deck
$b_o$ = average rib width
e = distance from center of stud to mid-height of deck web on loaded side

The equation above was found to be more consistent with test results than the equation proposed by Grant et al (1977). Unlike Grant's equation, this equation accounts for the position of the stud or studs in the rib.

Oehlers and Coughlam (1986) indicate that a longitudinal crack in front of a stud causes more compressive failure of the concrete, which can cause one of two results. If the "zone of compressive failure" is large, the stud head will rotate, causing tensile forces behind the stud head and thus tensile cracking of the concrete in a conical failure surface behind the stud. If the zone is small, the concrete compressive failure at the base of the stud and in front of the stud will simply increase the lever arm of the resultant force normal to the stud, causing more flexural forces on the stud and thus dowel failure at a reduced shear load. The author also argues that specifying the stud spacing will not always prevent concrete failure.

Lloyd and Wright (1990) performed 42 composite slab push-out tests. Variables included slab width, slab height, and the amount and position of reinforcement. They also studied the effect of applying transverse loading to the slab as well as the effect of sheeting-joint details on connection strength. Two types of deck were tested: deck without stiffeners with the studs welded centrally and a re-entrant profiled deck with stiffeners with the studs welded in the strong position. The stud size used was 19 mm x 100 mm (0.75 in. x 3.94 in.) and the slab was 115 mm (4.5 in.) thick and consisted of normal weight concrete. The slab width varied from 450 mm to 1350 mm (17.7 in. to 53.1 in.). The amount and position of reinforcement and the number of profile ribs were varied. Transverse moment was applied to some tests, until a longitudinal crack occurred, to simulate hogging action of a slab over a beam. Three different sheeting details were tested.

In almost all of the tests, surface cracks appeared along with separation of the concrete from the deck just before ultimate load was reached. After ultimate load, the slabs were seen to ride over the sheeting and cause extensive profile distortion. Wedge-shaped failure cones, not pyramidal-shaped cones as suggested by Hawkins and Mitchell (1984), occurred around the studs in all of the tests. This mechanism has been found to occur in a composite beam test. Some specimens also failed by rib shear. Tests with the deck parallel to the beam failed by longitudinal shear along the rib or by stud failure. The tests showed that the slip of the deck relative to the beam is half or less of the slip of the slab relative to the beam. Increasing the width of the specimen and varying the amount and position of reinforcement appear to have little effect on the connection strength. Applying a transverse moment to the specimen increased the ultimate strength “only marginally,” but caused high loads to be maintained, Long after ultimate load had been reached. Sheeting joints decreased the strength a small amount. The authors believe that a full rib of concrete should be provided beyond the connector position in order that the full strength of the connection is obtained. The authors developed expressions to predict the connection strength, for the total surface area of the wedge-shaped cone for cone failures.



For single studs,

$$A_{c\,(ss)} = 2w_1\sqrt{(w^2{}_1/4 + h^2{}_p)} + w_1\sqrt{(w^2{}_1 + 2h^2{}_p)} + 2w_2\sqrt{(3D^2{}_p)} \qquad (2.14)$$

For double studs,

$$A_{c\,(ds)} = A_{c\,(ss)} + 2s\sqrt{(w^2{}_1/4 + h^2{}_p)} \qquad (2.15)$$

These expressions are more sensitive to the deck and stud geometry than those by Hawkins and Mitchell (1984) (see Appendix for more details).

The connection resistance can then be found as

$$Q_K = 0.92(A_c \sqrt{f_{cu}})^{0.349} \quad (2.17)$$

Where, $f_{cu}$ = concrete strength

or for design,

$$Q_K = (A_c \sqrt{f_{cu}})^{0.34} \quad (2.18)$$

Sublett et al (1992) performed 36 push-out tests to determine the strength of studs in composite open web steel joists. Test parameters were base member thickness, deck rib geometry, slab thickness, stud position, and normal load application. The authors concluded that strength of concrete highly influences the stud ultimate strength: higher strength concrete may increase stud ultimate strength, and lower strength concrete may decrease stud ultimate strength. The stud strength may be assumed to vary approximately linearly with base member thickness. Studs in the strong position exhibited a larger stiffness than those in the weak position. The AISC strength predictions were un-conservative. Weak position studs had an average strength of 52% of predicted strength using AISC and the strong position studs showed 72% of predicted strength. The authors recommend the Mottram and Johnson (1990) method over the current AISC specification, or its results are 30% more conservative than AISC, perhaps because Mottram and Johnson included the stud location as a test parameter. The authors recommended for future tests, that the concrete strength should be varied to include low strengths to test the validity of Eurocode 4 (EN 2001) specification. They also recommended that the AISC specification should include stud position as a variable affecting stud strength. Also, studs used with thin flange sections, having thicknesses less than 2 1/2 times the stud diameter, should have a reduced strength.



Johnson and Yuan (1997) developed equations based on theoretical models for seven modes of failure. Results of over 300 push out tests and 34 new ones were used to determine the accuracy of the models. This method, unlike many of the methods specified in design codes around the world, considers the position of the stud within the rib of the metal deck. Five modes of failure are considered for transverse sheeting. They are shank shearing (SS), rib punching (RP), rib punching with shank shearing (RPSS), rib punching with concrete pull-out (RPCP), and concrete pull-out (CPT).

## 2.4 Behaviour of Shear Connector

Shear connectors can be classified as ductile or non-ductile. Ductile connectors are those with sufficient deformation capacity to justify the simplifying assumption of plastic behaviour of the shear connection in the structure considered. Shear-slip curves are obtained by push-out tests. Figure 2.2 shows examples of both ductile and non-ductile behaviour. A ductile connector has an elastic-plastic type of curve with a yield plateau corresponding to the connector characteristic resistance $P_{Rk}$ and to a high ultimate slip capacity $s_u$. Eurocode 4(2001*)* considers that connectors having a characteristic slip capacity higher or equal to 6 mm can be assumed to be ductile, provided that the degree of shear connection is sufficient for the spans of the beam being considered.

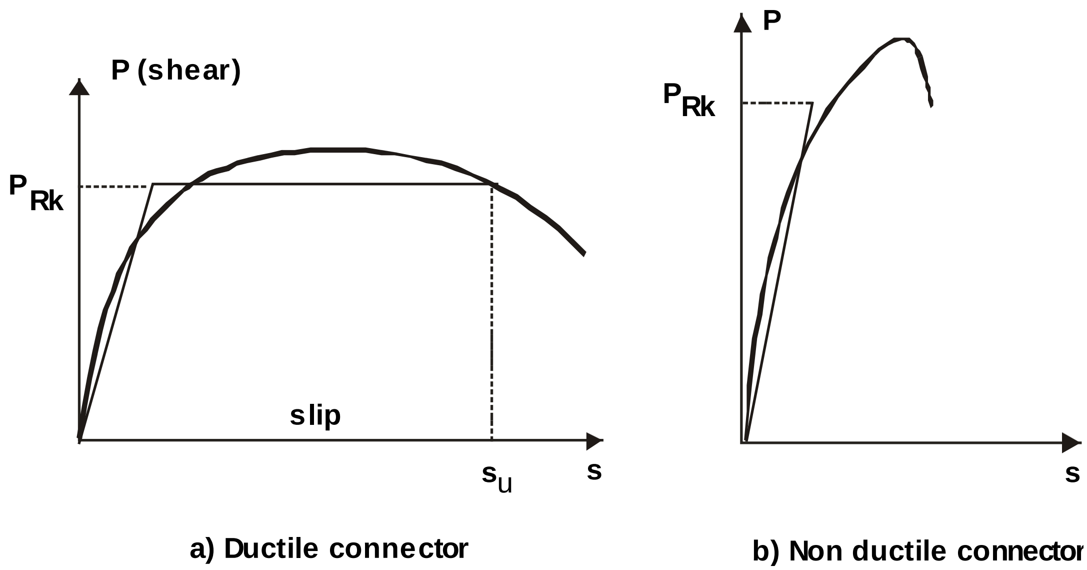


**Figure 2.2: Standard Connection Behaviour**



In brief, Conventional headed shear connectors' shear carrying capacity increase with concrete grade and concrete top cover. The strong position of headed studs has more shear resistance than weak position of studs. The common shear failure pattern of headed studs is cone failure. The existing prediction equation in Eurocode 4 (EN 2001) is not conservative and prediction equation does not influence by position of shear stud (strong/ weak), geometry of steel deck and concrete top cover.