Analysis of dry connections for precast concrete low-rise buildings

Alexander Maes

Supervisors: Prof. dr. ir. Luc Taerwe, Prof. Xiong Feng (Sichuan University)

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Chairman: Prof. dr. ir. Luc Taerwe
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Notification

"This thesis was an exam. The formulated remarks which were made during the defence were not included."

Preface

Before starting my thesis, I would like to thank everybody who made it possible for me making this thesis. Especially Prof. F. Xiong and Prof. L. Taerwe.

I would like to thank Sichuan University and the University of Ghent who made this exchange possible and were always available for advice and help during the stay in China. Furthermore I am very grateful towards the CSC which gave me this scholarship.

I also would like to thank other people outside the Universities which helped me mentally in this difficult exchange but also getting information for this thesis.

Alexander Maes

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Abstract

Nowadays many wet joint connections are used throughout the world. Next to wet joint connections there are dry joint connections. These are used less often due to the fact that the connections are more ductile and less strong as the wet joint connection.

China is strongly interested in the use of dry joint connections to build farmhouses or low-rise buildings on the rural parts of the country. For starters a good comparison will be made between the two concepts.

Since the dry joint connections are easier to apply and definitely will be used one day, a comparison between the theory, laboratory test and numerical analysis will be made.

The design equations from the theory will be obtained by doing a literature study in the European Codes. These equations will be compared with the results of the tested elements in the laboratory and the numerical analysis. Afterwards some conclusions will be made and some recommendations will be given which could make the connection more optimal.

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Used symbols

d	the nominal diameter of a bolt
d_0	the hole diameter for a bolt
e_1	the end distance from the centre of a fastener hole to the adjacent end of any part, measured in the direction of load transfer
e_2	the edge distance from the centre of the fastener hole to the adjacent edge of any part , measured at right angles to the direction of load transfer
l_{eff}	the effective length of fillet weld
p_1	the spacing between centres of fasteners in a line in the direction of load transfer
$p_{1,0}$	the spacing between centres of fastener in an outer line in the direction of load transfer
p_2	the spacing measured perpendicular to the load transfer direction between adjacent lines of fasteners
a	the base size of the throat section for fillet welds
A	the gross cross-section of bolt
A_s	the tensile stress area of the bolt or of the anchor bolt
A_{sw}	the area of shear reinforcement
f_{ck}	Characteristic strength of the concrete (differs from load to load)
f_{cd}	$f_{ck}/1,5$
$F_{v,Rd}$	the design shear resistance per bolt
$F_{b,Rd}$	the design bearing resistance per bolt
$F_{w,Ed}$	the design value of the weld force per unit length
$F_{w,Rd}$	the design weld resistance per unit length
f_v	the shear stress of bolt in Chinese code
$f_{vw.d}$	the design shear strength of the weld.
f_{ywd}	the steel quality of the shear reinforcement
f_{yk}	characteristic strength of the steel
f_{yd}	$f_{yk}/1,15$
β_W	the appropriate correlation factor taken from table 2.
$\sigma_{\perp}$	the normal stress perpendicular to the throat
$\sigma_{\parallel}$	the normal stress parallel to the axis of the weld
$\tau_{\perp}$	the shear stress (in the plane of the throat) perpendicular to the axis of the weld
$\tau_{\parallel}$	the shear stress (in the plane of the throat) parallel to the axis of the weld

Introduction

In China there is a demand for easy-to-build houses in rural areas. These rural areas are often hard to reach by heavy machinery to build these houses. That is why they would like to build houses which are easy to assemble by means of prefabricated concrete elements. These elements can not be connected to each other by wet joint connections because it requires heavy equipment for pouring the concrete. Another disadvantage is the long waiting time till the concrete is hardened.

Another option is a dry joint connection which will be more elaborated in this thesis. These connections exist out of steel bolts and plates which connect both elements. Another possibility is for example welded plates.

The main idea of the Chinese government is to design a catalogue with different houses. So a family can choose the house they like. These houses can be prefabricated at a certain plant over the whole country and transported to its desired place. Upon arrival all these parts can be assembled trough the dry joint connection.

1 Review connection methods [1]

In this chapter different connection methods will be outlined, with some examples. A global representation of a connection between two concrete elements will be given.

1.1 General

Connections are part of the most essential aspect in the prefabrication process. Their role is to connect single prefabricated elements towards a more coherent and rigid structure in order to bear all acting loads. These include shrinkage, creep, thermal deformations, fire and so on. To make a good design a lot of insight in the size and way the forces are acting is needed. These insights come from the vertical and horizontal loads which run through the construction. A good understanding is necessary between the connection and the whole structure.

Prefabricated connections require a certain criteria which are related with the design and the implementation. Their most important function is to transfer the forces through the connection so the interaction between both prefabricated elements is achieved. This interaction can have different purposes:

- Connecting the elements with the bearing structure;
- Caring for the intended operation of subsystems such as rigid floor diaphragm, shear force function of walls or tubes;
- Transferring the forces from their starting point to the stability components.

Other aspects can lead to specific demands, for example towards the sustainability and looks of the connections. The details require to a lot of demands which are applied to the production, transport and assembly of the elements.

Designing the connections is not only making the right choice of the appropriate connection means. The connection has to be fully checked (the joints, joint sealants, edges and transition zones in the prefabricated elements). These zones make sure the force transmission occurs from the connection to the construction element. They have to be detailed and reinforced to take up the acting forces and possible deformations.

1.2 Design criteria

Design of constructive connections in prefabricated buildings have to reckon with certain criteria which are related to their structural behaviour, dimensional tolerances, fire resistance, fabrication, manipulation and assembly. The most important are described below.

1.2.1 Structural behaviour

- ***Strength***

Connections have to withstand to the forces which are acting on top of them for the rest of their life. Some of these forces are caused by their own weight, other gravitational loads are: wind, earthquakes and ground or water pressure. Other forces are present because of blockages and deformations of the elements. Additional forces are those that act of unforeseen obliquity of load bearing columns and walls or walls with unforeseen eccentricities.

When designing the connections, accidental connections have to be accounted for too. Forces can arise in the connections due to explosions, collisions etc. When these accidental forces inflict great damage to a building, the connetions will try to transfer the forces by redistributing the forces or by an alternative load bearing way. The connections have to make these transformations possible because it is an essential part of the construction. In these situations the design does not only need a good force transfer but also have sufficient deformability and ductility.

- ***Influence of volume changes***

The combined effect of creep, shrinkage and temperature fluctuations can cause tensile forces in prefabricated elements and their connections. Two methods exist to account the volume changes. The first method can make the deformations possible in-situ, the other method is to strengthen the connections to prevent these. In the last method, the connections must be able to take up the blockage forces. In practice a solution between both methods is possible. When a relative deformation is possible, for instance because of elastic deformation of the construction element and the connection details, the blockage forces will diminish. The same effect can be achieved by a partial movability. Not only the force transfer capacity of the connections has to be checked but also the possible consequences of the partial displacement of the forces and the deformability of the connection.

- ***Deformations***

Connections may have no obstacles for some deformations which are inherent to prefabricated constructions. These are usual deformations of beams and floors due to pretension and useful loads. A typical example is the case of a vertical element which is connected to a beam or floor on a certain distant from the support. When the connection impedes the vertical movement of the beam or floor, damage can arise in the connection itself or in adjacent elements. Even when no damage occurs, undesired forces can arise in the elements and cause harmful deformations. The solution is to provide the connection of a rolling support, or design it as a hinge.

- ***Ductility***

It is always advisable to design a connection and detail it so a brittle fracture is prevented when overpressure occurs. For instance when acting forces are being underestimated. Ductile behaviour of a connection is desirable. Ductility means that plastic deformations may arise without any substantial loss of force transfer capacity. Ductility is expressed by a ductility factor, that gives the relation between deformations in its ultimate limit state and deformations at the end of the elastic phase.

Ductility may not be confused with deformability, nor the transmission of bending moments. In case of overpressure the ductile connection will reach the yield strength and plastic deformation will occur. The plastic deformation will lower the blockage forces and a new equilibrium will arise. Large deformations arise, but the possibility to bear the loads will still be present and a brittle fracture with damage in the adjacent connection zones will be prevented. The large deformations also show a warning that something goes wrong.

To assure ductile behaviour of a connection a balanced design has to be applied. The principle is illustrated in Figure 1 by means of a connection which transfers mainly the tensile forces

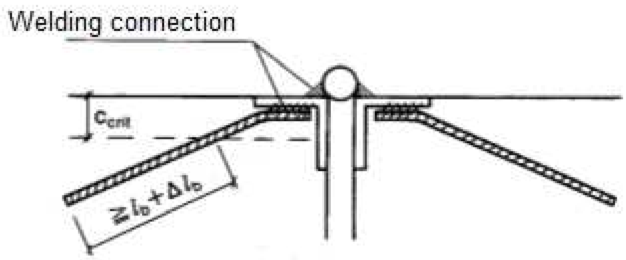


Figure 1: Principle of a balanced design

C_{crit} is the minimal concrete coverage which is necessary to anchor the rod without bursting the concrete

The connection exists out of different parts, which can be considered as links in a chain. The anchorage bars are the link with the largest contribution to the total plastic deformation of the connection. Balancing this connection holds that the full plastic deformation of the anchorage bars is achieved before the connection breaks. Premature brittle fractures in other links have to be prevented. This means that all parts in the connection, also the anchorage of the reinforcement bars in the element, the L-profiles, the steel rods and welds are designed so they do not only exceed the yield capacity but also the ultimate strength of the anchorage bars.

- ***Sustainablitiy***

Sustainability depends on the risk of corrosion of the reinforced steel and of the cracks which burst off the concrete, where especially the environment have to be accounted for. Steel which is exposed in an aggressive environment needs a permanent protection. This can be a layer of epoxy paint, a corrosion resistant paint, bitumen or encasement the bar in concrete. In many cases the connections can not be inspected or maintained because the building is finished. In those cases the life expectancy of the connections has to be larger than the construction. When exposed steel can not be maintained, corrosion free steel has to be used. There is a risk galvanic corrosion occurs when two different metals come into contact with each other. Galvanic corrosion occurs when two metals with different alloys have electric contact by means of an electrolyte as water.

1.2.2 Dimensional tolerances

Dimensional deviations are inevitable when constructing a prefabricated structure and the fabrication of the elements. Serious problems have to be accounted for and prevented when designing the connections for assembly. A typical example is the bearing length of the prefabricated floors. Both the length of the floor element and the support will deviate from the original plans. The deviations are the most sensitive at the supports. In this case they should be controlled by changing the bearing length and the use of bearing materials.

Another principle when looking at the dimensional deviations and tolerances is that all connections, from all types, should be able to change in all three dimensions so the elements can be placed at the right level. This is shown in Figure 2.

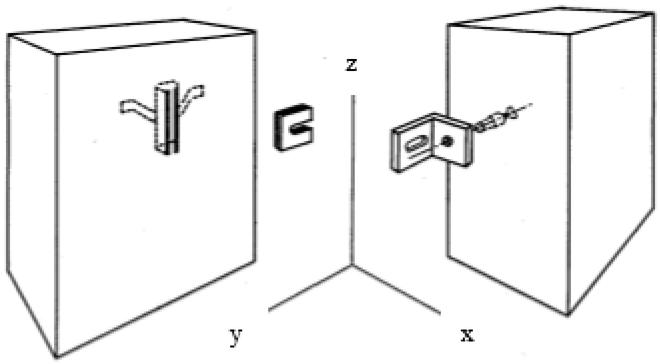


Figure 2: Connections have to be adjustable in all three dimensions

In the example given by Figure 2 the adjustments in the z-direction can be done with a vertical anchor rail. The x-direction can be changed with plates between the L-profile, the anchor rail and the y-direction can be changed by the oval crevice in the L-profile.

1.2.3 Fire resistance

Two aspects have to be examined when designing connections with regards to fire exposure. First there is the effect of fire on the strength of the connection, next the flashover through the connections should be prevented. When connection details are directly exposed to fire, the force transfer can be lowered because of the high temperatures. That is why the vital connection details should be protected like every other construction element. This protection can be achieved by encasing it by concrete or by using fire protecting materials. Steel connection parts which are partly encased with concrete will heat up slower than not encased steel because of the thermal conductivity of the surrounding concrete.

Many connections in prefabricated concrete are not affected by fire and do not demand special treatment. No special precautions have to be taken at supports between floors on beams or beams on columns. When these connections exist out of elastomeric materials or even flammable materials, the fire will only affect a small part of the suspension material, and above all no complete agitation will lead to large deformations or collapses.

In case of fire, the walls and floors have an important protection function when looking at thermic isolation and flashover. Connections between walls and floors should be designed to prevent passage of flames or hot gasses.

1.3 Wet joint connection[2]

A wet joint connection is a connection between two concrete element by applying reinforcement bars or other constructive provisions and pouring concrete at the construction site. Figure 3 shows reinforcement bars sticking out of the structure which are waiting when the next section of the concrete element is poured.

Figure 3: Waiting reinforcement bars (wet joint connection)

Despite a wet joint connection (or wet node) is more labour intensive and more expensive than a dry joint connection (or dry node). It is has the advantage that it is much stronger because after hardening it is one entirety with both of the prefab elements. When connecting the floor of the basement with a wall of the basement, the wet joint connection offers a good resistance against the water flow. Another advantage is that the structure is one massive part (monolithic). A part of the advantages of prefab elements are lost because of this connection because of the time it needs to achieve its complete strength.

Depending on the assumed demands, shape and size of the joints, they are being carried out in concrete or mortar. More often factory made dry premixed shrinkage free mortar is used.

The specification of the mortar has to suffice to the mortar type, strength class and environment class. The mortar types are distinguished in grout, mortar screed and bedding mortar.

In Europe the strength class are being expressed in K-values. This is the characteristic mortar strength determined on prismatic cubes of 40x40x160 after days of hardening under controlled conditions. The environment class is being expressed as described in NEN 8005. Because of the low water-cement ratio's (<0,45), factory made shrinkage free mortar always suffices to the necessary environment class.

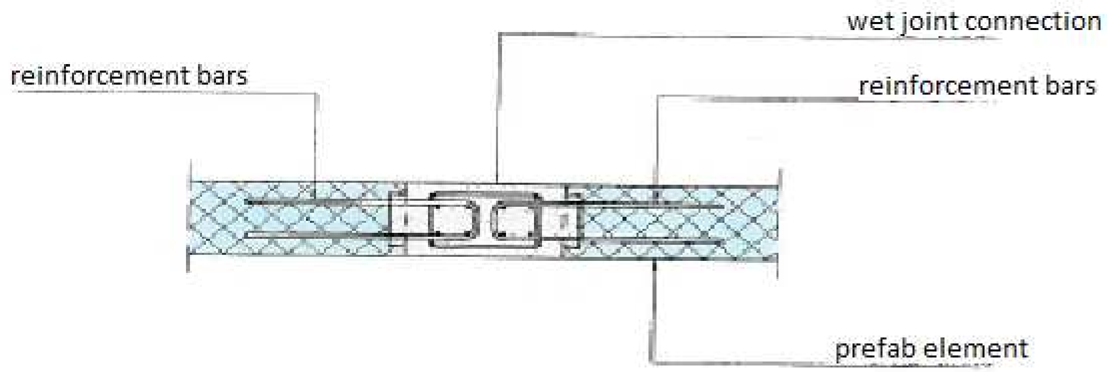


Figure 4: Detail of wet joint connection

1.4 Dry joint connection[2]

A dry joint connection which is made at the construction site is a connection between two prefabricated elements or one prefabricated element and a casted in situ concreter member. The elements are free supported by means of cams which are connected, welded or bolted. Figure 5 shows a dry joint connection.

Figure 5: Dry joint connection

Dry joint connection are divided in two groups:

- Free support, coupled with cams
- Welded or bolted connection

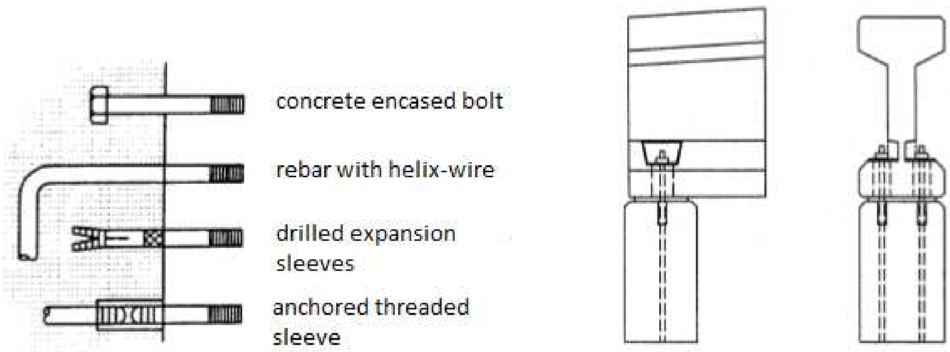
The advantage of dry joint connections is that they are less labour intensive and connected directly to strength. This makes the dry joint connection also cheaper. This is the opposite of a wet joint connection because the mortar which is applied to rely on its strength has to dry first. A wet joint connection is stronger than a dry joint connection because the mortar and makes the whole construction monolithic.

Next a difference can be made between bolted and welded connections, these two connections are used in the laboratory for research.

1.4.1 Bolted connections

Bolted connections can be used to transfer tensile and compression forces. There are different ways to realise bolted connections. See Figure 6.

- Encapsulated bolts with wire at the end
- Encased anchor rails with or without anchorage (depends on size of the force)
- Anchored threaded sleeves
- Drilled expansion sleeves



Example of a bolted anchorage

Example of a bolted connection

Figure 6: Bolted connections

The dimensional tolerances are taken in the oversizing of the holes in the elements which have to be connected

1.4.2 Welded connections

Welding can be used to connect steel parts with each other, for example waiting bars that overlap. See Figure 7a. A metal spacer can be used as an alternative solution to connect two concrete elements. The spacer can be welded to the protruding metal (Figure 7b) or anchor plates/L-profiles which are encased in the concrete at the height of the element. See Figure 7c . The anchor plates are connected to the concrete elements by means of welded dowels that act on adhesive or through the end anchors based on the principles set out above.

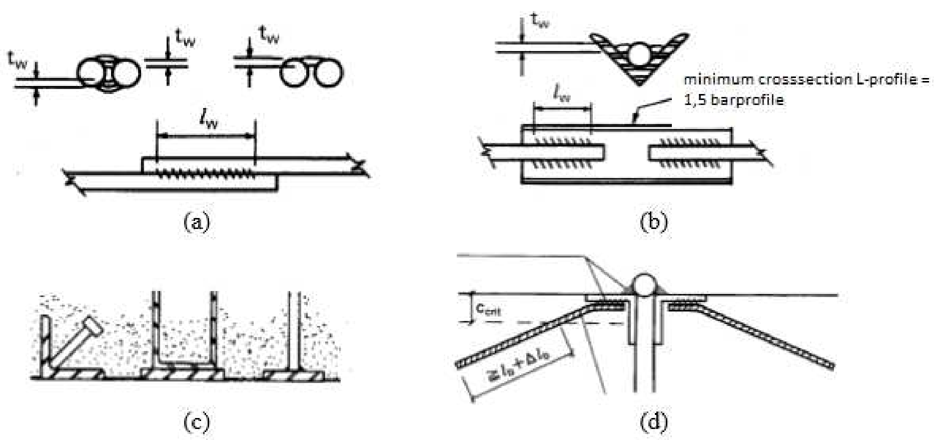


Figure 7: Welded connections

- ***Type of welded connections***
  - Fillet weld

After applying the fillet weld a small gap remains between both connected pieces. Depending on the direction of the acting force a difference is made between the type of fillet welds. We speak of a head fillet weld or a side fillet weld. There is no difference between the two welds. The fillet weld will be discussed in more detail in the next chapter.

  - Butt weld

There is no gap between the two connected pieces for a butt weld. The two pieces can lie in each other elongation or the weld can connect two pieces which for an angle. The pieces which have to be connected are prepared before the welding is done. A difference in the preparation is made. U- and V-welds do exist.

  - Plug welds

Plug welds may be used to transmit shear, to prevent the buckling or separation of lapped parts or to inter connect the components to built-up members. But should not be used to resist externally applied tension

1.5 Other type of connections

1.5.1 Containment

A connection can be realised by shoving the element in the other element and filling the surrounding space with concrete or glue. The last hole is not used that much in prefabricated concrete. A classic example of a containment is the connection between a prefabricated column and a well foundation. Another example is the containment of a massive steel profile, which is used as a hidden console, in a column. The space between the hole and the profile is filled with epoxy glue.

1.5.2 Mechanic interlocking

Shear forces can be transmitted by toothed joints. The tooth’s work as a mechanic interlocking. To actually use these joints, they may not open up when the loads are present. Chains are being used at the top and bottom of the elements. Another solution is to use transverse reinforcement bars of the whole length of the joint.

1.5.3 Skid steers

Shear forces can be transmitted because of skid steers and vertical joints by walls. For mechanic interlocking the shear forces are transmitted by pressure struts. For skid steers these forces are transmitted from one element to the other one.

1.5.4 Posttensioning

Posttensioning is used for segment buildings and sometimes in walls of high rise structures. The cables are being applied inside the elements and after assembly the cables are posttensioned. The joints between the elements should be able to take up these tensile and shear forces.

1.6 Transfer of forces

Dry joint connections, which are being used to make these farmhouses, mostly transfer tensile or shear forces. In next paragraph this transfer will be discussed. For the sake of completeness the transfer of compression forces will be explained too.

1.6.1 Tensile force transfer

Tensile forces between concrete elements are being transferred by different kind of steel connections: overlap of waiting bars, dowels, bolts, welds and mechanic connections.

The size of the tensile force which goes through the connection depends on the strength and cross-section of the parts and/or the capacity of the anchorage capacity. This can be obtained by the adhesive along the anchorage bars or by means of different types of end anchorages.

Anchorage by overlap is often used to connect prefabricated elements. The waiting bars from the element are being encased in concrete after the assembly. See Figure 8. End anchorages happen with anchorage heads, bended bars, hooks or similar details. The force transfer is obtained by overlapping with the reinforced bars in the element. Sometimes this happens in combination of dowels, see , or by other means.

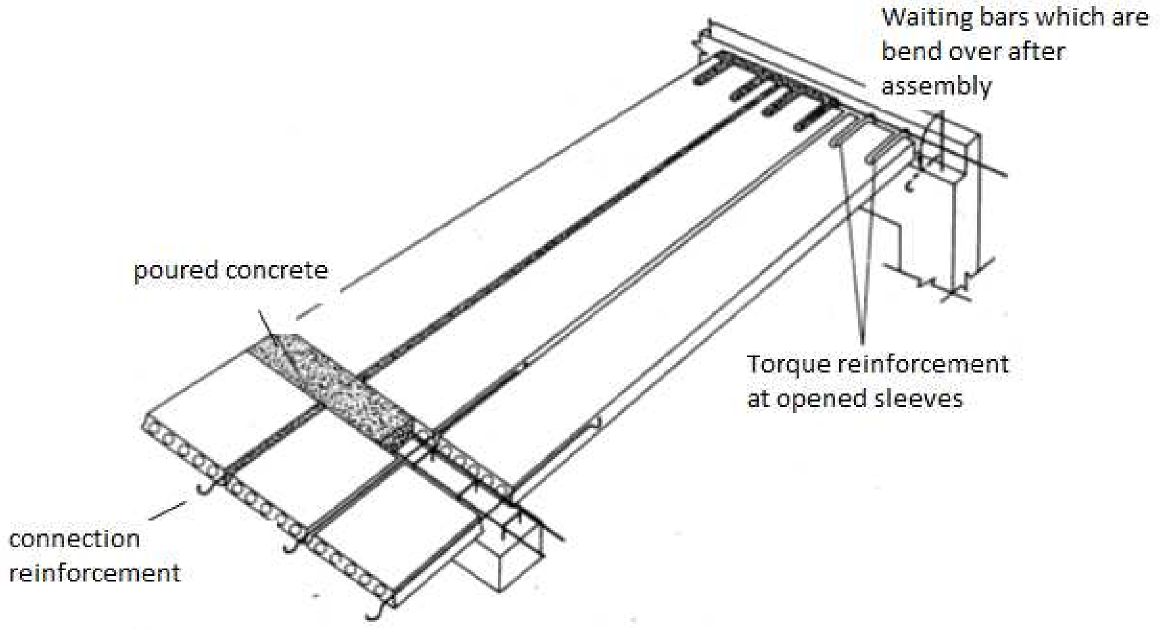


Figure 8: Tensile connections with waiting bars and other reinforcements

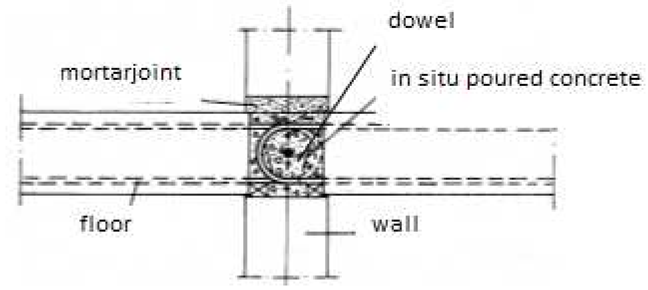


Figure 9: Tensile connection by overlapping in combination with dowels

Figure 10 gives an example of a facade element that is connected with the floor by a bolted connection. The necessary tolerances can be achieved by using anchor rails, oval holes in the L-profiles and plates between the elements which have to be connected.

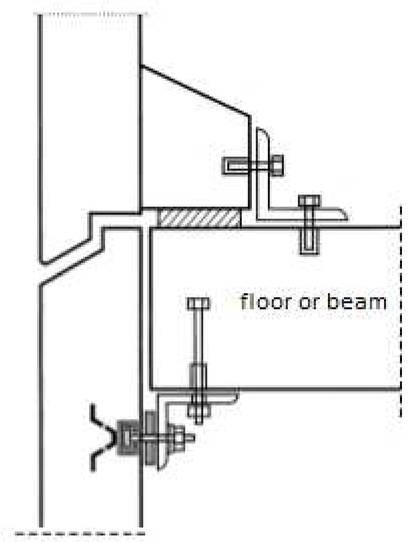


Figure 10: Bolted tensile and compression connection

Figure 11 gives the same example as Figure 10 , but with a welded connection. A steel L-profile is begin welded to the anchorage plate in the façade element and a L-profile which is anchored in the beam of the floor. A small crevice is recommended next to the encased steel plate so concrete which will jump off because of thermal displacements of the plate because of the weld can be avoided

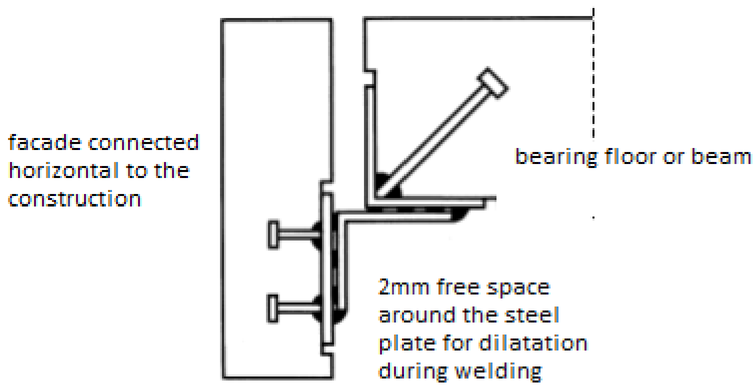


Figure 11: Welded tensile and compression connection

As a last remark this can be said. The force transfer between two elements can also be done by shear. In this paper this is neglected due to the fact the elements won't touch each other during the lab sessions.

1.6.2 Compression force transfer

Compression forces can be transferred through connecting elements (immediate contact), mortar joints or other suspension material.

When compression force connections are used, there is a chance that uneven contact surfaces give peak stresses in contact zones, eccentricities in the seizing points of the forces and torsion effects. See Figure 12.

Direct contact between elements without suspension material can only be used when a large accuracy can be achieved during fabrication, and when the compression stresses are small (≤0,3 f_{cd}).

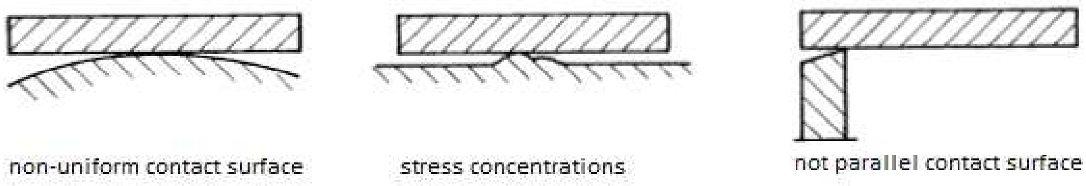


Figure 12: Examples of non-uniform contact surfaces for suspensions

The irregularities in the joint surfaces can be diminished by mortar or fine concrete. This method is often used in joints between bearing elements, like columns and walls and sometimes between floors and beams. But seldom underneath beams. The normal joint thickness is 10 to 30 mm for mortar and 30 to 50 mm for fine concrete.

Elastic suspension material like neoprene can absorb the irregularities of the surface and divide the contact stresses. They are often used at floor and beam supports. The thickness of the layers can differ from 2 to 10 mm. The greater thicknesses are used to allow longitudinal deformations and rotations and so to avoid stress concentrations near the connections. Neoprene can deform to 45°. When the necessary thickness is too large (more than 10 mm), or at loads above 0,7 N/mm², they will make the neoprene suspension of different layers with in between each layer a thin plate in stainless steel. This are 'chipped off suspensions'. These suspension are available everywhere and are used for bridge girders.

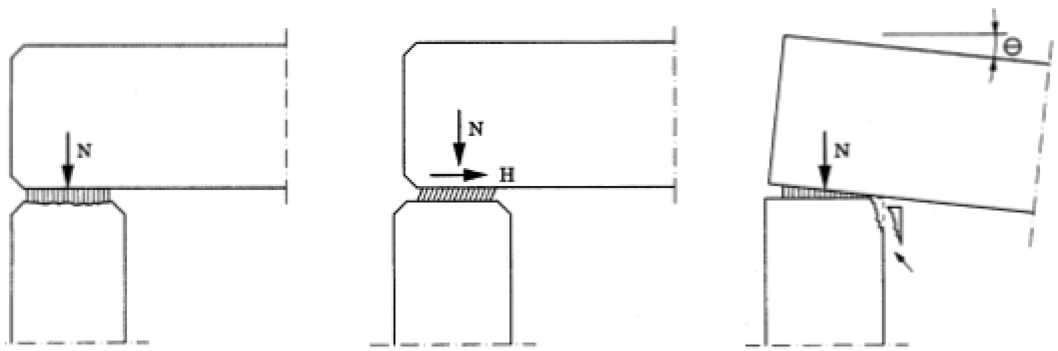


Figure 13: Loading conditions for elastomeric suspensions.

The suspension material has to be placed on a certain distance of the edge of the suspension construction to avoid the concrete from bursting. After all, the beam and floor have to be able to bend without having any connection between the element and the suspension edge.

Hard suspension materials can be used for large loads or for welded connections between the element and the support construction.

Compression connections can lead to important tensile stresses in adjacent elements. A classic example is the connection between two columns which are placed on top of each other. When the elasticity modulus of the joint material is at least similar to 70% of the elements (this is a hard suspension), transverse tension stresses will arise in the adjacent elements on a small distance from the suspension (see Figure 14a). When the elasticity modulus of the suspension material is a lot smaller than the connection elements (soft suspension) splitting stress will arise at the end of the elements, due to transverse deformations of the suspension material (see Figure 14b). The transverse stresses in the joint have to be taken by the adjusted reinforcement bars in the end zones of the connecting elements.

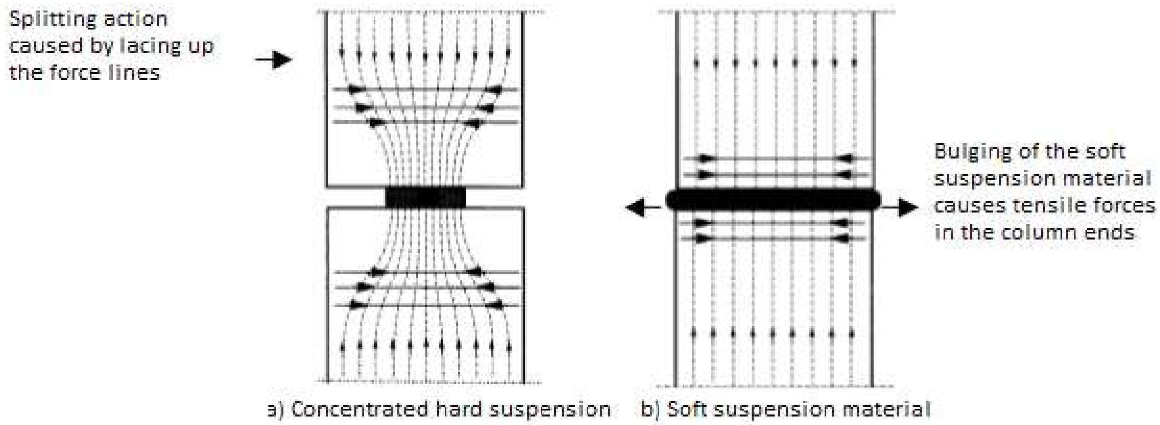


Figure 14: Transverse tensile stresses at compression connections

Mortar joints and joint fillings with fine concrete between walls or columns are hard suspensions. In principle the bearing forces of these connections have to be decided by the strength of the adjacent elements and not by the joint material.[3]

1.6.3 Shear force transfer

Shear forces between adjacent concrete elements can be transferred by adhesives, friction in the joint edges, interlocking, dowel action because of transverse reinforcement bars or special mechanical connections. Examples of these connections which take up shear forces are shown in Figure 15.

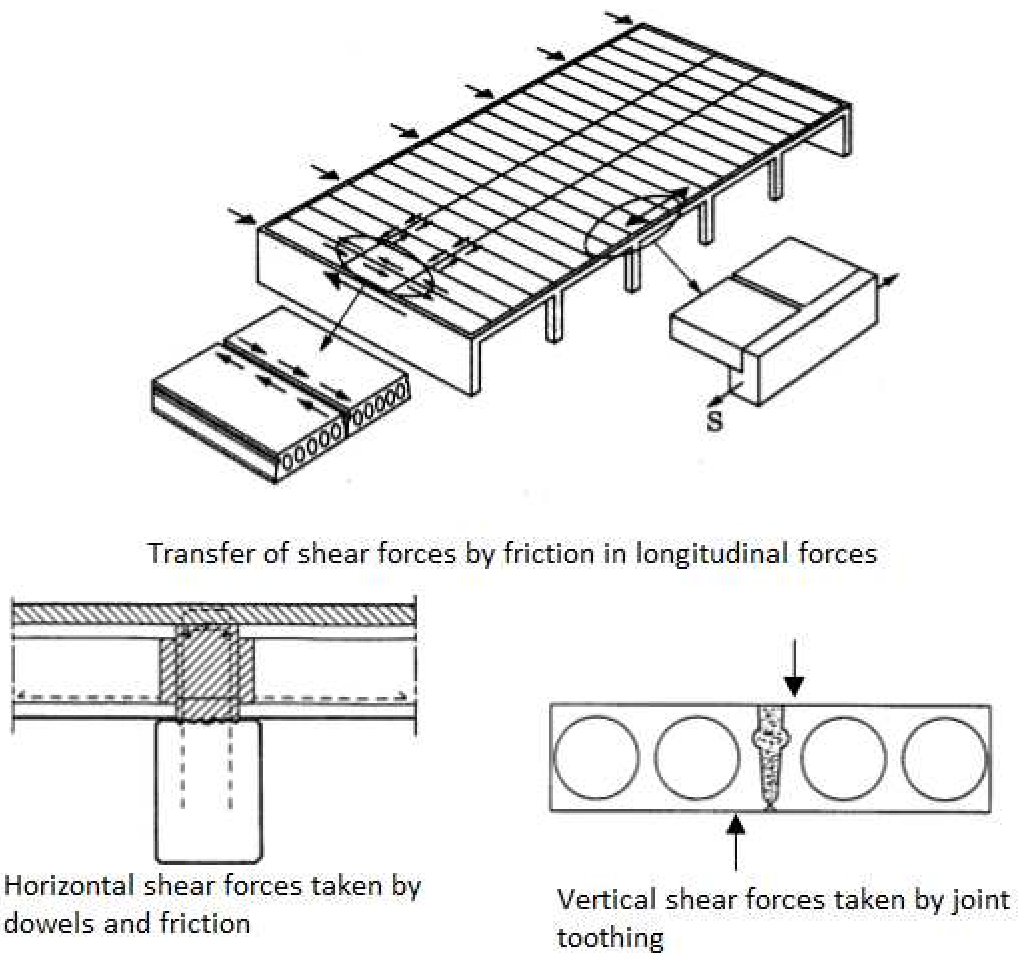


Figure 15: Examples of connections which take shear forces

2 Equation to design[4][5]

In this chapter several equations will be summed so the forces can be transferred between two elements. Both bolted and welded connections will be discussed.

2.1 Bolted connections

For the bolted connections, which are connections in tension and shear, multiple equations have to be fulfilled. Is the shear strength and bearing strength of the bolts high enough to transfer the strength? Another one checks if the plate which connects the elements is strong enough. How are the forces transferred to each bolt. Note it is not known for which force the concrete was designed. When comparing the test results a conclusion will be made.

The bolted connections during the tests are always assumed to be loaded under double shear. A partial safety factor is present in the formulae ($\gamma_{M2} = 1{,}25$). In Figure 16 all bolt classes can be checked. The bolts used during the test is of class 4.8 and not conform the Eurocode. That is why the Chinese codes are mentioned as well with its bolt characteristics.

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

Figure 16: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

2.1.1 Shear strength of the bolts

- Eurocodes

The shear force is an average shear stress in a shear plane coinciding by the cross-section of the bolt. The shear force in that cross-section is divided by $\frac{\pi d^2}{4}$. From many tests it seemed that the value of the shear force when the bolts failed, was between 0,70 and 0,86 times the tensile strength of the bolt material.[6]

When keeping all factors in mind, it can be said that $\alpha_v * f_{ub}$ is the design factor of the shear resistance for the design preload. When calculating the connection it is clear one bolt can take this shear force $\frac{\pi d^2}{4} * \alpha_v * f_{ub} * n\ (= F_{v,rd})$. But if the shear plane meets the helix wire of the bolt, $\frac{\pi d^2}{4}$ has to be changed by A_s.

A_s can be calculated with this formula: $\frac{\pi}{4}(\frac{d_k+d_f}{2})^2$

- d_f = average of the core diameter and the outer diameter
- d_k = the core diameter

So the total shear strength of all bolts is the shear strength of one bolt (with 2 shear planes) multiplied by the number of bolts. This designed load should be larger than the applied load on the elements. n in the formula below is the number of shear planes.

$$F_{applied} < \frac{\frac{\pi d^2}{4} * \alpha_v * f_{ub}}{\gamma_{M2}} * n \tag{1}$$

Where the shear plane passes through the threaded portion of the bolt (A is the tensile stress area of the bolt As):

- o for classes 4.6, 5.6 and 8.8: $\alpha_v = 0{,}6$
- o for classes 4.8, 5.8, 6.8 and 10.9: $\alpha_v = 0{,}5$

Where the shear plane passes through the unthreaded portion of the bolt (A is the gross cross section of the bolt): $\alpha_v = 0{,}6$.

The design shear resistance $F_{v,rd}$ per shear plane should only be used where the bolts are used in holes with nominal clearances not exceeding those for normal holes.

- Chinese codes [8][9]

The bolts used during the tests were not conform the Eurocodes. Because of this the Chinese codes are also given.

The Chinese formula is not much different from the Eurocodes.

$$F_{applied} < \frac{\pi d^2}{4} * n * f_v^b$$

With:

- o n = The number of shear planes
- o f_v^b = The shear force for 4.6 and 4.8 (C class) = 140 N/mm²

2.1.2 Tearing or fracture of the connecting plate

For this part, two failure modes are possible. The piece of plate between the hole and the end can tear but in between two holes the plate can tear too.

The pressure between the bolt and the hole edge can not be too large, neither for the shank or the connected parts. The pressure is distributed uneven over half of the perimeter of the shank and over the length of the shank. This nominal stress is calculated with $\frac{F}{td}$. See Figure 177 for the details.

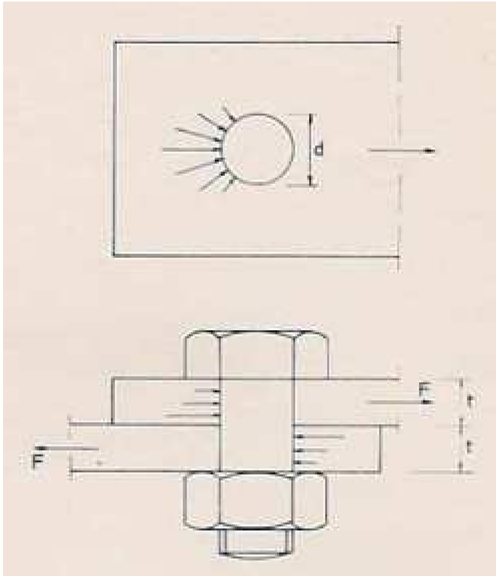


Figure 17: Dimensions of the connection and pressure distribution

For a single shear connection the bolt can become inclined when the external forces are applied. The pressure distribution in the plate and bolts are then even more unevenly distributed. A double shear connection is better because the pressures are distributed symmetrically.

$$F_{b,Rd} = \frac{k_1 * \alpha_b * f_u * d * t}{\gamma_{M2}} \qquad (2)$$

Where α_b is the smallest of α_d ; $\frac{f_{ub}}{f_u}$ or 1 in the direction of the load transfer:

- For end bolts: $\alpha_d = \frac{e_1}{3d_0}$
- For inner bolts: $\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4}$

Perpendicular to the direction of the load transfer:

- For edge bolts: k_1 is the smallest of $2{,}8\frac{e_2}{d_0} - 1{,}7 \; or \; 1{,}4\frac{p_2}{d_0} - 1{,}7 \; and \; 2{,}5$
- For inner bolts: k_1 is the smallest of $1{,}4\frac{p_2}{d_0} - 1{,}7 \; or \; 2{,}5$

Table 1: Minimum and maximum distances

	Minimum	Maximum	
		exposed	not exposed
End distance e_1	$1{,}2d_0$	4t + 40	
Edge distance e_2	$1{,}2d_0$	4t + 40	
Distance e_3 in slotted holes	$1{,}5d_0$		
Distance e_4 in slotted holes	$1{,}5d_0$		
Spacing p_1	$2{,}2d_0$	the smaller of 14t or 200mm	the smaller of 14t or 200mm
Spacing $p_{1,0}$		the smaller of 14t or 200mm	
Spacing $p_{1,i}$		the smaller of 14t or 200mm	

Spacing p_2	$2{,}4d_0$	the smaller of 14t or 200mm	the smaller of 14t or 200mm

The symbols can be found by Figure 18.

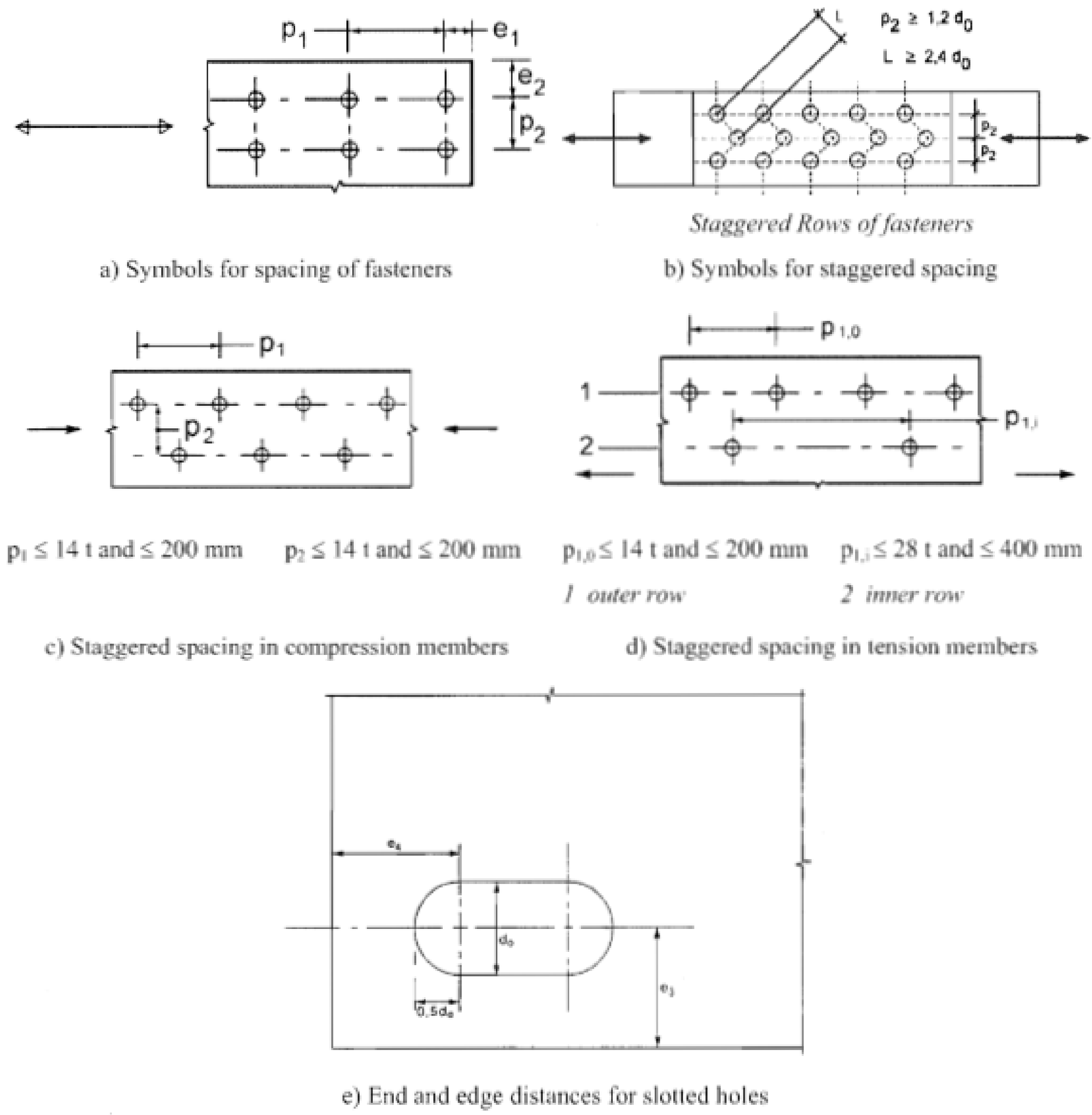


Figure 18: Sympbols for end and edge distances and spacings of fasteners.

Remarks : The bearing resistance $F_{b,Rd}$ for bolts

- For oversized holes it is 0,8 times the bearing resistance for bolts in normal holes.
- For slotted holes, where the longitudinal axis of the slotted hole is perpendicular to the direction of the force transfer, is 0,6 times the bearing resistance for bolts in round, normal holes.

2.1.3 Shear resistance of the plate[7]

Three failure modes are possible for the shear resistance of the plate. The basic requirement is $V_{Ed} \leq V_{Rd,min}$, where $V_{Rd,min} = \min(V_{Rd,g}; V_{Rd,n}; V_{Rd,b})$. These shear forces are respectively the shear resistance of the gross section, net section and the block tearing resistance.

- Shear resistance of gross section

$$V_{Rd,g} = 2 * \frac{h_p t_p}{1{,}27} \frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$

Note: the coefficient 1,27 takes into account the reduction in shear resistance due to the presence of the nominal in-plane bending which produces tension in the bolts.

- Shear resistance of net section

$$V_{Rd,n} = 2 * A_{v,net} \frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

$$A_{v,net} = t_p(h_p - n_1 d_0)$$

γ_{M2} is the partial factor for the resistance of net sections

- Block tearing resistance

$$V_{Rd,b} = 2 * (\frac{f_{u,p} A_{nt}}{\gamma_{M2}} + \frac{f_{y,p} A_{nv}}{\sqrt{3}\gamma_{M0}})$$

But if $h_p < 1{,}36 p_3$ and $n_1 > 1$ then:

$$V_{Rd,b} = 2 * (\frac{0{,}5 f_{u,p} A_{nt}}{\gamma_{M2}} + \frac{f_{y,p} A_{nv}}{\sqrt{3}\gamma_{M0}})$$

$$A_{nt} = t_p(e_2 - 0{,}5 d_0)$$

$$A_{nv} = t_p(h_p - e_1 - (n_1 - 0{,}5) d_0)$$

Where p_3 is the gauge (cross centres).

2.1.4 Most loaded bolt

In every bolt/plate pattern there are certain bolts which will have a larger load. To check if this bolt will fail sooner than the group load bearing, next formulae has to be checked. In this case, the assumption is made that F is unknown. This way it is possible to check when one certain bolt in the pattern will fail. This check will only be done for LSZ 12 and LSZ 13 because it is possible to compare them afterwards.

$$N_{1Fy} = \frac{F}{n}$$

With:

- F = the force which is acting (unknown)
- n = number of bolts used = 6 in both cases

$$N_{1Tx} = \frac{M * y_i}{\sum x_i^2 + \sum y_i^2}$$

$$N_{1Ty} = \frac{M * x_i}{\sum x_i^2 + \sum y_i^2}$$

With:

- M = acting moment due to the force eccentricity
- xi and yi are the distance from the centre of the plate to the bolt.

$$\sqrt{{N_{1Tx}}^2 + (N_{1Ty} + N_{1Fy})^2} \leq N_v^b$$

2.2 Welded connections

As described in chapter 1, different welding connections exist. When constructing these farmhouses fillet welds will be used to connect both steel elements. The safety factor for welded connections is again $\gamma_{M2} = 1{,}25$.

2.2.1 Fillet welds

Fillet welds may be used for connecting parts where the fusion faces from an angle of between 60° and 120°. Angles smaller than 60° are also permitted. However, in such cases the weld should be considered to be a partial penetration butt weld. For angles greater than 120° the resistance of fillet welds should be determined by testing. Fillet welds finishing at the ends or sides of parts should be returned continuously, full size, around the corner for a distance of at least twice the leg length of the weld, unless access or the configuration of the joint renders this impracticable.

2.2.2 Rules for correct application

2.2.2.1 Length of welds

The effective length of a fillet weld l_{eff} should be taken as the length over which the fillet is full-size. This may be taken as the overall length of the weld reduced by twice the effective throat thickness a. provided that the weld is full size throughout its length including starts and terminations, no reduction in effective length need to be made for either the start or the termination of the weld. A fillet weld with an effective length less than 30 mm or less than 6 times its throat thickness, whichever is larger, should not be designed to carry loads.

2.2.2.2 Effective throat thickness

The effective throat thickness, a, of a fillet weld should be taken as the height of the largest triangle (with equal or unequal legs) that can be inscribed within the fusion faces and the weld surface, measured perpendicular to the outer side of this triangle. See Figure 19. The effective throat thickness of a fillet weld should not be less than 3 mm. In determining the design resistance of a deep penetration fillet weld, account may be taken of its additional throat thickness provided that preliminary tests show that the required penetration can consistently be achieved.

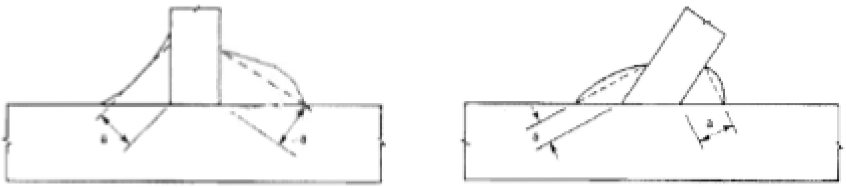


Figure 19: Throat thickness of a fillet weld

2.2.3 Design resistance of fillet welds

There are two design methods which can be used, the directional method or the simplified method.

- ***Directional method***

In this method, the forces are transmitted by a unit length of weld and are resolved into components parallel and transverse to the longitudinal axis of the weld and normal and transverse to the plane of its throat. The design throat A_w should be taken as $A_W = \sum a * l_{eff}$. The location of the design throat should be assumed to be concentrated in the root. A uniform distribution of stress is assumed on the throat section of the weld, leading to the normal and shear stresses shown in Figure 20, as follows

- $\sigma_{\perp}$ is the normal stress perpendicular to the throat
- $\sigma_{\parallel}$ is the normal stress parallel to the axis of the weld
- $\tau_{\perp}$ is the shear stress (in the plane of the throat) perpendicular to the axis of the weld
- $\tau_{\parallel}$ is the shear stress (in the plane of the throat) parallel to the axis of the weld

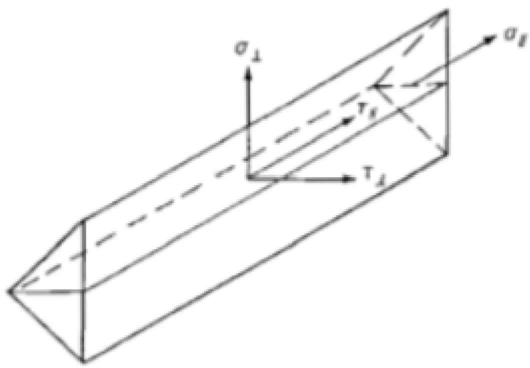


Figure 20: Stresses on the throat section of a fillet weld

The normal stress $\sigma_{\parallel}$ parallel to the axis is not considered when verifying the design resistance of the weld. The design resistance of the fillet weld will be sufficient if the following are both satisfied:

$$(\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2))^{0,5} \leq \frac{f_u}{\beta_W \gamma_{M2}} \, and \; \sigma_{\perp} \leq \frac{0,9 f_u}{\gamma_{M2}} \tag{3}$$

With:

- o f_u is the nominal ultimate tensile strength of the weaker part joined
- o β_W is the appropriate correlation factor taken from table 2.

Table 2: Correlation factor β_W for fillet welds

Standard and steel grade			Correlation factor β_w
EN 10025	EN 10210	EN 10219	
S 235 S 235 W	S 235 H	S 235 H	0,8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,9
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,0
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,0

Welds between parts with different material strength grades should be designed using the properties of the material with the lower strength grade.

- ***Simplified method***

Alternative to the directional method the design resistance of a fillet weld may be assumed to be adequate if, at every point along its length, the resultant of all forces per unit length transmitted by the weld satisfy the following criterion:

$$F_{w,Ed} \leq F_{w,Rd} \tag{4}$$

With:

- o $F_{w,Ed}$ is the design value of the weld force per unit length
- o $F_{w,Rd}$ is the design weld resistance per unit length

Independent of the orientation of the weld throat plane to the applied force, the design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw.d} * a$$

With:

- o $f_{vw.d}$ is the design shear strength of the weld.

The design shear strength $f_{vw.d}$ of the weld should be determined from:

$$f_{vw.d} = (f_u/\sqrt{3})/(\beta_W \gamma_{M2})$$

3 Test setup

3.1 Bolted connections

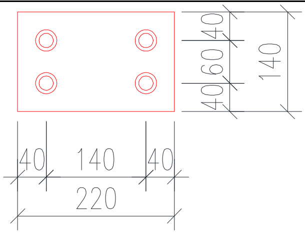
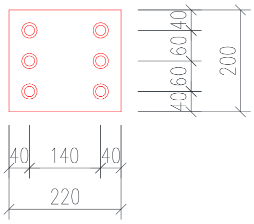
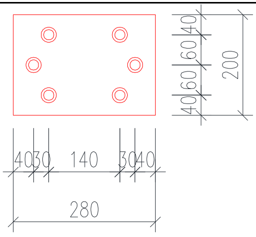
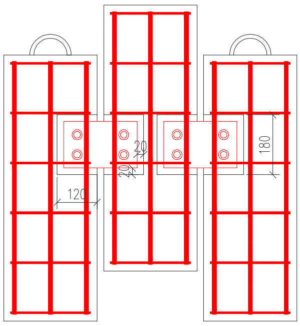
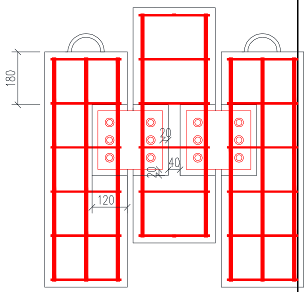
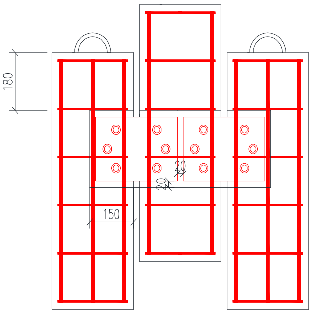
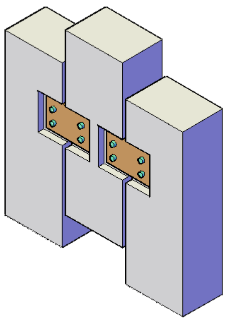
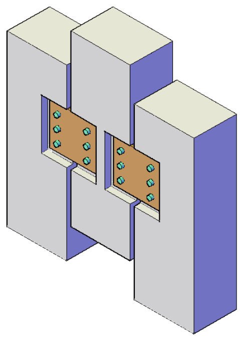
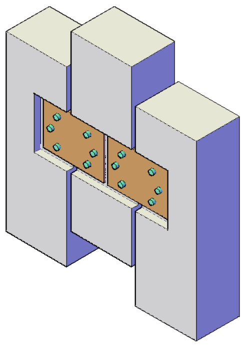
In this paragraph the bolted connections will be outlined.

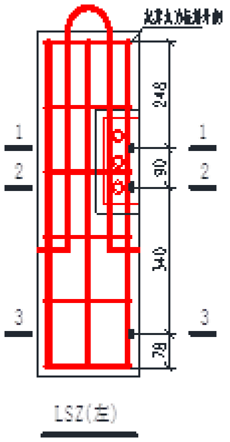
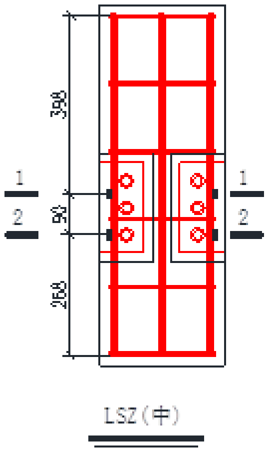
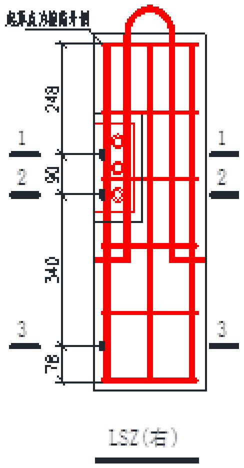
3.1.1 Performed tests

The experiments of assembled shear wall structure are made by Sichuan University and Department of industrial construction in order to explore and study Building industrialization. Testing components are all shaped structural columns with small dimensions. There are totally 18 components with 280 mm long, 200 mm wide and 800 mm high. Each component needs high precision.

Two different batches were made. LSZ 4 and LSZ 5 are part of the first batch. These elements were test elements to check if there are no flaws during the test. Also less stirrups were present when comparing it with the second batch. In the second batch three different tests (different bolt pattern) were performed only once. The test names are LSZ 11, LSZ 12 and LSZ 13. Table 3 shows the lay-out of each plate and number of bolts which were used. LSZ 11 is the same as LSZ 4 and LSZ 5 but there are more reinforcements in the concrete elements.

Table 3: Test properties

	Component 1 – LSZ 11	Component 2 – LSZ 12	Component 3 – LSZ 13
Bolt location			
Component shape			
Characteristics	There are four bolts in each connection. Two rows in vertical sequence. The size of the steel plate is different from the initial test elements.	There are six bolts in each connection. Two rows in vertical sequence. There are two more bolts in comparison with component 1 to enhance vertical shear.	There are six bolts in each connection. Two more bolts compared with component 1 to enhance vertical shear and bending.
3d photograph			

Strain gauge position			

3.1.2 Performed tests

Figure 211 shows the test setup for the bolted connections. An axial load is placed on the middle element. Afterwards the forces are transferred to the adjacent elements. When applying the equations which are discussed in the previous chapter, only one side is calculated. Keeping this in mind the result has to be multiplied by two.

Figure 21: Test setup bolted connection for three bolts (LSZ 12), before and after

3.2 Welded connections

3.2.1 Performed tests

These experiments are also performed by Sichuan University and Department of industrial construction in order to explore and study building industrialization.

Again two batches were made. The difference between both batches is that in batch 1 (HJZ 2 and HJZ 3) only 6 stirrups are present while in batch 2 (HJZ 7 and HJZ 8) 7 stirrups are present. Next figure represents the tested elements.

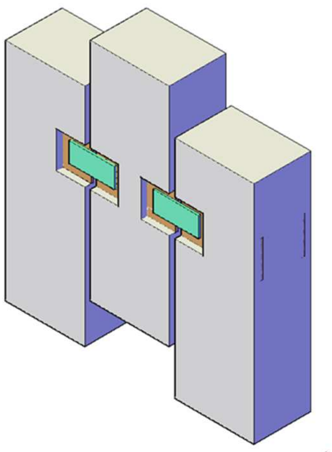


Figure 22: Layout welded test element

4 Analysis

The material characteristics have to be known to use the equations which were discussed in chapter 2. These are represented in Table 4 and 5.

Table 4: Steel plate characteristics

Steel type of the plate	f_y(MPa)	f_u(MPa)	measure(mm) length×width×thickness
Q235 = S235	235	376	800×220×140 (LSZ 11) 800×220×200 (LSZ 12) 800×280×200 (LSZ 13)

Table 5: Bolt characteristics

Class of bolt	f_y(Mpa)	f_u(Mpa)	diameter(mm)
M4.8	320	400	20

Next, these equations from Chapter 2 can be solved. The steel which is used for welding both parts together is also Q235.

4.1 Check LSZ 11

4.1.1 Shear strength of bolts

- Eurocode

The formula for checking the bearing strength

$$F_{applied} < \frac{\frac{\pi d^2}{4} * \alpha_v * f_{ub}}{\gamma_{M2}} * n$$

With:

- d = Diameter of bolts = 20 mm
- α_v = Partial factor = 0,5
- f_{ub} = The ultimate tensile strength of bolt = 400 N/mm²
- n = Amount of shear planes = 4

$$\frac{\pi(20\ mm)^2}{4} * 0{,}5 * 400 \frac{N}{mm^2} * 4 = 251.327\ N$$

These final value has to be multiplied by two which forms 502.655 N. With safety factor this is 402.124 N.

- Chinese code

$$F_{applied} < \frac{\pi d^2}{4} * n * f_v^b$$

$$F_{applied} < \frac{\pi * (20\ mm)^2}{4} * 4 * 140 = 175.929\ N$$

These final value has to be multiplied by two which forms 351.858 N.

4.1.2 Bearing strength of bolts

The formula for the bearing strength of the bolts is:

$$F_{b,Rd} = \frac{k_1 * \alpha_b * f_u * d * t}{\gamma_{M2}}$$

Where α_b is the smallest of α_d ; $\frac{f_{ub}}{f_u}$ or 1 in the direction of the load transfer:

- For end bolts: $\alpha_d = \frac{e_1}{3d_0}$
- For inner bolts: $\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4}$

Perpendicular to the direction of the load transfer:

- For edge bolts: k_1 is the smallest of $2{,}8\frac{e_2}{d_0} - 1{,}7\ or\ 1{,}4\frac{p_2}{d_0} - 1{,}7\ and\ 2{,}5$
- For inner bolts: k_1 is the smallest of $1{,}4\frac{p_2}{d_0} - 1{,}7\ or\ 2{,}5$

With:

- e_1 = Distance to the end of the plate = 40 mm
- d_0 = Size of the bolt hole = 22 mm
- p_1 = Distance between second row = 0 mm
- e_2 = Distance to the end of the plate = 40 mm
- p_2 = Distance between two holes = 60 mm
- d = Diameter of the bolt hole = 20 mm
- t = Thickness of the steel plate = 8 mm

After filling in these values:

- For the end bolts
  - $\alpha_b = \frac{e_1}{3d_0}\ or\ \frac{f_{ub}}{f_u}\ or\ 1$ (take the smallest)

$\alpha_b = \frac{40}{3*22}\ or\ \frac{400}{376}\ or\ 1$ ➔ = 0,606

  - k_1 is the smallest of $2{,}8\frac{e_2}{d_0} - 1{,}7\ or\ 1{,}4\frac{p_2}{d_0} - 1{,}7\ and\ 2{,}5$

$2{,}8\frac{40}{22} - 1{,}7\ or\ 1{,}4\frac{60}{22} - 1{,}7\ and\ 2{,}5$ ➔ = 2,118

$$F_{b,Rd} = 2{,}118 * 0{,}606 * 376\frac{N}{mm^2} * 20\ mm * 8\ mm = 77.215{,}8\ N$$

There are no inner bolts present. Now these values have to be added up to see the total bearing capacity of the bolts.

$$77.215{,}8\ N * 2 = 154.432\ N$$

This value has to be multiplied by two to know the total force of the test: 308.863 N. With safety factor this is 247.090 N.

4.1.3 Shear strength of steel plate

- Shear resistance of gross section

$$V_{Rd,g} = 2 * \frac{h_p t_p}{1{,}27}\frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$

With:

- h_p = Height of the plate = 140 mm
- t_p = Thickness of the plate = 8 mm
- $f_{y,p}$ = Yield stress of the plate = 235 N/mm²
- γ_{M0} = Partial safety factor = 1

$$V_{Rd,g} = 2 * \frac{140 * 8}{1{,}27}\frac{235}{\sqrt{3}} = 239.305\ N$$

- Shear resistance of net section

$$V_{Rd,n} = 2 * A_{v,net}\ \frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

$$A_{v,net} = t_p(h_p - n_1 d_0)$$

With:

- n_1 = Number of bolts divided by two = 2
- d_0 = Diameter of the bolt hole = 22 mm
- $f_{u,p}$ = Ultimate tensile stress of the plate = 376 N/mm²
- γ_{M2} = Partial factor = 1,25

$$A_{v,net} = 8 * (140 - 2 * 22) = 768\ mm^2$$

$$V_{Rd,n} = 2 * 768\ \frac{376}{\sqrt{3} * 1{,}25} = 266.752\ N$$

- Block tearing resistance

h_p is lower than $1{,}36p_3$ and $n_1 > 1$ then:

$$V_{Rd,b} = 2 * (\frac{0{,}5 f_{u,p} A_{nt}}{\gamma_{M2}} + \frac{f_{y,p} A_{nv}}{\sqrt{3}\gamma_{M0}})$$

$$A_{nt} = t_p(e_2 - 0{,}5 d_0)$$

$$A_{nv} = t_p(h_p - e_1 - (n_1 - 0{,}5) d_0)$$

p_3 = Distance between both rows = 140 mm

$$A_{nt} = 8 * (40 - 0{,}5 * 22) = 232\ mm^2$$

$$A_{nv} = 8 * (140 - 40 - (2 - 0{,}5) * 22) = 536\ mm^2$$

$$V_{Rd,b} = 2 * \frac{0{,}5 * 376 * 232}{1} + \frac{235 * 536}{\sqrt{3}} = 215.232\ N$$

- Shear resistance

$$V_{Rd,min} = \min\left(V_{Rd,g}; V_{Rd,n}; V_{Rd,b}\right) = \min(239\ kN; 257\ kN; 215\ kN) = 215{,}32\ kN$$

The lowest value is 215,32 kN for one side of the connection. Multiplying this by two makes it 430,46 kN.

4.2 Check LSZ 12

4.2.1 Shear strength of bolts

- Eurocode

The formula for checking the bearing strength

$$F_{applied} < \frac{\frac{\pi d^2}{4} * \alpha_v * f_{ub}}{\gamma_{M2}} * n$$

With:

- o d = Diameter of bolts = 20 mm
- o α_v = Partial factor = 0,5
- o f_{ub} = The ultimate tensile strength of bolt = 400 N/mm²
- o n = Amount of shear planes = 6

$$\frac{\pi (20\,mm)^2}{4} * 0{,}5 * 400 \frac{N}{mm^2} * 6 = 376.991\,N$$

These final value has to be multiplied by two which forms 753.982 N. With safety factor this is 603.186 N.

- Chinese code

$$F_{applied} < \frac{\pi d^2}{4} * n * f_v^b$$

$$F_{applied} < \frac{\pi (20\,mm)^2}{4} * 6 * 140 = 263.894\,N$$

These final value has to be multiplied by two which forms 527.788 N.

4.2.2 Bearing strength of bolts

The formula for the bearing strength of the bolts is:

$$F_{b,Rd} = \frac{k_1 * \alpha_b * f_u * d * t}{\gamma_{M2}}$$

Where α_b is the smallest of α_d ; $\frac{f_{ub}}{f_u}$ or 1 in the direction of the load transfer:

- For end bolts: $\alpha_d = \frac{e_1}{3d_0}$
- For inner bolts: $\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4}$

Perpendicular to the direction of the load transfer:

- For edge bolts: k_1 is the smallest of $2{,}8 \frac{e_2}{d_0} - 1{,}7$ or $1{,}4 \frac{p_2}{d_0} - 1{,}7$ and $2{,}5$

- For inner bolts: k_1 is the smallest of $1{,}4\frac{p_2}{d_0} - 1{,}7 \; or \; 2{,}5$

With:

- e_1 = Distance to the end of the plate = 40 mm
- d_0 = Size of the bolt hole = 22 mm
- p_1 = Distance between second row = 0 mm
- e_2 = Distance to the end of the plate = 40 mm
- p_2 = Distance between two holes = 60 mm
- d = Diameter of the bolt hole = 20 mm
- t = Thickness of the steel plate = 8 mm

After filling in these values:

- For the end bolts
- $\alpha_b = \frac{e_1}{3d_0} \; or \; \frac{f_{ub}}{f_u} \; or \; 1$ (take the smallest)

$\alpha_b = \frac{40}{3*22} \; or \; \frac{400}{376} \; or \; 1$ ➔ = 0,606

- k_1 is the smallest of $2{,}8\frac{e_2}{d_0} - 1{,}7 \; or \; 1{,}4\frac{p_2}{d_0} - 1{,}7 \; and \; 2{,}5$

$2{,}8\frac{40}{22} - 1{,}7 \; or \; 1{,}4\frac{60}{22} - 1{,}7 \; and \; 2{,}5$ ➔ = 2,118

$$F_{b,Rd} = 2{,}118 * 0{,}606 * 376\frac{N}{mm^2} * 20\,mm * 8\,mm = 77.215{,}8\,N$$

- For the inner bolt
- $\alpha_b = \frac{p_1}{3d_0} \; or \; \frac{f_{ub}}{f_u} \; or \; 1$ (take the smallest of these 3)

  Since p_1 is not present in this case, it can be neglected. So the smallest value is 1.

- k_1 is the smallest of $1{,}4\frac{p_2}{d_0} - 1{,}7 \; or \; 2{,}5$

$1{,}4\frac{60}{22} - 1{,}7 \; or \; 2{,}5$ ➔ 2,118

$$F_{b,Rd} = 2{,}118 * 1 * 376\frac{N}{mm^2} * 20\,mm * 8\,mm = 127.419{,}8\,N$$

Now all of these values have to be added up to see the total bearing capacity of the bolts.

$$77.215{,}8\,N * 2 + 127.419{,}8\,N = 281.851\,N$$

This value has to be multiplied by two to know the total force of the test: 563.703 N. With safety factor this is 450.962 N.

4.2.3 Shear strength of steel plate

- Shear resistance of gross section

$$V_{Rd,g} = 2 * \frac{h_p t_p}{1{,}27} \frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$

With:

- h_p = Height of the plate = 200 mm
- t_p = Thickness of the plate = 8 mm
- $f_{y,p}$ = Yield stress of the plate = 235 N/mm²
- γ_{M0} = Partial safety factor = 1

$$V_{Rd,g} = 2 * \frac{200 * 8}{1{,}27} \frac{235}{\sqrt{3}} = 341.864\ N$$

- Shear resistance of net section

$$V_{Rd,n} = 2 * A_{v,net} \frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

$$A_{v,net} = t_p(h_p - n_1 d_0)$$

With:

- n_1 = Number of bolts divided by two = 3
- d_0 = Diameter of the bolt hole = 22 mm
- $f_{u,p}$ = Ultimate tensile stress of the plate = 376 N/mm²
- γ_{M2} = Partial factor = 1,25

$$A_{v,net} = 8 * (200 - 3 * 22) = 1072\ mm^2$$

$$V_{Rd,n} = 2 * 1072 \frac{376}{\sqrt{3}} = 372.342\ N$$

- Block tearing resistance

h_p is higher than $1{,}36 p_3$ and $n_1 > 1$ then:

$$V_{Rd,b} = 2 * (\frac{f_{u,p} A_{nt}}{\gamma_{M2}} + \frac{f_{y,p} A_{nv}}{\sqrt{3}\gamma_{M0}})$$

$$A_{nt} = t_p(e_2 - 0{,}5 d_0)$$

$$A_{nv} = t_p(h_p - e_1 - (n_1 - 0{,}5) d_0)$$

p_3 = Distance between both rows = 140 mm

$$A_{nt} = 8 * (40 - 0{,}5 * 22) = 232\ mm^2$$

$$A_{nv} = 8 * (200 - 40 - (3 - 0{,}5) * 22) = 840\ mm^2$$

$$V_{Rd,b} = 2 * (\frac{376 * 232}{1{,}25} + \frac{235 * 536}{\sqrt{3} * 1}) = 402.402\ N$$

- Shear resistance

$$V_{Rd,min} = \min(V_{Rd,g}; V_{Rd,n}; V_{Rd,b}) = \min(342\ kN; 372\ kN; 402\ kN) = 341{,}86\ kN$$

The lowest value is 342 kN for one side of the connection. Multiplying this by two makes it 683,73 kN.

4.3 Check LSZ 13

4.3.1 Shear strength of bolts

- Eurocode

The formula for checking the bearing strength

$$F_{applied} < \frac{\frac{\pi d^2}{4} * \alpha_v * f_{ub}}{\gamma_{M2}} * n$$

With:

- d = Diameter of bolts = 20 mm
- α_v = Partial factor = 0,5
- f_{ub} = The ultimate tensile strength of bolt = 400 N/mm²
- n = Amount of shear planes = 6

$$\frac{\pi (20\ mm)^2}{4} * 0{,}5 * 400 \frac{N}{mm^2} * 6 = 376.991\ N$$

These final value has to be multiplied by two which forms 753.982 N. With safety factor this is 603.186 N.

- Chinese code

$$F_{applied} < \frac{\pi d^2}{4} * n * f_v^b$$

$$F_{applied} < \frac{\pi (20\ mm)^2}{4} * 6 * 140 = 263.894\ N$$

These final value has to be multiplied by two which forms 527.788 N.

4.3.2 Bearing strength of bolts

The formula for the bearing strength of the bolts is:

$$F_{b,Rd} = \frac{k_1 * \alpha_b * f_u * d * t}{\gamma_{M2}}$$

Where α_b is the smallest of α_d ; $\frac{f_{ub}}{f_u}$ or 1 in the direction of the load transfer:

- For end bolts: $\alpha_d = \frac{e_1}{3d_0}$
- For inner bolts: $\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4}$

Perpendicular to the direction of the load transfer:

- For edge bolts: k_1 is the smallest of $2{,}8\frac{e_2}{d_0} - 1{,}7 \; or \; 1{,}4\frac{p_2}{d_0} - 1{,}7 \; and \; 2{,}5$
- For inner bolts: k_1 is the smallest of $1{,}4\frac{p_2}{d_0} - 1{,}7 \; or \; 2{,}5$

With:

o e_1	= Distance to the end of the plate	= 40 mm
o d_0	= Size of the bolt hole	= 22 mm
o p_1	= Distance between second row	= 0 mm
o e_2	= Distance to the end of the plate	= 100 mm
o p_2	= Distance between two holes	= 60 mm
o d	= Diameter of the bolt hole	= 20 mm
o t	= Thickness of the steel plate	= 8 mm

After filling in these values:

- For the end bolts
  - o $\alpha_b = \frac{e_1}{3d_0} \; or \; \frac{f_{ub}}{f_u} \; or \; 1$ (take the smallest)

$\alpha_b = \frac{70}{3*22} \; or \; \frac{400}{376} \; or \; 1$ ➔ = 1

- o k_1 is the smallest of $2{,}8\frac{e_2}{d_0} - 1{,}7 \; or \; 1{,}4\frac{p_2}{d_0} - 1{,}7 \; and \; 2{,}5$

$2{,}8\frac{40}{22} - 1{,}7 \; or \; 1{,}4\frac{60}{22} - 1{,}7 \; or \; 2{,}5$ ➔ = 2,118

$$F_{b,Rd} = 2{,}118 * 1 * 376\frac{N}{mm^2} * 20\,mm * 8\,mm = 127.419{,}8\,N$$

- For the inner bolt
  - o $\alpha_b = \frac{p_1}{3d_0} \; or \; \frac{f_{ub}}{f_u} \; or \; 1$ (take the smallest of these 3)

    Since p_1 is not present in this case, it can be neglected. So the smallest value is 1.

  - o k_1 is the smallest of $1{,}4\frac{p_2}{d_0} - 1{,}7 \; or \; 2{,}5$

$1{,}4\frac{60}{22} - 1{,}7\ or\ 2{,}5$ ➔ 2,118

$$F_{b,Rd} = 2{,}118 * 1 * 376 \frac{N}{mm^2} * 20\ mm * 8\ mm = 127.419{,}8\ N$$

Now all of these values have to be added up to see the total bearing capacity of the bolts.

$$127.419{,}8\ N * 2 + 127.419{,}8\ N = 382.257\ N$$

This value has to be multiplied by two to know the total force of the test: 764.513 N. With safety factor this is 611.610 N.

4.3.3 Shear strength of steel plate

Note that the partial safety factors are neglected in the calculations.

- Shear resistance of gross section

$$V_{Rd,g} = 2 * \frac{h_p t_p}{1{,}27} \frac{f_{y,p}}{\sqrt{3}\gamma_{M0}}$$

With:

- o h_p = Height of the plate = 200 mm
- o t_p = Thickness of the plate = 8 mm
- o $f_{y,p}$ = Yield stress of the plate = 235 N/mm²
- o γ_{M0} = Partial safety factor = 1

$$V_{Rd,g} = 2 * \frac{200 * 8}{1{,}27} \frac{235}{\sqrt{3}} = 341.864\ N$$

- Shear resistance of net section

$$V_{Rd,n} = 2 * A_{v,net} \frac{f_{u,p}}{\sqrt{3}\gamma_{M2}}$$

$$A_{v,net} = t_p(h_p - n_1 d_0)$$

With:

- o n_1 = Number of bolts divided by two = 3
- o d_0 = Diameter of the bolt hole = 22 mm
- o $f_{u,p}$ = Ultimate tensile stress of the plate = 376 N/mm²
- o γ_{M2} = Partial factor = 1,25

$$A_{v,net} = 8 * (200 - 3 * 22) = 1072\ mm^2$$

$$V_{Rd,n} = 2 * 1072\ \frac{376}{\sqrt{3} * 1{,}25} = 372.342\ N$$

- Block tearing resistance

h_p is higher than $1{,}36p_3$ and $n_1 > 1$ then:

$$V_{Rd,b} = 2 * (\frac{f_{u,p}A_{nt}}{\gamma_{M2}} + \frac{f_{y,p}A_{nv}}{\sqrt{3}\gamma_{M0}})$$

$$A_{nt} = t_p(e_2 - 0{,}5d_0)$$

$$A_{nv} = t_p(h_p - e_1 - (n_1 - 0{,}5)d_0)$$

p_3 = Distance between both rows = 140 mm

$$A_{nt} = 8 * (40 - 0{,}5 * 22) = 232\ mm^2$$

$$A_{nv} = 8 * (200 - 40 - (3 - 0{,}5) * 22) = 840\ mm^2$$

$$V_{Rd,b} = 2 * (\frac{376 * 232}{1{,}25} + \frac{235 * 536}{\sqrt{3} * 1}) = 402.402\ N$$

- Shear resistance

$$V_{Rd,min} = \min(V_{Rd,g}; V_{Rd,n}; V_{Rd,b}) = \min(342\ kN; 465\ kN; 402\ kN) = 341{,}86\ kN$$

The lowest value is 342 kN for one side of the connection. Multiplying this by two makes it 683,73 kN.

4.4 Most loaded bolt

Due to the bolt pattern in the steel plate and the eccentricity the force acts, each bolt takes up different forces. The next paragraph discusses LSZ 12 and LSZ 13. The objective is to look for the force when the bolts will fail.

The eccentricity force is the force which acts on top of the middle element divided by 4. '4' comes from the 4 plates which are used to transfer the force. This force acts on the plate with an eccentricity of ¼ the length of the plate from the centre. The eccentricity for LSZ 12 and LSZ 13 is 55 mm and 70 mm respectively.

4.4.1 LSZ 12

Force F is the unknown so has the value 'x' in the equations. At the end it is visible which bolt has the lowest value and will fail first. First the shear strength of one bolt in one shear plane has to be calculated.

$$N_v^b = n_v * \frac{\pi * d^2}{4} * f_v^b = 1 * \frac{\pi * 20^2}{4} * 140 = 43{,}96\ kN$$

Some more general information before using the equations

$$The\ applied\ force\ for\ one\ plate\ is\ F = \frac{x}{4}\ and\ moment\ M = \frac{x}{4} * 55\ mm.$$

$$\sum x_i^2 + \sum y_i^2 = 70^2 * 6 + 60^2 * 4 = 43.800\ mm^2$$

$$N_{1Fy} = \frac{x/4}{6} = \frac{x}{24}$$

First the corner bolts are calculated:

$$N_{1Tx} = \frac{\frac{x}{4} * 55\ mm * 60\ mm}{43.800\ mm^2}$$

$$N_{1Ty} = \frac{\frac{x}{4} * 55\ mm * 70\ mm}{43.800\ mm^2}$$

$$\sqrt{\frac{\frac{x}{4} * 55\ mm * 60\ mm}{43.800\ mm^2}^2 + \left(\frac{\frac{x}{4} * 55\ mm * 70\ mm}{43.800\ mm^2} + \frac{x}{24}\right)^2} \leq 43{,}960\ kN$$

Solve for x.

$$x = 662{,}344\ kN$$

Now the middle bolts are calculated:

$$N_{1Tx} = \frac{\frac{x}{4} * 55\ mm * 0\ mm}{43.800\ mm^2} = 0$$

$$N_{1Ty} = \frac{\frac{x}{4} * 55\ mm * 70\ mm}{43.800\ mm^2}$$

$$\sqrt{\left(\frac{\frac{x}{4} * 55\ mm * 70\ mm}{43.800\ mm^2} + \frac{x}{24}\right)^2} \leq 43{,}960\ kN$$

Solve for x.

$$x = 690{,}744\ kN$$

4.4.2 LSZ 13

Some more general information before using the equations, the shear force remains the same.

$$The\ applied\ force\ for\ one\ plate\ is\ F = \frac{x}{4}\ and\ moment\ M = \frac{x}{4} * 70\ mm.$$

$$\sum x_i^2 + \sum y_i^2 = 70^2 * 4 + 100^2 * 2 + 60^2 * 4 = 54.000\ mm^2$$

$$N_{1Fy} = \frac{x/4}{6} = \frac{x}{24}$$

First the corner bolts are calculated:

$$N_{1Tx} = \frac{\frac{x}{4} * 70\ mm * 60\ mm}{54.000\ mm^2}$$

$$N_{1Ty} = \frac{\frac{x}{4} * 70\ mm * 70\ mm}{54.000\ mm^2}$$

$$\sqrt{\frac{\frac{x}{4} * 70\ mm * 60\ mm}{54.000\ mm^2}^2 + (\frac{\frac{x}{4} * 70\ mm * 70\ mm}{54.000\ mm^2} + \frac{x}{24})^2} \leq 43{,}960\ kN$$

Solve for x.

$$x = 653{,}920\ kN$$

Now the middle bolts are calculated:

$$N_{1Tx} = \frac{\frac{x}{4} * 100\ mm * 0\ mm}{54.000\ mm^2} = 0$$

$$N_{1Ty} = \frac{\frac{x}{4} * 70\ mm * 100\ mm}{54.000\ mm^2}$$

$$\sqrt{(\frac{\frac{x}{4} * 70\ mm * 100\ mm}{54.000\ mm^2} + \frac{x}{24})^2} \leq 43{,}960\ kN$$

Solve for x.

$$x = 593{,}460\ kN$$

4.4.3 Summary

The middle bolts of element LSZ 13 have the lowest force. Next table summarizes the results. This means that when loading the elements the middle bolt can break first so afterwards the strength only can rely on the 2 outer bolts.

Table 6: Load on each bolt individually

	Corner bolts (kN)	Middle bolts (kN)
LSZ 12	662,34	690,74
LSZ 13	653,92	593,46

4.5 Bearing resistance welded connections

In the performed test the value for ‘a’ was 6 mm. When using the value in the equations, it is possible to calculate the bearing resistance of the welded plates. The simplified method is used. Since Q235 is the same as S235 from table 2. The value of β_W is 0,8.

$$f_{vw.d} = \frac{\frac{376\ N/mm^2}{\sqrt{3}}}{0{,}8 * 1{,}25} = 217{,}08\ N/mm^2$$

$$F_{w,Rd} = 217{,}08 \frac{N}{mm^2} * 6\ mm = 1302\ N/mm$$

This value of 1302 N/mm has to be multiplied by the length of the weld. The length of the weld is 2 time 70 mm plus 90 mm.

$$1302 \frac{N}{mm} * 210\ mm = 273.420\ N$$

The total value one side of the elements can bear is 273,42 kN. Double sided this is 546,84 kN.

5 Conclusions

5.1 Conclusions bolted connections

The results of the tests and calculations are shown in Table 7.

Table 7: Results of test and calculations

	Concrete Strength	*Failed test force*	*Failed displacement*	*Shear strength bolts Eurocodes*	*Shear strength bolts Chinese codes*	*Bearing strength bolts*	*Minimum shear strength Vrd,min*
	（MPa）	*(kN)*	*(mm)*	*(kN)*	*(kN)*	*(kN)*	*(kN)*
LSZ 4		256,7	55,23				
LSZ 5		259,7	52,08				
LSZ 11	33,72	417,9	67,68	402,12	351,86	247,09	430,46
LSZ 12	23,87	572,1	91,69	603,19	527,79	450,96	683,73
LSZ 13	25,31	429,8	40,64	603,19	527,79	611,61	683,73

It can be noticed that for LSZ 11 and LSZ 12 the computed forces of the bearing strength of the bolts lie lower than the failed force of the test. This can also be seen when checking the test material after the test. In all cases the bolts have failed together with the concrete. It is still not yet decided whether the bolts or concrete failed first.

(A) (B)

Figure 23: Failed bolts (A) and concrete (B)

Adding the results of LSZ 4 and LSZ 5 to the conclusion can give a better representation of the stirrups in the concrete. LSZ 4 and LSZ 5 have one stirrup less than the other three elements. As stated above, it was not yet sure which of the two materials failed first (bolts or concrete). With this additional results, most likely the concrete will have failed first due to shortage of shear reinforcement. Since it is not known for what force the concrete is designed it is not yet 100% sure the concrete failed first. In chapter 6 this will be checked as a recommendation.

When comparing the results of the Eurocodes and the Chinese codes it can be seen that the shear forces of the Chinese codes are the smallest. These forces also below the failed force. For LSZ 13 this is not the case but this is explained below. Since the bolts are not conform the Eurocodes, the Chinese codes have to be followed. These calculated shear forces lie below the failed forces so it can be said that a good, safe result is obtained.

It can be noticed that the failed force of LSZ 13 lies much lower than LSZ 12 despite it can handle the same shear forces. This is probably due to the lay-out of the bolts. Paragraph 4.4 proves that the middle bolts of LSZ 13 break with a lower force. These values which were calculated are almost double than those achieved in the test. That is why it is safe to assume that the concrete failed first. Then due to the large deformations, the bolts are not loaded in its desired way, so could break faster.

5.2 Conclusion welded connection

Table 8: Results welded connection

	yield displacement (mm)	*yield force* (kN)	*concrete Strength* (MPa)	*Failed force* (kN)	*Failed displacment* (mm)	*Bearing resistance* (kN)
HJZ 2				259,9	21,32	546,84
HJZ 3				297,4	22,68	
HJZ 7	5,82	360,2	35,70	497,2	23,98	
HJZ 8	2,92	280,5	37,80	551,1	28,51	

Table 8 gives the results of the tests and the value of the calculated bearing resistance. The value of the bearing resistance is with the safety factor included.

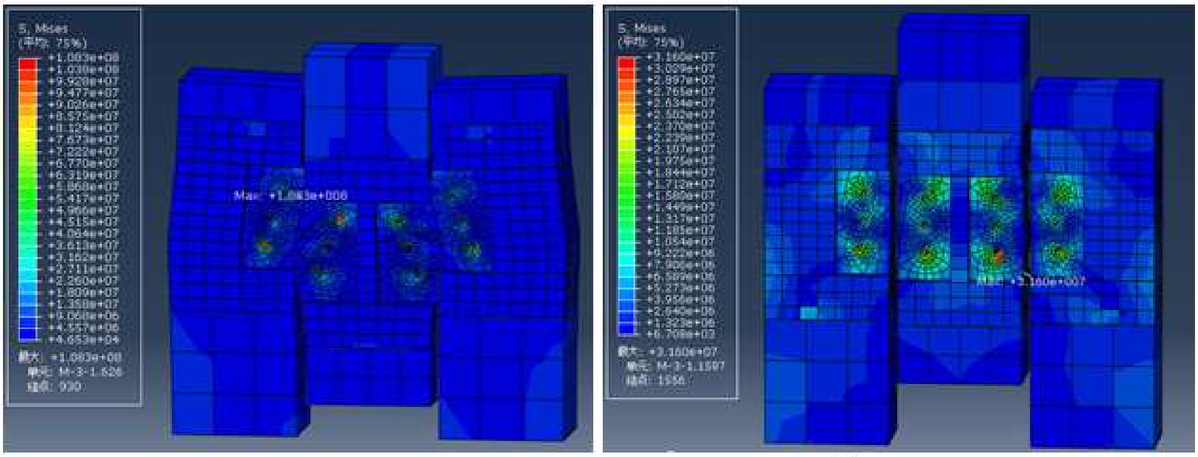
Same as for the bolted connections it is clear that the reinforcement in the concrete shows great influences on the failed force (batches with different amount of stirrups). This intensifies the suspicion that the reinforcement in the concrete elements is too low and has to be calculated.

For HJZ 7 and HJZ 8 the failed force lies in the neighbourhood of the bearing resistance of the welded connection. This is a good result in such a way that safety is guaranteed. Both failed forces lie close to each other. The values of HJZ 2 and HJZ 3 are very low which can imply that the welds did not fail, but the concrete did.

5.3 Comparison with numerical analysis

The numerical analysis was developed by Xu Haoyang. The final results which are achieved can be compared with the laboratory tests. Before the specimen yielded, the steps were load controlled. As soon the specimen yielded it was displacement controlled. The numerical analysis was done with the program ABAQUS.

Next figures show the load and displacement controlled steps.



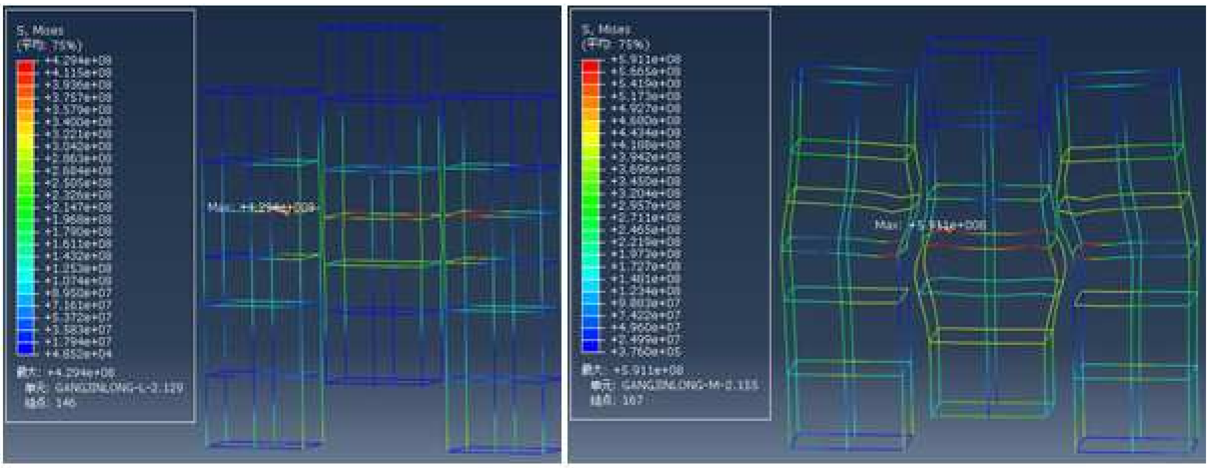
A B

Figure 24: (A) Displacement controlled, (B) Load controlled

From these figures it is impossible to see which element (bolt, plate, concrete or reinforcement bars) failed first. For this, the whole process has to be shown. Xu Haoyang concluded from his results that the concrete failed first. This conclusion is conform the conclusions which were made in paragraph 5.1 and paragraph 5.2. However the next chapter tries to prove this.

The parameter analysis showed that an increase of stirrups is beneficial to the performance of the fabricated bolted connections. It improves the bearing capacity of the connection and the stiffness of the concrete elements

Figure 25 shows the stresses in the reinforcement bars.



A B

Figure 25: Stresses in the reinforcement bars (A = load controlled, B = displacement controlled)

This figure shows that the highest stresses are in the shear reinforcement. This can be kept in mind when calculating the reinforcement in the first recommendation in the next chapter!

All figures from the numerical analysis can be seen in the appendix (Chapter 8).

- Important notice!

The comparison between the laboratory tests and the numerical analysis is not 100% correct. Some secondary factors existing in the experiment are not considered in the finite element analysis nor theory. For example the influence of the PVC-pipe which is embedded in the bolt hole and steel plate. Therefore it is inevitably that some small dissimilarities arises between all the results. However this most likely can be ignored according the Phd/master students.

6 Recommendations

In this chapter a few recommendations will be addressed which can make the bearing loads from the elements larger. These suggestions can also be taken into account when a next batch of test will be executed. Multiple batches have to be made and tested to ensure that everything is statistically correct because in any batch a small error can occur for one of the tests. This could impose bad results which are not conform to the theories.

6.1 Strength of concrete[10]

In this paragraph the concrete strength will be discussed. In the previous chapter it could be concluded that the concrete, around the connections, would fail first (visual and numerical analysis). By increasing the load, which the connections should bear, the concrete strength will change too.

Here follows the explanation of how the reinforcement bars in the concrete are calculated. In the laboratory it is noticed that the elements stay together (not move away from each other). Now suppose that this means that the force resultant does not exceed over 45 degrees with the concrete. The most conservative way to calculate the reinforcement is: in one case calculate the longitudinal bars and in the other case the shear reinforcement. These are the two most outer possibilities. For the longitudinal bars (case 1) the force is placed vertically. For the shear reinforcement (case 2) the force acts in 45 degrees. The actual force acts with an angle between 45 and 90 degrees with the concrete. The place where the load acts is in the centre of bolt holes. (so for LSZ 12: acts at the height of the middle bolt). See figure 25 for more clarification.

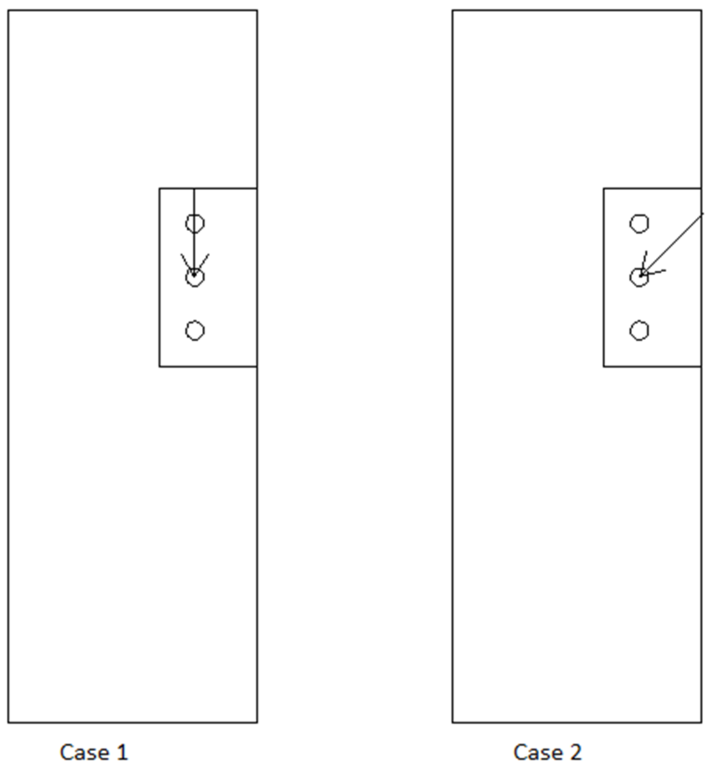


Figure 26: The two most conservative possibilities to calculate the reinforcement bars

The objective is to use the same reinforcements inside the concrete but the concrete strength increases per designed load. This will be achieved for the longitudinal reinforcement, but not for the shear reinforcement. Afterwards the longitudinal and shear reinforcement is placed over the total height of the element.

In this section the calculations are outlined to achieve the desired concrete strength for a certain load. The concrete strength and the reinforcement bars in the concrete are calculated for five different forces (100 kN, 200 kN, …, 500 kN) which are transferred to one element. This means that the actual force on top of the middle element is twice as large!

Only the reinforcement bars for case 2 are calculated. For case 1 the minimum shear reinforcement is always needed and the longitudinal reinforcement does not increase until the load is larger than 500 kN. All calculations are done by the European codes[10].

6.1.1 Calculation of longitudinal reinforcement bars

The general dimensions of the concrete element are

- b = width of the element = 200 mm
- d = distance from the bars to the other edge = 240 mm
- e = eccentricity of the load = 80 mm
- f_{ck} = Characteristic strength of the concrete (differs from load to load)
- f_{cd} = $f_{ck}/1,5$
- f_{yk} = characteristic strength of the steel = 500 N/mm²
- f_{yd} = $f_{yk}/1,15$ = 435 N/mm²

The design codes for a symmetrical reinforced cross-section are used.

- N_{sd} = design force = 100 kN = 70,7 kN (vertically)
- M_{sd} = design moment = 70,7 kN * 0,08 mm = 5,7 kNm
- f_{ck} = 12 N/mm²

$$\nu_d = \frac{N_{sd}}{b * d * f_{cd}} = \frac{70{,}7 * 10^3}{200 * 240 * 8} = 0{,}18$$

$$\mu_d = \frac{M_{sd}}{b * d^2 * f_{cd}} = \frac{5{,}7 * 10^6}{200 * 240^2 * 8} = 0{,}06$$

Now the value of ω is chosen by using the compound bending for a rectangular cross-section. The interaction diagram for a symmetrical reinforced cross-section (steel quality 500).

But first the value of $\delta_l = \frac{d_l}{d} = \frac{40}{240} = 0{,}1667$. Therefore by using the graph for δ_l, ω is obtained (=0). With this value it is possible to calculate the desired reinforcement

$$A_{s1} = A_{s2} = \omega * b * d * \frac{f_{cd}}{f_{yd}} = 0$$

But there has to be a minimum of reinforcement bars! The largest value of next formula has to be taken.

$$A_{min} = \begin{cases} \dfrac{0,15 * N_{sd}}{f_{yd}} = \dfrac{0,15 * 70,7 * 10^3}{435} = 24\,mm^2 \\ 0,003 * A_c = 0,003 * 200 * 240 = 168\,mm^2 \end{cases}$$

A total reinforcement of 4 times diameter 8 has to be taken (201 mm²). Each bar is placed in the corner of the element. Now it is possible to calculate the stirrups. This process has to be repeated for the higher loads

6.1.2 Calculation of the stirrups (shear reinforcement)

First it has to be checked if there is actually need for shear reinforcement. This can be done by solving next equation. This equation has to be higher than V_{sd} (= 70,7 kN) so no reinforcement is needed.

$$Vrd1 = 0,12 * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} * b * d = 7148\,N$$

Where:

- $k = 1 - \sqrt{\frac{200}{d}} = 1 - \sqrt{\frac{200}{240}} = 0,91$
- $\rho_l = \frac{A_{s1}}{b*d}$
- $A_{s1} = 100,5\,mm^2$

V_{rd1} is much smaller than V_{sd} so stirrups are needed! Next V_{rd2} is calculated which checks the pressure path.

$$V_{rd2} = 0,45 * b * d * \nu * f_{cd} = 0,45 * 200 * 240 * 0,64 * 8 = 110592\,N$$

The value of V_{rd2} is larger than V_{sd} which was required. For the other acting forces f_{cd} is made larger so this constraint is fulfilled.

Next, the diameter of the stirrups and the step can be computed. Again V_{sd} has to be smaller than V_{rd3}.

$$V_{rd3} = V_{rd1} + V_{wd}$$

$$V_{wd} = \frac{A_{sw}}{s} * 0,9 * d * f_{ywd}$$

Where:

- A_{sw}= the area of shear reinforcement
- f_{ywd} = the steel quality of the shear reinforcement
- s = the distance between two stirrups. This value has to be smaller than
  - The smallest dimension of the cross-section = 200 mm
  - < 300 mm
  - 12 * diameter of longitudinal reinforcement = 12*8 mm = 96 mm

A distance of 80 mm is taken for calculating the area of the stirrups. Since A_{sw} is computed now (= 46,3 mm²), the stirrup can be chosen. A stirrup with diameter 8 (A_s = 50 mm²) is large enough.

This means that in total 4 longitudinal bars are needed with a diameter of 8 mm. Next to complete the steel frame of the reinforced concrete, steel stirrups are added of diameter 8 mm with a distance of 80 mm between each other.

Next table shows the other characteristics of all forces from 100 kN to 500 kN.

Table 9: Reinforcement bars for each load

	100 kN	200 kN	300 kN	400 kN	500 kN
Nsd (kN)	70,7	141,4	212,1	282,8	353,5
Vsd (kN)	70,7	141,4	212,1	282,8	353,5
Longitudinal reinforcement (Bars x Diameter)	4x8	4x8	4x8	4x8	4x8
Shear reinforcement diameter (s = 80 mm)	8	12	14	20	20
Concrete strength (f_{ck} = N/mm²)	12	16	30	40	50
Longitudinal reinforcement (case 1)	4x8	4x8	4x8	4x8	4x8

6.1.3 In short

The calculations show that the shear reinforcement is the most important reinforcement. For any increase in force both the shear reinforcement AND concrete strength has to be increased. When comparing this calculations with the number of shear reinforcement in the test it is allowed to say the concrete will have failed first. This ‘recommendation’ supports the conclusion which is made in the analysis (Chapter 5).

The first condition (force acts with an angle of 45 degrees) is the most severe. With this concrete strength, which is needed to guarantee the stability of the concrete element, the longitudinal reinforcement for a vertical load is calculated (force acts with an angle of 90 degrees). The table shows that no change occurs in the longitudinal reinforcement even when the forces are increased to 100 kN, …, 500 kN.

Figure 26 shows the reinforcement bars which are placed inside the concrete for the tests. Some changes can be made with these new calculations. First only 4 longitudinal bars are only needed with a diameter of 8 mm. Next the number of stirrups has to change. 10 stirrups are needed with a step of 80 mm. The diameter of the stirrup and the concrete strength changes together with the size of the load which is acting (see table 9)

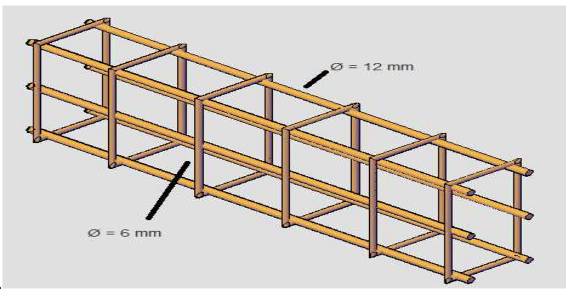


Figure 27: Visualisation of reinforcement bars in the concrete

6.2 Pattern of the bolts in the steel plate

Paragraph 4.4 already discusses the small differences in the pattern of the plate. The objective is to make the force larger which is more favourable.

6.2.1 Moment

Reducing the moment can be done by making the plate width smaller. This is because the eccentricity becomes smaller. It is not allowed to decrease the width just like that. The minimum distances noted in Table 1 still have to be used. Furthermore the concrete coverage also have to be kept in mind.

6.2.2 Bolt locations

When reducing the moment the location of the bolts may not change! Changing the bolt locations can either enlarge the force or reduce it. This process differs from each pattern possibility and has to be checked first if no better pattern is available.

6.2.3 Conclusion

As stated in paragraph 4.5, the bolt pattern of LSZ 12 is better than LSZ 13. When performing new batches of test, the pattern of LSZ 12 is the favourite.

6.3 Change of material

6.3.1 Strength of bolts

Another suggestion is to change the strength of the bolts. Now class 4.8 bolts are used and did not break first. When another type of concrete is used and it happens that these bolts will break first it is possible to use a higher class like for example class 8.8.

6.3.2 Plate

Several material properties can changed of the plate. The first one is to enlarge the thickness of the plate. Another possibility is to use higher grade steel.

In the performed test it was clearly that the plates did not fail. It is in that way not economical to change the material properties since the plate is not yet used to its full potential. Further tests have to prove when this is useful.

6.4 Numerical analysis

The development of the test in ABAQUS is very useful for further research. It is designed in such a way that it is easy to alter some material characteristics so new first impressions of the test can be achieved. For example the concrete strength, type of bolts or the yield strength of the steel plates can be changed.

Next to these small modifications it is possible to change the dimensions of the plates or the bolt pattern. These modifications can be very useful to decrease the number of batches which can be made to investigate the problem.

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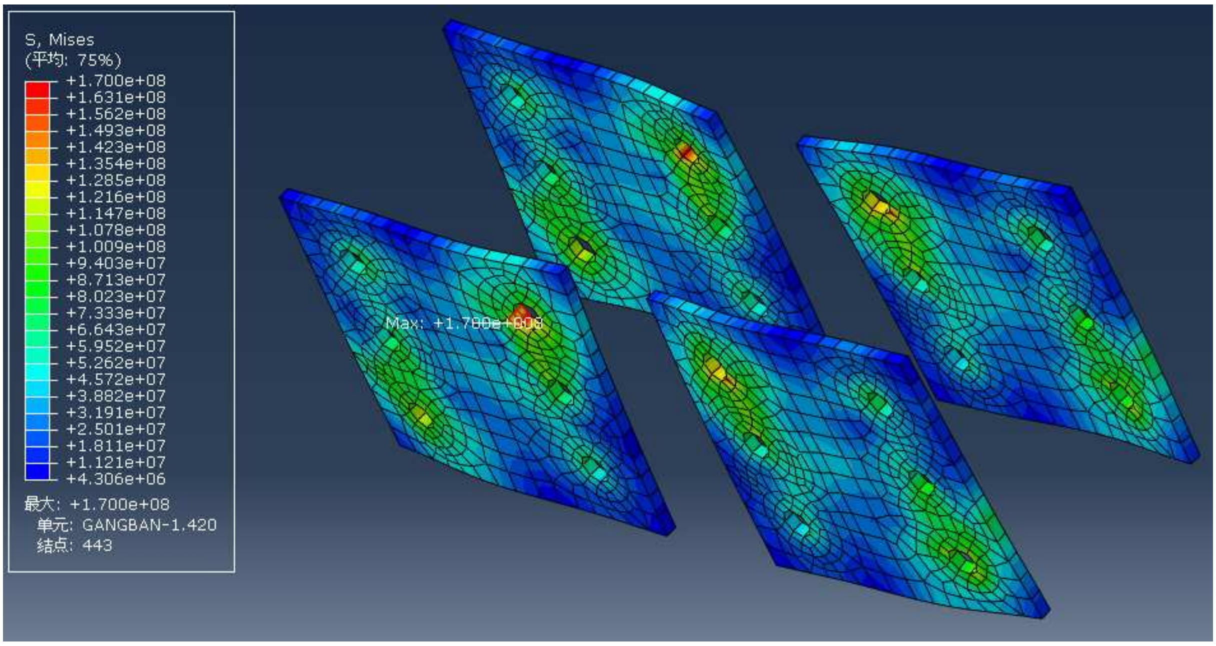
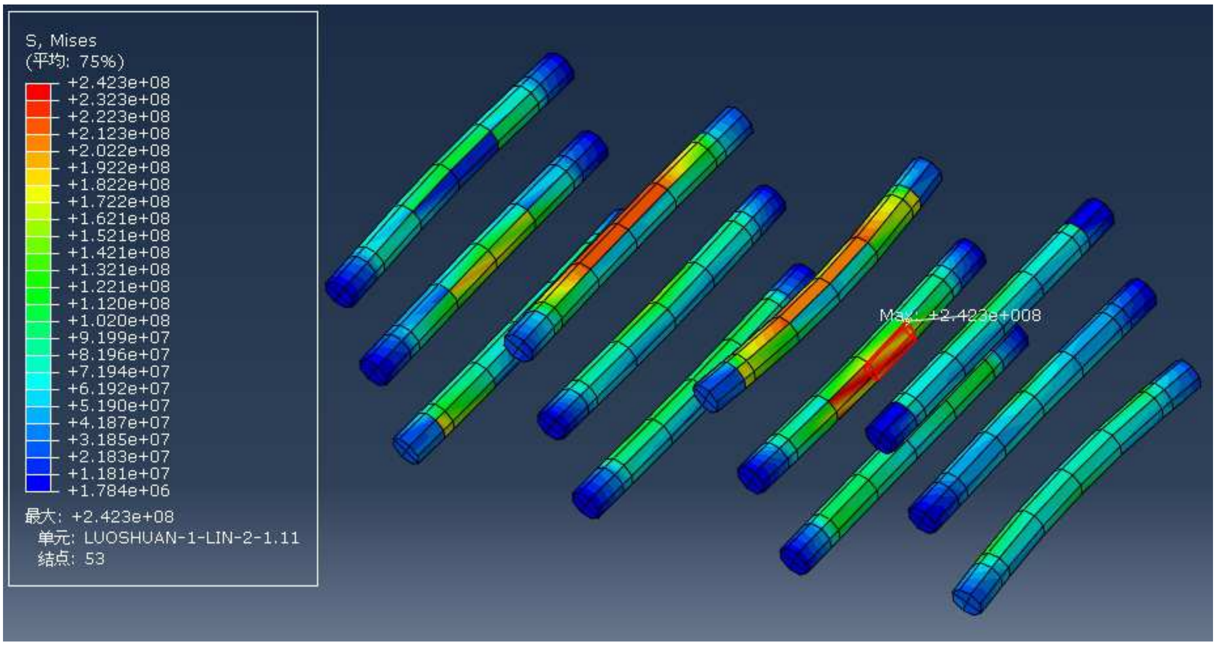
8 Appendix

Appendix A: Load controlled

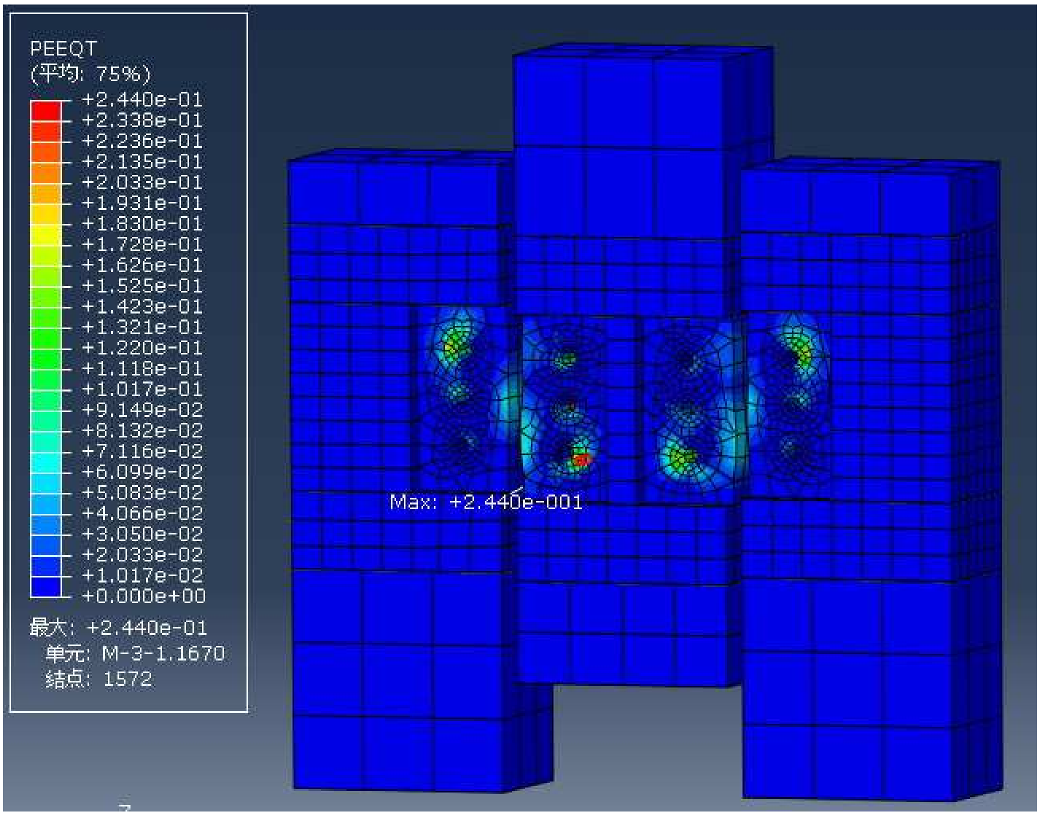
- Stresses in the plates



- Stresses in the bolts

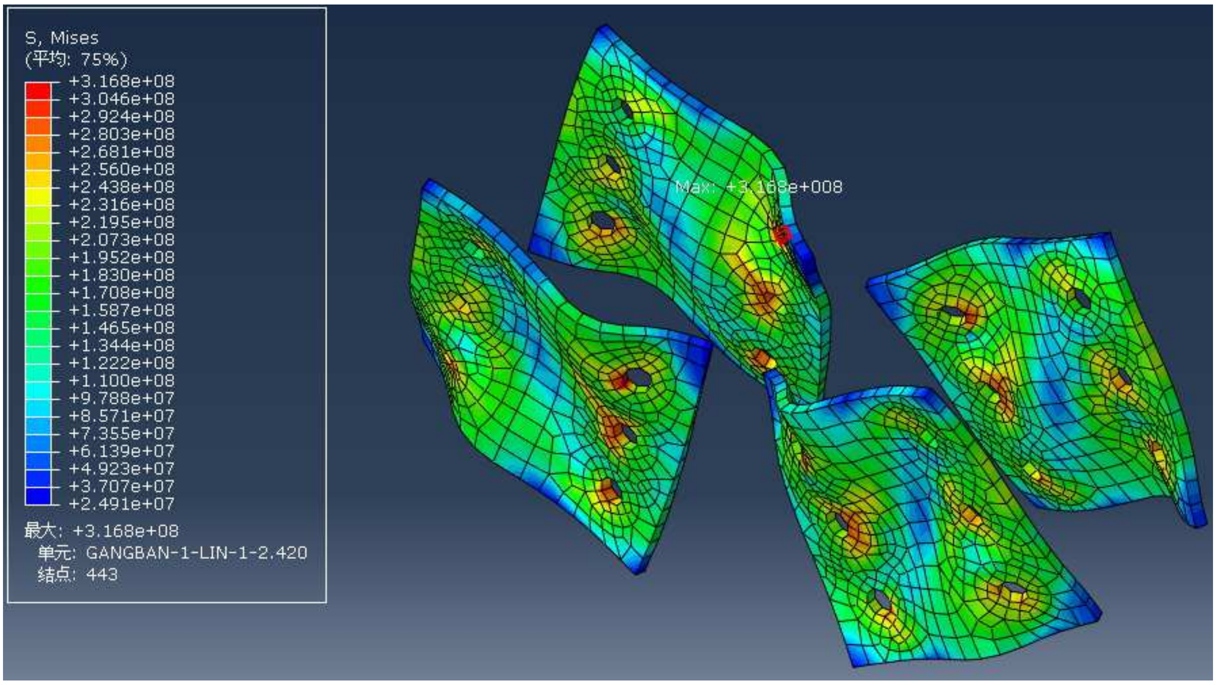


- Stresses in the concrete

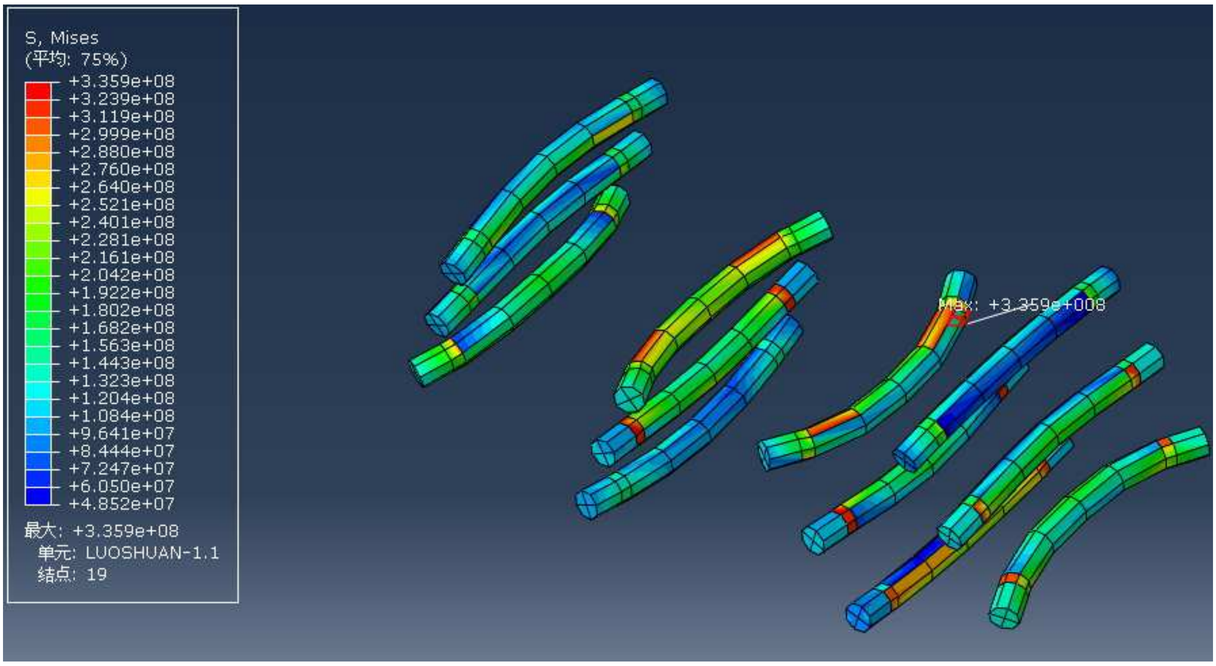


Appendix B: Displacement controlled

- Stresses in the plates



- Stresses in the bolts



- Stresses in the concrete