SHEAR CONNECTORS

by

1.0 INTRODUCTION

The bond which must be achieved between the steel and concrete elements in the case of composite members, is a crucial issue. Where the two elements are in contact at their interface, they have to be tied together mechanically using what is called as "shear connectors". Basically these shear connectors are designed to resist (i) longitudinal shear forces at the steel/concrete interface and (ii) a tension which is caused by the tendency of separation of the steel/concrete elements at the interface. In the case of composite members, where one element encases the other element as in the case of columns and tubes etc, the two elements are tied together by interface forces induced by the geometry of the encasement and bond strength. The present chapter focuses on the mechanical forms of shear connection used in the steel/concrete composite construction.

2.0 TYPES OF SHEAR CONNECTORS

The common types of mechanical shear connectors are shown in Fig.1, out of which Fig.1(a) represents the 'stud shear connector' which is probably the most common type of mechanical shear connector used in practice. Stud shear connectors consist of a bolt (without thread) that is electrically welded to the steel member using an automatic welding procedure. The shank and the weld collar adjacent to the steel element are designed to resist the longitudinal shear load, whereas the head is designed to resist the tensile loads that are normal to the steel/concrete interface. The commonly used studs are of 19 mm (3/4") in diameter and have a shear strength of about 12 tonnes. These studs are attached directly to the flange prior to the casting of the concrete, through friction welding, using a stud-welding gun. As shown in Fig.1(b), these bolts can also be bolted to the flange, but this procedure needs holes to be made in the flange. Fig.1(c) shows a hand welded channel used as shear connector. Here, the longitudinal shear is resisted by the bottom flange of the channel whilst its top flange resists the tensile loads normal to the steel/concrete interface. Another type of shear connection is to explosively drive pins through a hook shaped mechanical connector as shown in Fig.1(d). The block connectors shown in Fig.1(e) are very stiff and strong in shear connection and the spiraled or hooped bars resist the normal tensile loads. The angle connector as shown in Fig.1(f) behaves more like the channel connection, but has a reinforcing bar welded to it to resist the debonding tensile forces. The types of shear connectors shown in Fig.1(g) & 1(h) are attached after the casting of the concrete.

There is a variety of mechanical shear connectors varying in shape, size and method of attachment. However, all of them have the following important similarities. They are basically steel dowels embedded in a concrete medium, they have a component that is designed to resist the longitudinal shear forces. Further, they have a component designed to resist the normal tensile forces and hence prevent separation at the steel/concrete interface. They all impart highly concentrated loads to the concrete elements.

The shear transfer in composite slabs are provided by using profiled sheeting, which could be termed as 'rib shear connectors' as shown in Fig (2). Their shapes are generally chosen as a compromise between enhancing the bond at the steel/concrete interface and enhancing the performance of the permanent shuttering to resist the construction load and any instability effects due to the wet concrete. In all the cases, the longitudinal shear is transmitted by the encased ribs. The bond performance of these ribs is improved by rolling indentations and protrusions into the rib, so that the longitudinal shear is also transmitted by mechanical action that is analogous to the transfer of shear in cracked reinforced concrete sections by aggregate interlock. The present chapter is mainly concerned with the former type of shear connectors.

3.0 MACRO - MECHANICAL BEHAVIOUR OF SHEAR CONNECTORS

A simply supported composite beam is shown in Fig.3. In the composite beam, the resultant of the flexural forces in the concrete element of the composite beam is compressive and the resultant forces in the steel element is tensile so that the cross-section of the beam can be considered to consist of both a steel and concrete element as in Fig.3(c). If steel and concrete elements were to independently bend, the concrete fibres adjacent to the steel/concrete interface would tend to expand under the flexural forces, whereas the steel fibres adjacent to the interface would contract under the flexural loads. The shear connectors tend to prevent this relative deformation at the interface. This relative deformation distorts the connectors, causing them to bear onto the concrete in the zones marked with asterisk. The connectors would, therefore, apply a thrust onto the concrete that is directed towards the midspan of the beam and are themselves subjected to horizontal shear forces. The flexural distortion of the composite beam also induce vertical separation between the steel and concrete elements and the tensile component of these forces is also resisted by the shear connectors.

3.1 Interface behaviour

The deformations, stress distribution and modes of failure of composite beams depend on the behaviour of the shear connection between the steel and concrete elements. This behaviour of this bond is represented by the relationship between the interface longitudinal shear-load and slip as shown in Fig.4. It could be seen from Fig.4 that the bond behaviour varies from extremely rigid and brittle as in curve A to extremely ductile as in curve B. It is also seen from Fig.4 that the L shaped rib connectors sustain a large amount of slip before failure. This is because the rib is fully encased by the concrete and so, can slide through the concrete without detaching and hence this form is well suited for composite construction. The stud, bolt and angle connectors in Fig.1 exhibit substantial plastic regions, but will fracture at a finite slip as in curve 'C' in Fig.4. This is because the slip capacity is now controlled by the deformation capacity of the connector as compared to profiled L-shaped ribs which simply slide through the concrete. Block connectors as shown in Fig.1 have very limited plastic regions as in curve A in Fig.4 and can, therefore, be considered as nonductile. All mechanical shear connectors have finite slip capacities and fracture of the connector, at this finite slip. Amount of slip as high as S_{ult} can cause premature failure of the composite beam if this limit slip capacity is not designed against.

3.2 Load carrying mechanism of shear connectors

Unlike profiled rib-connectors mechanical connectors tend to impose very high concentrations of load into the concrete element. This concentrated load is transferred from the steel element to the concrete element through the dowel action of the connectors. All mechanical connectors are simply steel dowels embedded in a concrete medium as shown in Fig.5(a). The resistance of a connector to this dowel action is referred to as the dowel strength and this strength is often quoted in different codes of practice. The concentrated load is dispersed into the concrete element and the action of this dispersal can induce tensile cracking as shown in the plan view of the concrete element in Fig.5(b). These tensile cracks are induced by ripping, shear and splitting actions. Tensile cracking can also be induced by the dowel action, particularly when the connector is also resisting separation at the steel/concrete interface of the component beam and these cracks which are conical in shape are referred to as 'embedment cracks', as shown in Fig.5(a). These four forms of tensile failure of the concrete as shown in fig.5 can affect both the dowel strength and ductility of the shear connection.

4.0 MICRO - MECHANICAL BEHAVIOUR OF SHEAR CONNECTORS

When flexural forces are applied to a composite member, slip is induced at the interface of the steel and concrete elements which is resisted by the dowel action of. the mechanical shear connectors as shown by the deformation of the connectors in Fig.3. It is, therefore, worth noting that mechanical shear connectors only resist the longitudinal shear after they slip, as compared to full interaction and rib connectors which can resist shear without interface slip.

The transfer of longitudinal shear by the dowel action of mechanical shear connectors is illustrated in an idealised and simplified form in Fig.6, where the stress resultants induced by the relative slip movement are shown. In this example, the steel dowel is trying to move to the right which causes it to bear onto the concrete at the right of the dowel. For the dowel action to work, the concrete adjacent to the bearing zone has to withstand compressive stresses of about seven times the cylinder strength of the concrete f_c and this could be achieved by the biaxial restraint imposed on this region by the steel element, the dowel and the surrounding concrete.

The resultant force in the bearing zone in Fig.6 is F, which occurs at an eccentricity 'e' from the steel/concrete interface. This force is in horizontal equilibrium with the shear force in the steel element as shown, and in order to maintain rotational equilibrium, a moment '$F \times e$ is induced at the base of the dowel. The steel dowel must, therefore, resist both flexural and shear forces, and these forces cause high tensile stresses in the steel failure zone.

The concrete can, therefore, crush in the bearing zone and the steel can fracture in the steel failure zone. It should be emphasized that this is a highly idealised model of a very complex stress distribution and mechanism of failure. For example, the interface frictional shear forces between the dowel and the concrete in the bearing area have been ignored. The mechanism of dowel failure is, therefore, governed by the interaction between the steel and concrete failure zones and can be described by considering the equivalent dowel mechanism shown in Fig.7.

The dowel behaviour can be considered to be analogous to a steel beam resting on a concrete medium where section a-a at midspan of the steel beam in Fig.7 is equivalent to section a-a at the steel/concrete interface in Fig.6. Consider the distribution of pressure at the steel/concrete interface in Fig.7 for a constant applied force 2F and for different configurations and material properties. When the steel modulus E_s approaches infinity, the pressure at the interface can be considered to be uniform and hence the resultant force F in one shear span will act at an eccentricity of $e=h/2$. The section at midspan has, therefore, to resist a shear force of 'F' and a moment $F \times h/2$. Conversely, when E_s approaches zero, the steel beam can be visualised as a layer of paper such that the resultant interface force in one shear span is now almost in line with the applied load so that 'e' tends to be zero, and so the midspan of the beam now only resists a shear load F. It can, therefore, be seen that as E_s increases relative to the concrete modulus E_c, the strength of the dowel reduces because the flexural component. Fxe at midspan increases.

The same argument holds good when E_c is changed relative to E_s. In this case, increasing E_c is equivalent to reducing E_s which reduces 'e' and consequently leads to a reduction in the flexural component Fxe and thus increase in dowel strength. In conclusion it can be expected that the dowel strength is proportional to the modular ratio parameter E_c / E_s, no matter what the shape and size of the steel dowel.

4.1 Failure mechanism of shear connector

Consider now the mechanism of failure of the dowel in Fig.6. It has been shown that the eccentricity depends on E_c/E_s. As the shear load F is increased, 'e' will remain constant, and the stress in both the concrete and steel failure zones will increase with F. If the concrete starts to fail before steel, this will reduce E_c and hence increase 'e', thereby causing an increase in flexural component 'Fxe· and increased flexural stresses at the steel failure zone. Further increase in 'F' will lead to further reduction in E_c and both the increase in 'F' and reduction in E_c will further increase the stress in the steel failure zone until the dowel cracks in the steel failure zone. Cracking in the dowel is equivalent to reducing E_s. This reduction in E_s leads to reduction in 'e', which consequently leads to an increased bearing pressure and hence further concrete failure. Further, concrete failure will lead to a reduced E_c and thus the cycle continues.

The description of the failure mechanism given previously helps to illustrate the strong interaction between the steel and concrete elements on the dowel action. Exactly the same mechanism of failure occurs when the steel starts to fail before concrete. Cracking in the dowel is equivalent to a reduced E_s which will lead to reduced 'e', which in turn will cause increased bearing pressure and eventually the failure of the concrete.

It is seen from the earlier discussions that the strength of the dowel shear connection is proportionate to the compressive strength of the concrete 'f_{ck}' as stronger concrete can withstand a large force F before it starts to fail and reduce in stiffness. The strength of the dowel connection will also be proportionate to the tensile strength of the steel dowel 'f_u' and cross sectional area of the dowel A_d. It has been already shown that the dowel strength depends on the parameter E_c/E_s. Therefore, it is to be expected that the dowel strength D_{max} is a function of the following parameters, that is

$$D_{max} = f\,(A_d, f_u, f_{ck}, E_c/E_s)$$

5.0 CHARACTERISTIC STRENGTH OF SHEAR CONNECTORS

Because of large variety of mechanical shear connectors and because of the complexity of the dowel action, the strength and ductility of shear connectors are always determined experimentally. It is very difficult to determine the behaviour of shear connectors through composite beam tests. This is because the connectors are loaded indirectly from flexural forces within the beam as indicated by the deformations in Fig.3. It is clear that the force on a connector is not directly proportionate to the load applied to the beam, but depends on the stiffness of various components of the composite beam. Instead of beam tests, the behaviour of mechanical shear connectors is determined from the so-called 'push tests' in which the connectors are loaded indirectly. The experimental setup and the shear connector specimen is shown in Fig.8. Fig.9 shows the trend of some of the result of the push tests of some shear connectors. The strength of shear connectors in composite beams depends on their ability to redistribute the shear load from heavily loaded to lightly loaded connectors, and this depends on the load slip characteristics of the shear connectors. Mechanical shear connectors can be described as brittle when their ability to resist load diminishes rapidly after their peak carrying capacity has been achieved, as shown in the 'brittle connectors' in Fig.9. The block shear connectors as shown in Fig.11 may be classified into this category. Alternatively when they can maintain their peak carrying capacity over large displacements they are referred to as ductile. A ductile connector has a large plastic plateau and a brittle connector has no plastic plateau. As stated above, the main properties of these load/slip characteristics are the plastic plateau and the stiffness 'k'. The load/slip characteristics are idealised as shown in Fig.10. It can be seen that these idealised characteristics form two groups, namely those based on the plastic plateau, and hence the ductility of the mechanical shear connector, and those based on the stiffness 'k'. Ultimate strength analyses is usually based on the plastic plateau and serviceability analyses are usually based on the stiffness 'k'. Ultimate flexural strength analysis are based on the assumption that the three material components-concrete, steel and dowell- are fully stressed. This condition is often referred to as 'rigid plastic' and allows the flexural strength to be determined purely from equilibrium equation.

6.0 STUD SHEAR CONNECTORS

The stud shear connection shown in Fig.1(a) is probably the most common type of mechanical shear connection and is certainly the most researched and understood type of shear connection. These connections can be welded to the steel elements using an automatic welding procedure. The most important dimension is the shank d_{sh} which varies from about 13mm to about 22mm and a very common size is 19mm. The diameter of the head is usually above 1.5 d_{sh} and height of the connector is about 4 d_{sh} in order to ensure that the connector does not pull out of the concrete element. The weld collar has a diameter of about 1.3 d_{sh} and height of about 0.3 d_{sh}. The weld collar at the shank/flange interface in Fig.12 increases the dowel strength by increasing the bearing surface at the stud/concrete interface and so reduces the stresses in the concrete for a given force 'F'. The weld collar also increases the dowel strength by raising the steel failure zone, so that the steel failure zone now occurs at the weld collar/shank interface. Raising the steel failure zone means the zone is subjected to a portion F2 of the shear load F. Furthermore eccentricity 'e2' of F2 in Fig.12 is less than the eccentricity of 'e' in Fig.6; so the weld collar also reduces the flexural stresses at the failure zone.

6.1 Detailing of shear studs

The concrete failure zone is confined to a small region in front of the stud. The minimum spacing of the connector in a composite beam is controlled by this concrete failure zone, because it is necessary to ensure that these zones do not overlap, otherwise the dowel strength of these connectors will be reduced. This is achieved through detailing rules in design which require that the longitudinal spacing of stud shear connector is greater than about 5 d_{sh} and that the lateral spacing is greater than about 4 d_{sh}. It is also necessary to ensure that the steel element to which the stud is welded is strong enough to resist dowel action and this is achieved by ensuring that the flange thickness is > 0.40 d_{sh}.

6.2 Strength of shear studs

The strength of stud shear connectors is always derived empirically from push tests. However the strength in composite beams that is required for design can be deduced from the strength of the push tests. Ollgard, Slutter and Fisher in 1971 pioneered the research on the dowel strength of stud shear connectors and identified the important parameters that control the dowel strength. Using statistical analyses, they derived an equation for determining the mean dowel strength of stud shear connectors in push specimens in which the concrete slab had not failed prematurely through splitting, shear or embedment. This equation can be represented by:

$$D_{max} = 1.83\, A_{sh}\, f_{ck}^{0.3}\, E_c^{0.44} \qquad (1)$$

where the units are in FPS system; A_{sh} is the cross sectional area of shank of the stud;E_c is the Young's modulus of concrete and f_{ck} is the characteristic strength of concrete. The exponents are changed to make the equation dimensionally correct as

$$D_{max} = 0.5\, A_{sh}\, f_{ck}\, E_c \qquad (2)$$

Using the above expression the dowel strength of stud shear connectors could be found out.

7.0 SUMMARY

In this chapter, the behaviour of mechanical shear connectors is described. The important aspect of load/slip behaviour of shear connectors is highlighted. The interaction of steel dowel and concrete and its mechanism of load carrying are also described. The behaviour and strength of shear stud have been emphasized in this chapter.

8.0 REFERENCES

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2) L.S. Geschwindner, Robert O. Disque and Reider Bjorhovde, "Load and Resistance Factor Design of Steel Structures", Prentice Hall, Englewood Cliffs, NJ (1994).

3) Trahair N.S. and Bradford M.A., "The Behaviour and Design of Steel Structures", 2nd Ed., Chapman and Hall, London, (1991).

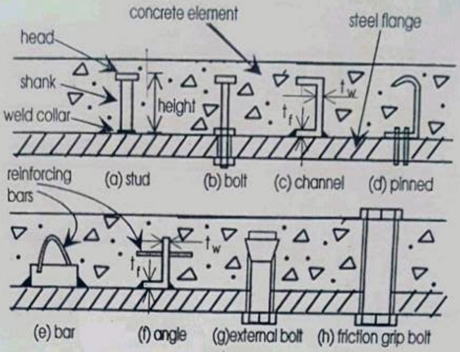


Fig.1 ***Types of mechanical shear connectors***

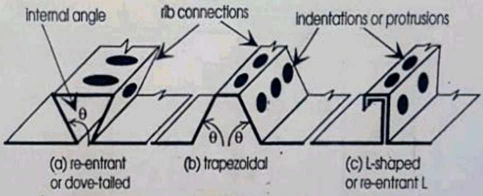


Fig.2 ***Some typical rib shear connectors***

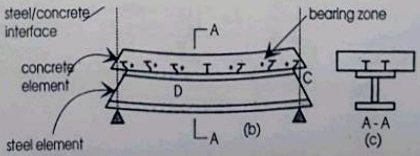


Fig.3 ***Relative Deformations at the steel/concrete interface***

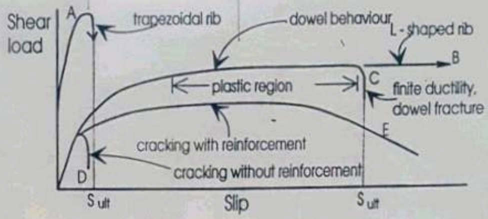


Fig.4 *Interface shear load/slip behaviour*

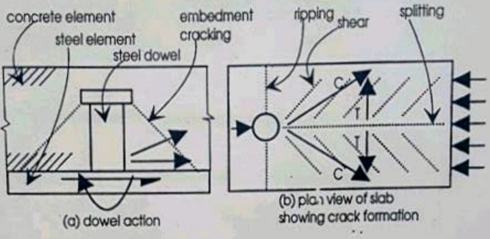


Fig.5 *Transfer of force at a shear connector*

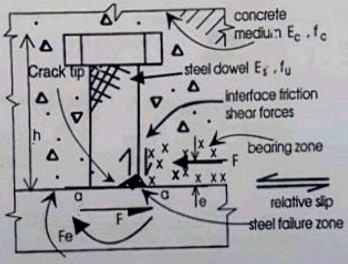


Fig.6 *Dowel Mechanism of shear studs*

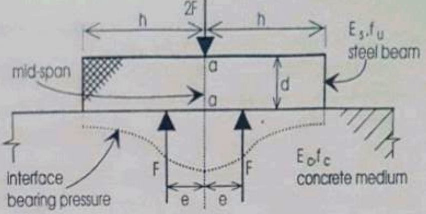


Fig.7 ***Equivalent dowel mechanism***

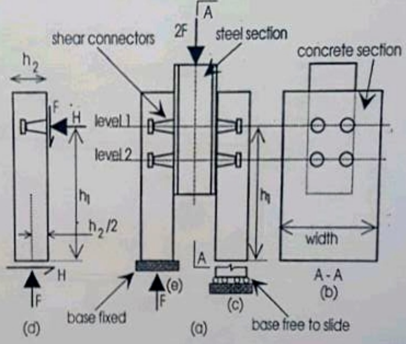


Fig.8 ***Push test and shear connector specimen***

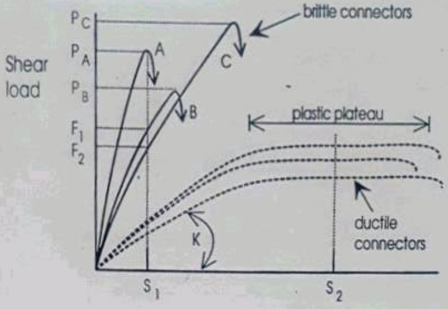


Fig.9 ***Load /slip characteristics***

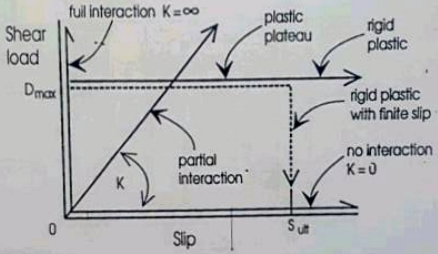


Fig.10 ***Idealised Load slip characteristics***

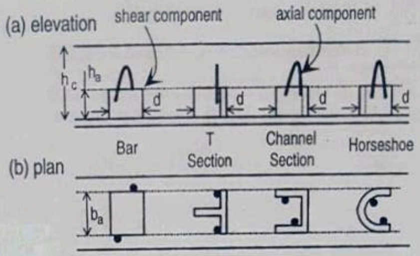


Fig.11 *Block shear connectors*

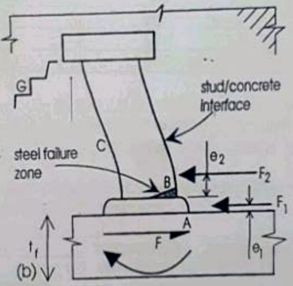


Fig.12 *Force distribution in a shear stud*

Appendix

A brief illustration of shear connector design

Design a shear connector for a composite bridge beam of 10 meter span. The section is shown below and the interface shear is given in Table 1

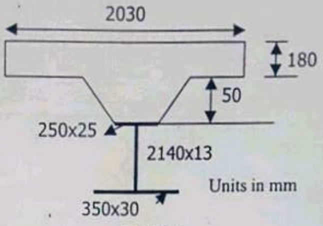


Table 1

Interface shear (kN/mm x 0.01)			
X (m)	DL	UD+KE	Total
0	4.88	15.2	20.1
5	3.62	12.9	16.5
10	2.46	11.1	13.5

DL –Dead Load
UD –Uniformly distributed load
KE –Knife edge load / line load

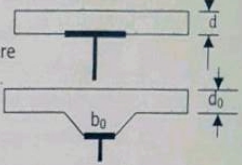
Table 2 Ultimate capacities of shear connectors for different concrete strengths

Types of Connectors		Connector material	Ultimate capacity (kN)		
Diameter (mm)	Height- (mm)		Concrete Strength (N/mm^{2})		
		Yield Stress 386 MPa	20	30	40
25	102		149	171	194
22	102		122	141	160
19	102		98	112	127
19	76		84	97	110
16	76		71	82	94
13	64		45	52	60
TEE with hoops 102x76x52 with13 mm hoops			253	290	326

Rigid TEE connector design:

Choose 100x76x50x9 mm thk TEE, 50 mm high to which 13 mm dia hoop is welded.

Allowable bearing stress:



Allowable bearing stress = $f = f_b(A/a)^{1/3}$ where

A= 2xdxd or b_0d_0

a -connector face area

f_b –unrestrained allowable bearing stress

Connector face area (a)=50x100=5000 mm^2

Area to which bearing transmitted(A) = 250x230=5.75x10^4 mm^2

A/a ratio = 5.75x10^4 mm^2 /5000 = >5.0 (A/a is restricted to 5)

Bearing Strength of Concrete M30 = 8.0 N/ mm^2

Allowable bearing stress f= 8.0(5.0)$^{1/3}$ =13.6 N/ mm^2

Working Capacity = (13.6x5000)/1000=68.0 kN

Spacing of the connectors = 68.0/(20.1x0.001) = 339 mm

The spacing must also be checked for longitudinal shear failure of the concrete.

Shear stud design:

Choose 19x102 mm studs

For M30 concrete the stud capacity=112 kN (From Table 2)

Use a safety factor of 4.0

The working capacity of the stud=112./4.=28kN

Using studs in groups of 3 we have the spacing

D = 3x28 /20.1x0.01=418 mm (spacing)

Check minimum spcaing

(a) min 610 mm

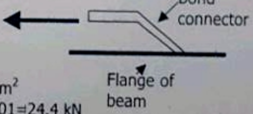
(b) 3xd_0 =3x180=540

(c) 4xheight of connector=4x102=408.

Hence 408 mm spacing governs. Alternatively using pair of studs

D=2x28./20.1x0.01=279 mm

Design of bond connector:



Choose 16 mm dia bar – area =246 mm^2

Allowable tensile stress in connector=140 N/mm^2

Allowable load on the connector=246x140x0.001=24.4 kN

Spacing = 34.4/20.1x0.01 =176 mm

This spacing must lie between 0.7-2.0 times depth of slab + haunch and hence

(i.e) 0.7x230 =161 mm and 2.0x230=460 mm

Hence the calculated spacing is acceptable.

Detailing aspect regarding shear connectors:

- A minimum centre to centre distance determined by the necessity to compact the slab concrete between them.
- A maximum centre to centre distance above which there is a possibility of local slab deformation.
- A minimum projection into slab to ensure adequate vertical tie down.
- A minimum distance between the connectors so that the bearing zones do not overlap.