Saimaa University of Applied Sciences
Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering

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CONNECTIONS OF WALL PRECAST CONCRETE ELEMENTS

Thesis 2018

Abstract

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Connections of wall precast concrete elements, 49 pages, 4 appendices
Saimaa University of Applied Sciences
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This study was commissioned by Pöyry Finland Oy. The purpose of the thesis was to study how to choose the right capacity of the fixing devices and to research different types of connections of the wall precast concrete elements. The thesis is concentrated on three types of fixing devices: wire loops for vertical shearing joints, wall shoes for vertical tension and reinforced bars for horizontal forces in horizontal joints. Also, the effect of the wind load on the precast concrete building was shown.

The first part of the thesis is theory. It considers the way of load transmitting, including the wind load, and types of connections. The theoretical part was gathered from Russian and European design literature and Internet sources. The second part is a preliminary calculation to choose the suitable connections. The calculation was done in accordance with Eurocode standards and corresponding design literature.

The results of the thesis show the preliminary calculation method of choosing the fixing devices. Also, the thesis shows different types of fixing devices and how the wind load affects at the structure and the force calculations.

Keywords: connection, wire loops, wall shoe, reinforced bars, precast concrete building

Table of contents

1 Introduction

The objective of the study is to research connections of wall precast concrete elements. Also, the objective of the study was to analyze how the wind load affects the force calculations of the precast concrete building.

The thesis presents different types of connections of the wall precast concrete elements. For future ability to apply the knowledges, received during the researching, the thesis is concentrated on three types of fixing devices: wire loops for vertical shearing joints, SUMO wall shoes for vertical tension and reinforced bars for horizontal forces in horizontal joints. Figure 1 shows the locations of three chosen fixing devices.

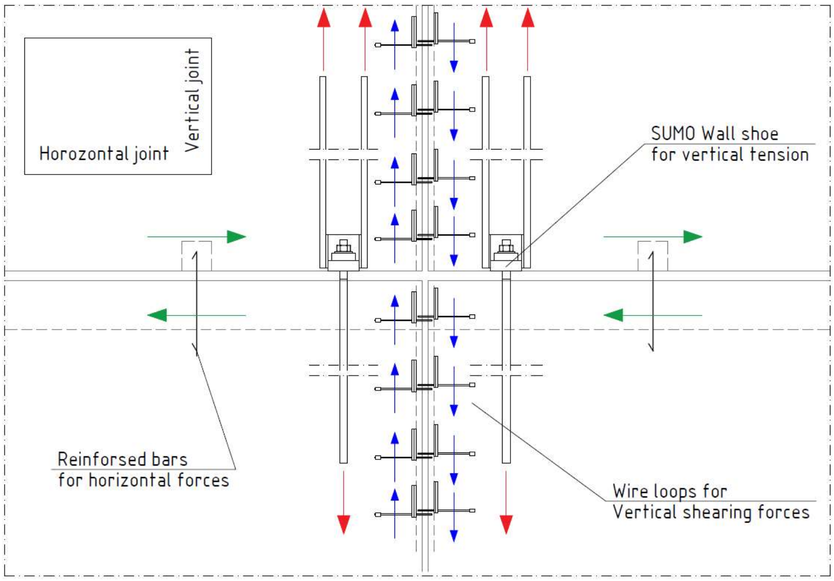


Figure 1. Three types of fixing devices

The other aim of the thesis is to study how to choose the right capacity of the fixing devices. For this it is needed to understand the work of fixing devices, what kind of loads they are designed for and how to collect the necessary loads. One more important thing is the right place and quantity of the devices into the elements, which transfer loads forward to foundations.

This topic is actual because nowadays the precast concrete buildings are very popular. They are easy and fast to erect and they have quite good quality because all elements are factory produced. The basic bearing element in this type of building are walls and the main question is the wall connections.

Walls can be classified as bearing and non-bearing walls. Bearing walls are used to support bridging components like floors, roofs or beams. Non-bearing walls are designed to carry own dead weight and the load from the wind. It means that the horizontal joints have to resist the weight of the wall elements above the level of the wall. Also, horizontal and vertical joints of the non-bearing walls resist the shearing forces caused by the wind. As an alternative, non-bearing facade walls might be fixed to the adjacent load bearing system. In that case the dead weight of each wall element and wind load are supported by the bearing system. The type of the wall also affects the choosing of connection.

There must be a conscious effort undertaken to ensure structural continuity when precast elements are put in place. The connections act as bridging links between the elements, forming together structural chains linking every element to the stabilizing elements, such as shear walls and cores. (1, p. 9)

Effective design and construction can be achieved by using suitable connections for all service, environmental and ultimate load conditions. The precast concrete elements can be joined by different ways, for example, by bolts, welding, reinforcing steel and concrete.

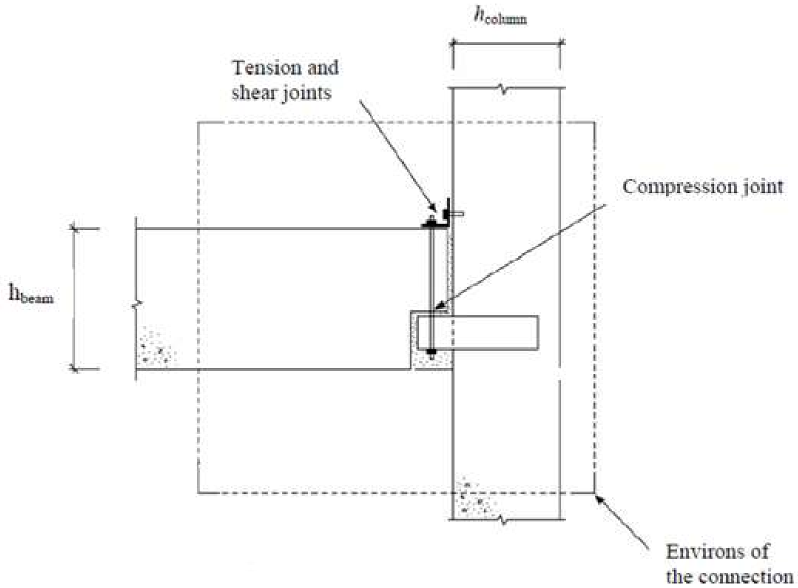


Figure 2. The difference between connection and joint (1, p. 3)

The role of the connections is not only to fix the elements together no less important roles are to ensure the structural continuity of the whole structure and to transfer forces between the precast elements when the system is loaded. The structural response will depend on the behavior and the characteristics of the connections.

Into a single connection there may be several loads that are transmitted by joints because of that it is necessary to understand the difference between a 'joint' and a 'connection'. A 'joint' is the interface between two or more structural elements, where the action of forces (e.g. tension, shear, compression) and or moments may take place. A 'connection' is an assembly, comprising one or more interfaces and parts of adjoining elements, designed to resist the action of forces or moments. Both of the terms are taken from FIB Bulletin 43: Structural connections for precast concrete buildings (1, p. 2). Figure 2 shows the difference between connection and joint.

2 Transmitting of the loads

All loads that effect on the building are classified according to Eurocode 1990 Basis of structural design (3, p.33):

- permanent actions (G), e.g. self-weight of structures, fixed equipment and road surfacing, and indirect actions caused by shrinkage and uneven settlements;
- variable actions (Q), e.g. imposed loads on building floors, beams and roofs, wind actions or snow loads;
- accidental actions (A), e.g. explosions, or impact from vehicles.

All of them are transmitted through the elements and the connections to the foundations.

The vertical loads are resisted by bridging elements (roof, floors, beams and stairs) and supporting elements (columns and load-bearing walls). There are self-weight of the structure, weight of equipment, people, snow and other. Horizontal forces are provided by stabilizing units that are capable to resist and transmit the forces to the foundation. There are wind, additional horizontal forces (tolerance and second degree effects), braking of crane truck in industrial building, forced forces (shrinkage, creep, thermal, expansion) and other.

Further, a residential building will be considered where the horizontal loads will be additional horizontal force and wind.

Wind is a short-term load and the source of the vibration in the structures. The reaction of the flexible structures depends on the natural resonant frequency. Rigid structures take the wind load as the static load. The wind load depends on not only the sizes of the structure and wind velocity it also depends on the design shape. The design shape is estimated by the aerodynamic coefficient. Also, it is important to have the required amount of information about the wind in the selected area. Otherwise it could lead to the failure of the building.

The walls have to be as the one structural unit consisting of interacting wall elements. Figure 3 shows the braced frames where the elements and connections form a chain to transmit the horizontal load to the ground.

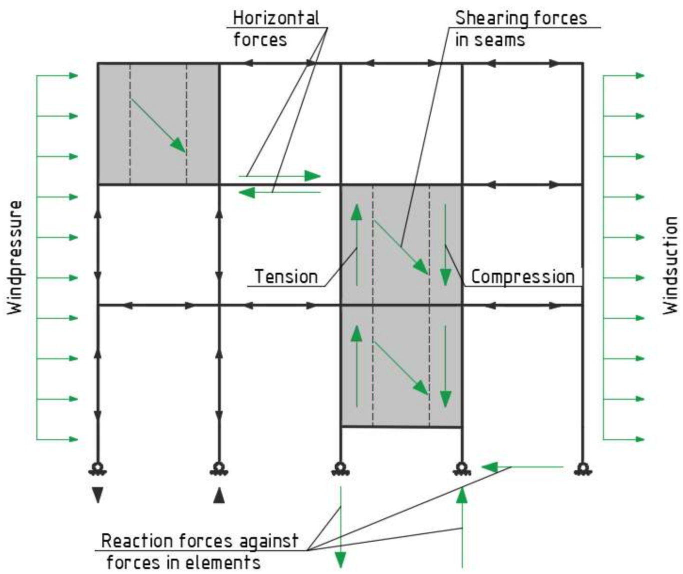


Figure 3. Transmitting of the loads

Shear walls are quite effective in resisting the horizontal loads, which act in the direction of the plan of the walls. But not only vertical systems transmit the loads. Floor slabs take a part in this process. The floor slabs that are supported by the walls also act as the stability elements. They transfer the horizontal forces into the shear walls. The reactions at each floor level are determined as shown in

Figure 4. There are two variants of the reactions in shear walls due to horizontal loads. Figure 4. A. shows the variant when the resultant H of the horizontal load goes through the shear centre S.C. In that case is only translation u_t. Figure 4. B. shows the variant when the resultant H of the horizontal load does not go through the shear centre. In that case is rotation u_r because of eccentric position of the stabilizing elements and translation u_t.

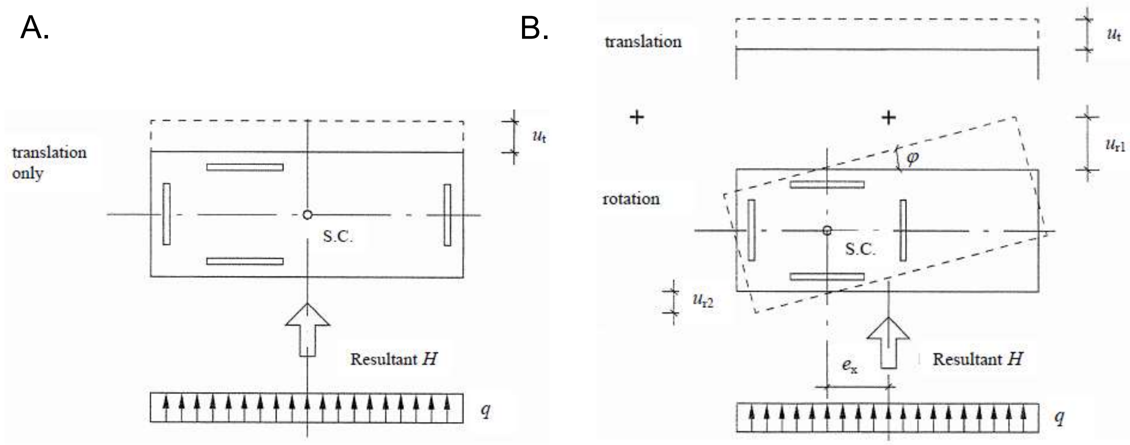


Figure 4. Reactions in shear walls (1, p. 11)

The position of the shear walls should seek to the variant shown in Figure 4. A. rather than to the variant shown in Figure 4. B. because of the big forces in the connections by the eccentricity in the latter one.

It is important to know the proportion of the distribution of the horizontal load to the relative stiffness of each wall in the building. The method of calculation is taken from Mosley, B., Bungey, J., Hulse, R. 2007. (11, p. 48-51)

For the systems with a symmetrical arrangement of the walls it needs to calculate the relative stiffnesses that are given by the second moment of area of each wall about its major axis such that

$$k_i = h \cdot b^3$$

where h is the thickness of the wall and b is the length of the wall. Then the force P_i distributed into each wall

$$P_i = H \cdot \frac{k_i}{\sum k}$$

Where H-resultant horizontal force.

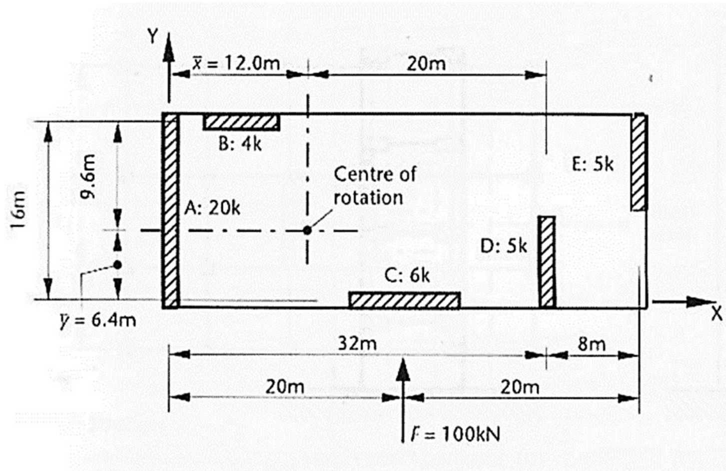


Figure 5. The systems with an unsymmetrical arrangement of walls (11, p. 51)

For the systems with an unsymmetrical arrangement of the walls, as shown in Figure 5, there is a torsional force on the structure about the centre of rotation (shear center S.C.) in addition to the direct forces caused by the translatory movement.

For the calculation the system is placed in the convenient axis and then determine the location of the centre of rotation by taking moments of the wall stiffnesses k about the axis

$$\bar{x} = \frac{\sum(k_x x)}{\sum k_x}$$

$$\bar{y} = \frac{\sum(k_y y)}{\sum k_y}$$

where k_x and k_y are the stiffnesses of the walls oriented in the x and y directions.

The torsional moment M_t on the group of shear walls is calculated as

$$M_t = H \cdot e$$

where e is the eccentricity of the resultant horizontal force H about the centre of the rotation.

The force P_i in case of the unsymmetrical arrangement of walls is calculated as the sum of the direct component P_d and the torsional rotation component P_t

$$P_i = P_d + P_t = H \cdot \frac{k_x}{\sum k_x} \pm M_t \cdot \frac{k_i r_i}{\sum(k_i r_i^2)}$$

where r_i is the eccentricity of the horizontal force F about the centre of rotation.

Thus, the buildings with the symmetrical arrangement of the walls are simpler for calculation and have less value of the loads in the connections. However, functional and architectural requirements do not always allow to do the building simpler.

3 Types of connections and joints

For the proper design of the concrete structures the movements due to temperature variations, concrete creep and shrinkage should be taken into account because they have influence on the design of the connection and on the structural design as a whole.

In many cases, the working experience would strongly influence on the connection design. There are several factors that seem unlikely for the engineers with little working experience:

- The stability of the frame. Unbraced (hinged) structures need the moment resisting foundations, whereas braced (rigid) structures do not need it.
- Structural model. The quantity and position of the structural elements such as columns, walls, cores, and others may influence on the connection design.
- Fire protection to important bearings and rebars.
- Easiness and economy of manufacturing.
- The requirements of the temporary stability for the ability of the frame erection and the need for immediate fixing (for example torsional resistance at the end of beams during floor erection).
- The chosen method of making joints (grouting, bolting, welding and other).
- The type of bearings.

- The material of the element.
- Access to the connection also may influence on the design.
- Application conditions. A temperature difference between the outer and inner environment especially in winter time could influence on the curling of slabs and therefore on the connections. It should be taken into account in the process of the connection design.

The structural system, the arrangement of stabilizing units and the design of the connections must be made consistently and with the awareness of the intended structural behavior.

3.1 Welded joints

Welded joints are made by welding plates or rods with the embeds details in precast panels. It is a rigid joint that can provide stability in the process of the erection of the elements. Also, it provides only small deformation of the connection during the service life. Disadvantages of the joint with the plate are the eccentricity of the load transferred from one panel to the other and the difficulties with the grouting. Because of that, the joint with the rod is more popular and it needs a smaller quantity of steel. The welded joints should be corrosion protected. The joints should have the additional cover layer of paint for the protection. Also, welded joints should be protected from the fire. That kind of protection can be achieved by the grout or mortar. The quality of the welding could influence on the durability of the joint. The quality of the welding depends on the weather conditions and skills of the welder. Figure 6 shows the different types of welded joints. Figure 6. A. shows the welded joint with the rod. The rod is welded together with the loops of the free lengths of reinforcing bars. Figure 6. B. shows the welded joint with the plate. The steel plate is welded together with two other steel plates that are used as the embedded details. Figure 6. C. shows the welded joint with anchor plate. The joints that are shown in Figure 6. A. and 6. B. are used in Russia. The joint shown in Figure 6. C. is used in Finland.

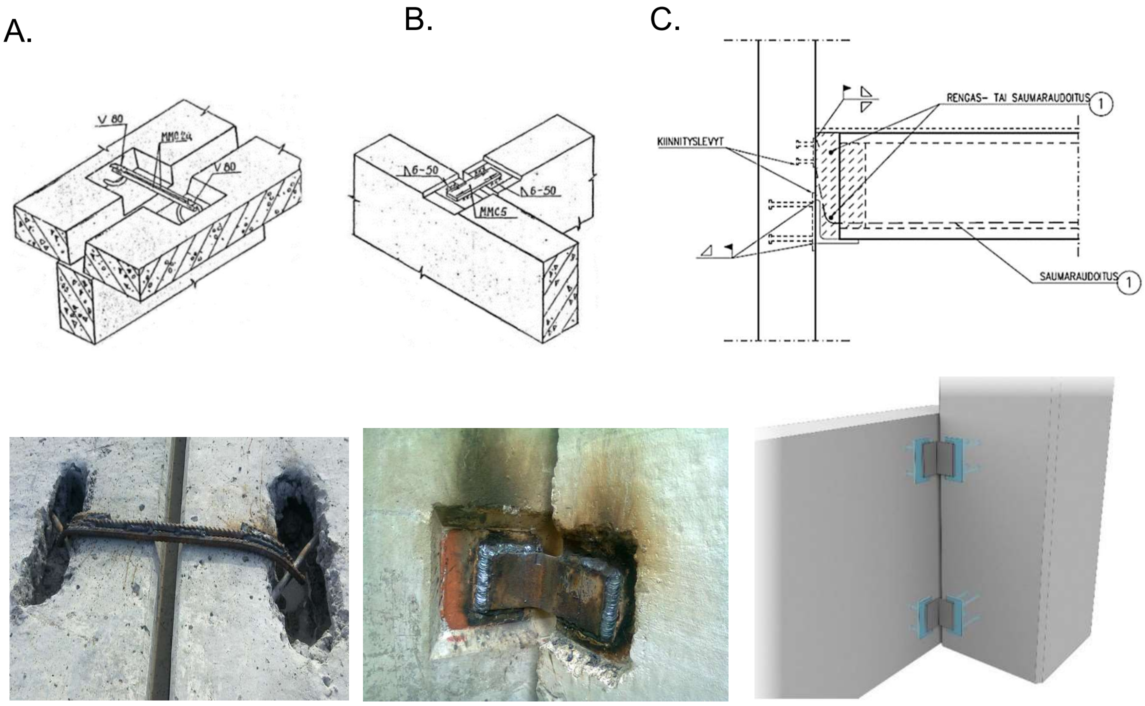


Figure 6. A. Welded joint with rod (17); B. Welded joint with plate (17); C. Welded joint with anchor plate (8)

3.2 Bolted joints

Bolted joints are made by fixing the embedded details in the precast panel and the special plates by the bolts. That type of joint is used in the areas where it is impossible to use welding (for example areas with extremely cold temperature). Bolted joints require high accuracy in the panel sizes and location of the embedded details. The possibility of adjustment during the installation creates favorable conditions for the installation of the elements strictly in the design position. But at the same time, it should be mentioned that this type of joint is quite deformable because of the holes in the plates. The joint starts to work only after little deformation of the connection. The problem can be solved by the tension-control bolts. Also, the corrosion and fire protection should be taken into account. Figure 7 shows two different types of bolted joints. Figure 7. A. shows a bolted connection with the plate. It is the connection of the outer and inner walls. Figure 7. B. shows the bolted connection of two inner walls.

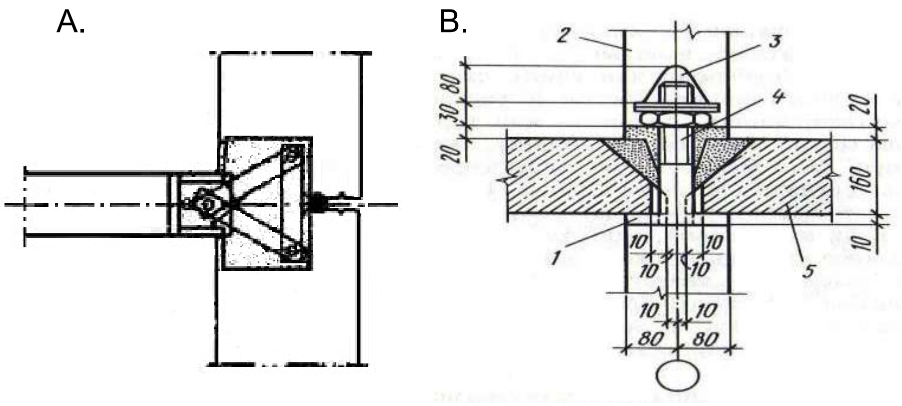


Figure 7. A. Bolted connection of the walls with the plate; B. Bolted connection of the walls (12)

Bolted connection with the plates is used in Russia. Bolted connection without the plate, as shown in Figure 7. A is used in Russia and in Finland also. The example of that kind of a connection is the wall shoes.

3.3 Mechanical interlocking joint

It is the system made by a special mechanism and then grouted. There are two different types of them.

The first is called "chigik-lovitel". It was popular in Russia at the end of the 20th century. The system consists of a steel cylindrical element and an embedded detail that work as a socket. It is a rigid joint that allows erecting elements without any temporary braces. It reduces the cost of the erection process. The rigid mechanical interlocking joint is quite universal because it allows to design X-, T-, L-shaped connections of precast panels. Usually, it is used in the inner walls connections because of the difficulties with the hermetic of the joints.

The second is loop connections with the steel braces. The braces are inserted into loops of the free lengths of reinforcing bars. Then the joint is grouted. The concrete of the grouting and braces influence on the strength of the joint. The main disadvantage is the deformation in the process of erection. That kind of a joint has a smaller labor intensity than the welded joint but requires more steel.

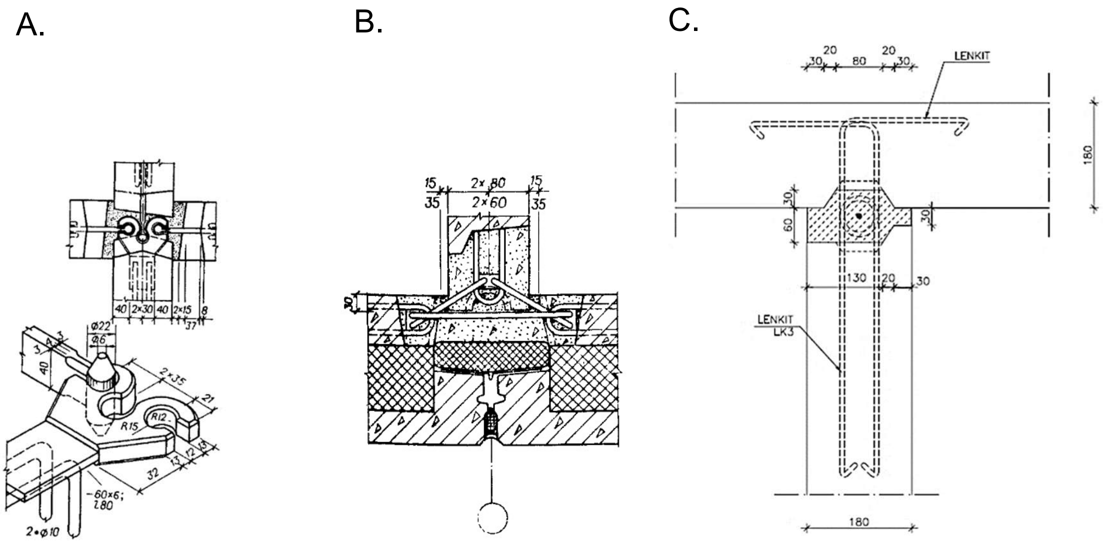


Figure 8. A. "Chigik-lovitel"; B. Loop connections with steel braces (15, p. 24,27); C. Loop connections with vertical rebar (8)

Figure 8 shows the types of mechanical interlocking joint. Joints that are shown in Figure 8. A. and Figure 8. B. are used in Russia. Joint that is shown in Figure 8. C. is used in Finland.

3.4 Shear key joints

There are joints without any steel elements. They consist of two panels that have a special shape of the abating end called "Dovetail". Then they are grouted by concrete. The joint can be done in both vertical and horizontal directions of the wall. That kind of joints have a good stiffness. The shear key joints have a small crack resistance. It reduces heat and sound insulation. One of the main disadvantages is possibilities of the water sorption through the grouting concrete. The special shape of the abating end is a decision to avoid the water sorption. Figure 9 shows the shear key joint. It is possible to see the "Dovetail" and the cross section with clearly visible protection against the water by the walls.

A. B.

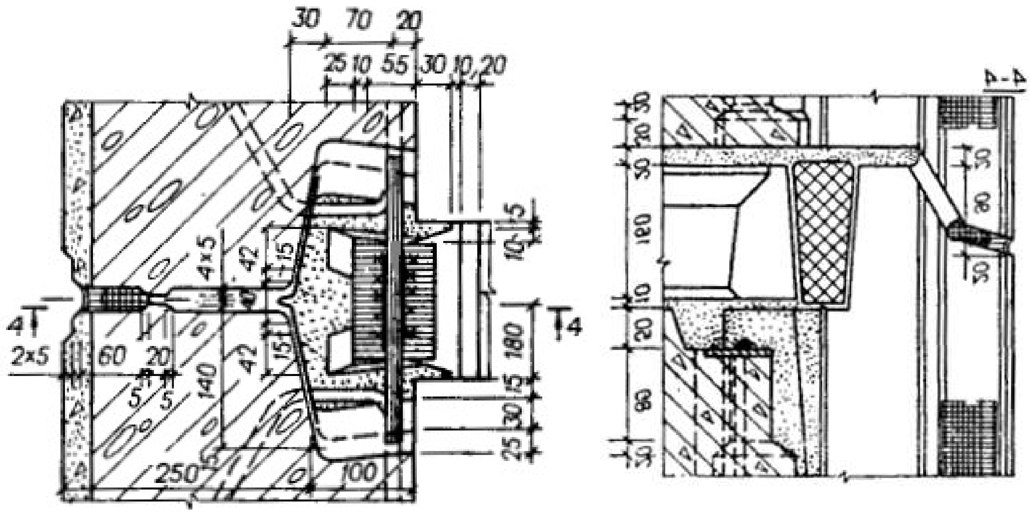


Figure 9. Shear key joint A. in plan; B. cross-section (15, p. 24)

Shear key joint was used in Russia but it is not used anymore.

5 Vertical joints of the wall panels. Wire loops connection

As it was said in the beginning the thesis concentrates on few types of connections. For vertical joints wire loops were chosen. They are commonly used in Finland and just started to be used in Russia.

Wire loops joint is some kind of a mechanical interlocking joint. That kind of a joint does not require welding. Wire loops are made as rope steel loops in special boxes. Wire loops boxes are placed into the formwork before the panel is cast. It is important to follow the rules of minimum distance between them and between the wire loops and the end of the panel. When the concrete has enough stress development the formwork is removed. Then the protection tape is also removed from the wire loops and they are opened by a hammer or a pin.

The boxes with the wire loops in one panel should have the pair of boxes with the wire loops in the other panel that are connected with the first. Then the vertical rebar is installed into the loops in that joint. The minimum diameter of the rebar is 12 mm. Figure 10 shows wire loops connection. There are two types of connection. The first is the connection of two outer walls. The second is the connection of outer and inner walls.

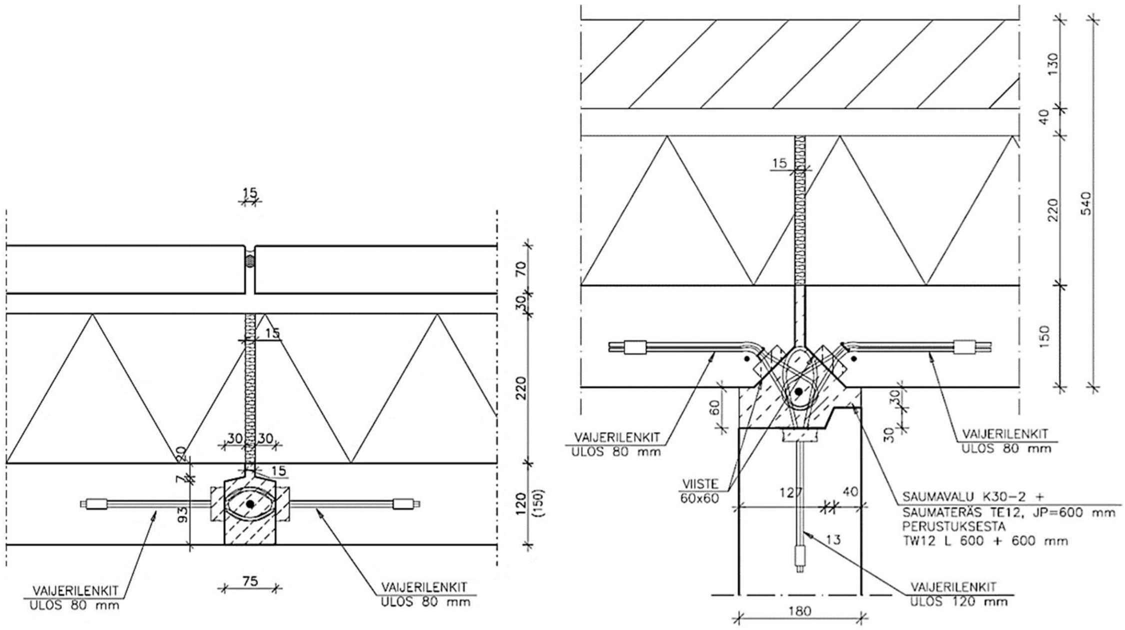


Figure 10. Wire loops connection (8)

The joint resists vertical shear force together with the concrete grout in the joint. Wire loops and bar in that case take the tension. The grouting concrete that works as a shear key takes the compression. The values of the tension and compression depends on the angle ϑ between the top of the one wire loop box and the bottom of the other wire loop box. The tension load is transferred to the overlapping cable loops. Figure 11 shows transmitting of the vertical shear force across the joint. The dimensions of the boxes can be different.

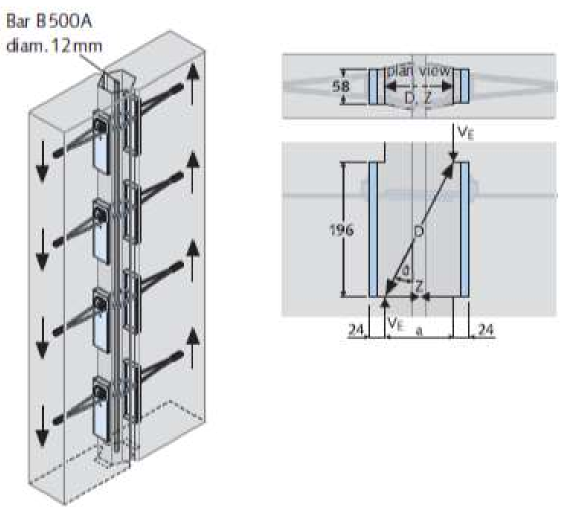


Figure 11. Vertical shear force in wire loops connection (10, p. 12)

Shear resistance in the direction perpendicular to the wall panels surface depends on the shape of the cross-section of the joint and reinforcement of the panel around the joint. As shown in Figure 12 the concrete strut is formed between the flanks of the opposing precast elements. The tension load is transmitting to the overlapping wire loops.

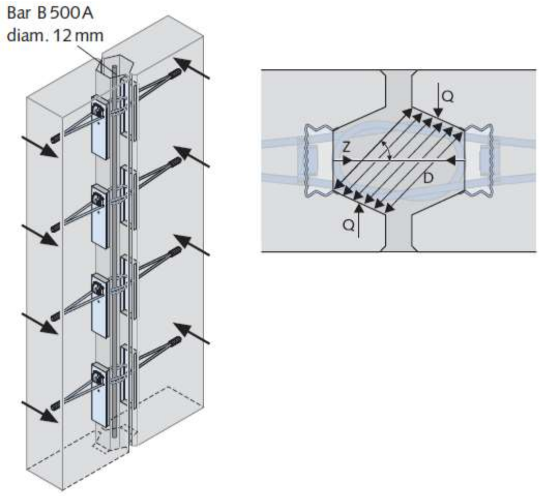


Figure 12. Perpendicular shear force in wire loops connection (10, p. 13)

Also, the wire loops transfer the tension perpendicular to the join. It results from the overlap of the wire loops. When the wire loops are tensioned they produce the local compression in the grouting fill. The compression loads are transmitted to the grout fill. The tension is taken by a vertical reinforcement bar. It is not recommended to use wire loops for the tension. Figure 13 shows the tension perpendicular to the joint in the wire loops connection.

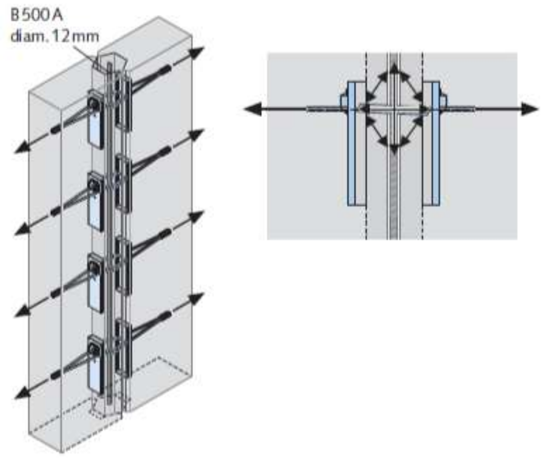


Figure 13. Tension perpendicular to the joint in wire loops connection (10, p. 12)

In the thesis only the case of the vertical shear force is considered.

During the process of design, it should be taken into account that the concrete grout shall have minimum the same compression strength as the concrete of the wall panels, minimum C25/30 (14). The resistance of the connection is defined according to the loop spacing and compression strength of the grouting concrete in the joint. It is assumed that no forces parallel to the wall panels and the loops effect to the joint. Only the shear forces in the vertical joint are taken into account.

The wire loops connection does not design for the seismic or dynamic strains that exceed the deformation capacity of the grouting concrete in the joint. Also, they must not be used as the lifting loops.

The wire loops are made of steel and because of that, the concrete cover thickness should be enough. In this case, they can be used in fire-resistant load bearing walls.

6 Horizontal joints of the wall panels. Wall shoes and reinforced bars

6.1 Wall shoes

The connection is the system that consists of wall shoes and anchor bolts. Wall shoes are commonly used in Finland and just started to be used in Russia. Wall shoes include the steel box with the anchor bars welded to the box, the nut, and the washer. Wall shoes are placed into the formwork together with the main and supplementary reinforcement. They are placed to the bottom part of the wall. Figure 14 shows the right location of the bolt shoes.

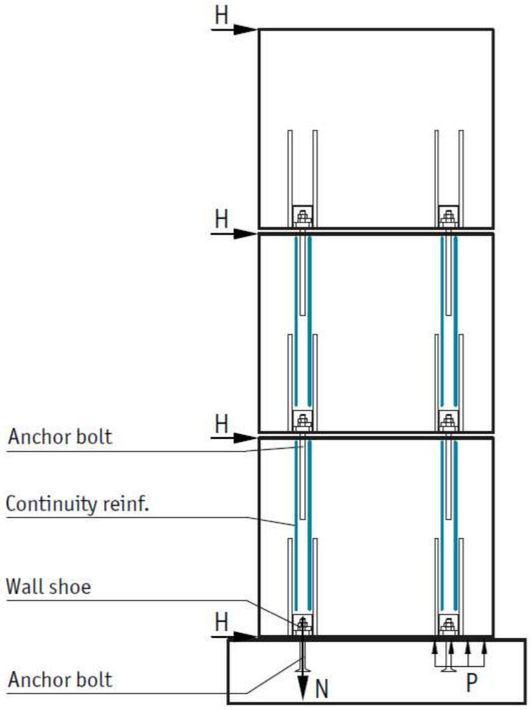


Figure 14. Bolt shoes joint (13, p. 5)

The connection is made by fastening the anchor bolts to the wall shoes by using nuts and washers. The bolting connection has sufficient assembly tolerance for adjusting the wall into the correct position. The walls should be supported with temporary braces. It is important to check the right position of the elements before the nuts are tightened by a slog ring spanner and a sledgehammer. Then the connection and the recesses of the boxes are grouted. The grout should be non-shrinking type. Figure 15 shows the process of erecting the walls with the wall shoes.

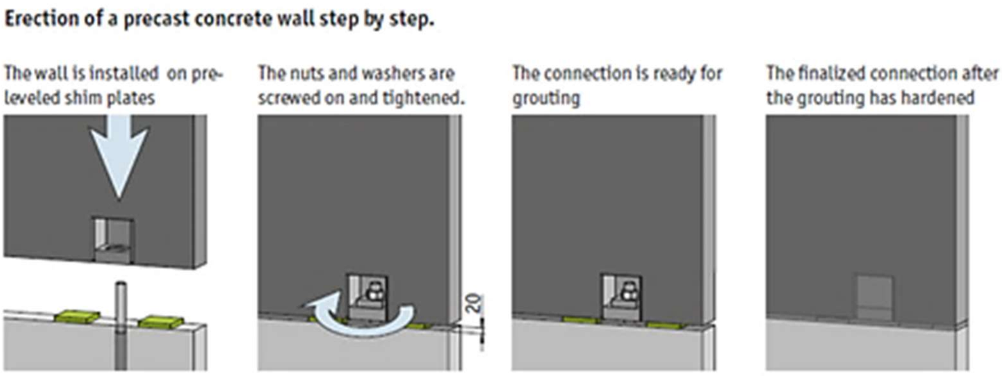


Figure 15. The process of erecting the walls with the wall shoes (13)

Wall shoe is designed to transfer the tensile forces between the wall elements. In the intermediate precast wall elements, the tensile forces are transferred from the bottom of the wall element (wall shoe) to the top of the next wall element (anchor bolt) by vertical continuity reinforcement (rebar B 500B lapped with wall shoe and anchor bolt). The tensile forces must be determined within the global analysis of the structure in accordance with the relevant design standards (13). A suitable model of the wall shoe and anchor bolt are selected so that they have the sufficient resistance compared to the design value of tensile force in the joint. The compressive forces are transmitted by the grouting of the joint. The connection is designed to carry static loads.

Wall shoes are made of steel because it is important to remember fire resistance. The concrete cover of the anchor bolts and the anchor bars of the wall shoes should be at least equivalent to the concrete cover of the reinforcement of the precast wall elements (13). If it is not enough it could be increased by increasing the thickness of the panel.

The advantage of the connection is the ability to transfer tensile forces immediately after elements are erected.

6.2 Reinforced Bars

The joint consists of the partly embedded steel bars. They work as the dowel and transfer shear force in the horizontal joint. That kind of a joint is used in Finland and not used in Russia. Figure 16 shows the joint.

There are several failure models. They depend on the strength and dimensions of the steel bar and the position of the bar relative to the element boundaries. The failure models are taken from FIB Bulletin 43: Structural connections for precast concrete buildings (1, p. 203): A weak bar in a strong concrete element might fail in shear of the bar itself. A strong steel bar in a weak element or placed with small concrete cover will more naturally result in splitting of the element itself. However, when the bar is placed in well confined concrete (large concrete covers) or the splitting effects are controlled by properly designed splitting reinforcement, the dowel pin will normally fail in bending by formation of a plastic hinge in the steel bar at some distance below the joint face.

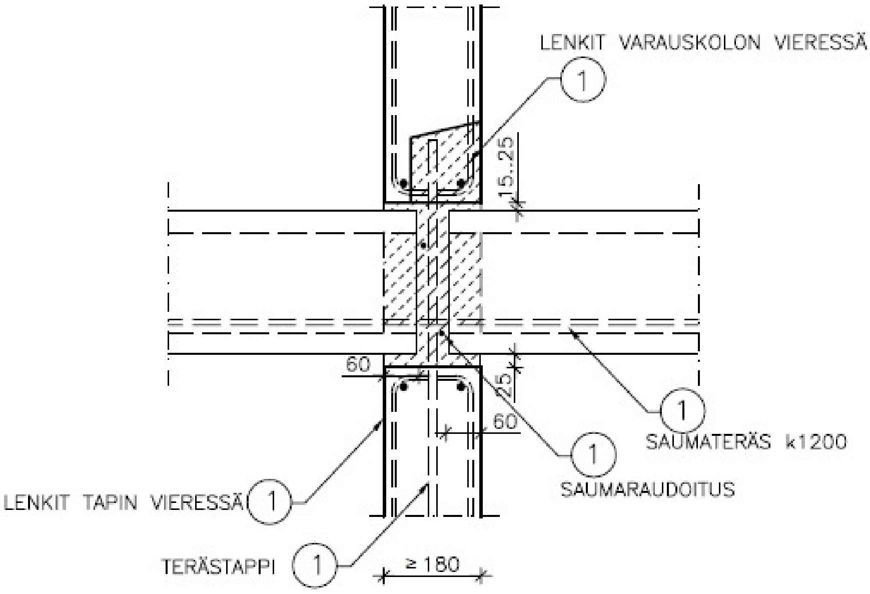


Figure 16. Reinforced bar joint (8)

When a joint between the two connected elements has a certain width, it may result in eccentric loading of the dowel pin, as shown in Figure 17.

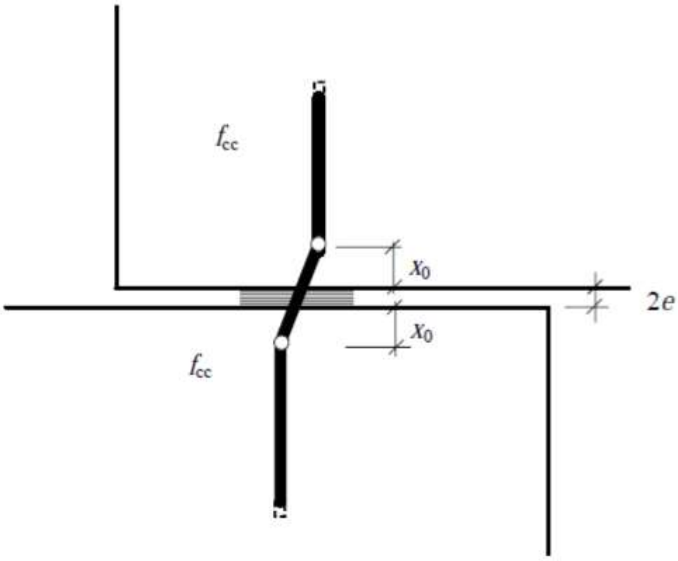


Figure 17. Double-sided plain dowel pin across a joint of a certain width (1, p. 214)

For the future ability to choose the right quantity of the reinforced bars, it is necessary to calculate their capacity. The diameter of reinforced bars is 16 mm.

The method of calculation is taken from FIB Bulletin 43: Structural connections for precast concrete buildings (1, p. 210).

The shear capacity of the dowel can be determined as

$$F_{vRd} = \propto_0 \cdot \propto_e \cdot \emptyset^2 \sqrt{f_{cd} \cdot f_{yd}} = 1 \cdot 0{,}867 \cdot 0{,}016^2 \sqrt{17 \cdot 10^6 \cdot 434{,}7 \cdot 10^6} = 19080\ N$$

$$= 19\ kN$$

where $\propto_0$ is a coefficient that considers the bearing strength of concrete $\propto_0 = \sqrt{\frac{\beta_c}{3}}$, can be taken as $\propto_0$=1,0 in design.

$\propto_e$ is a coefficient that considers the eccentricity

$$\propto_e = \sqrt{1 + (\varepsilon \cdot \propto_0)^2} - \varepsilon \cdot \propto_0 = \sqrt{1 + (0{,}459 \cdot 1)^2} - 0{,}459 \cdot 1 = 0{,}867$$

An eccentricity factor ε is defined as

$$\varepsilon = 3 \frac{e}{\emptyset} \sqrt{\frac{f_{cc}}{f_y}} = 3 \cdot \frac{0{,}01}{0{,}016} \sqrt{\frac{30 \cdot 10^6}{500 \cdot 10^6}} = 0{,}459$$

Where f_{cc} is compressive strength of concrete, cube (C 25/30)

f_y is Yield strength of reinforcement

e=10 mm is eccentricity

To better understand the process of the reinforced bar's design, dependencies of the shear capacity of the dowel on concrete strength and diameter of the dowel are calculated. Chart 1 shows dependence of the shear capacity of the dowel (diameter 16 mm) on the concrete strength. It is visible that the higher the concrete strength the bigger the shear capacity of the dowel. Chart 2 shows dependence of the shear capacity of the dowel on the diameter of the dowel. It is visible that the bigger the diameter of the dowel the bigger the shear capacity of the dowel.

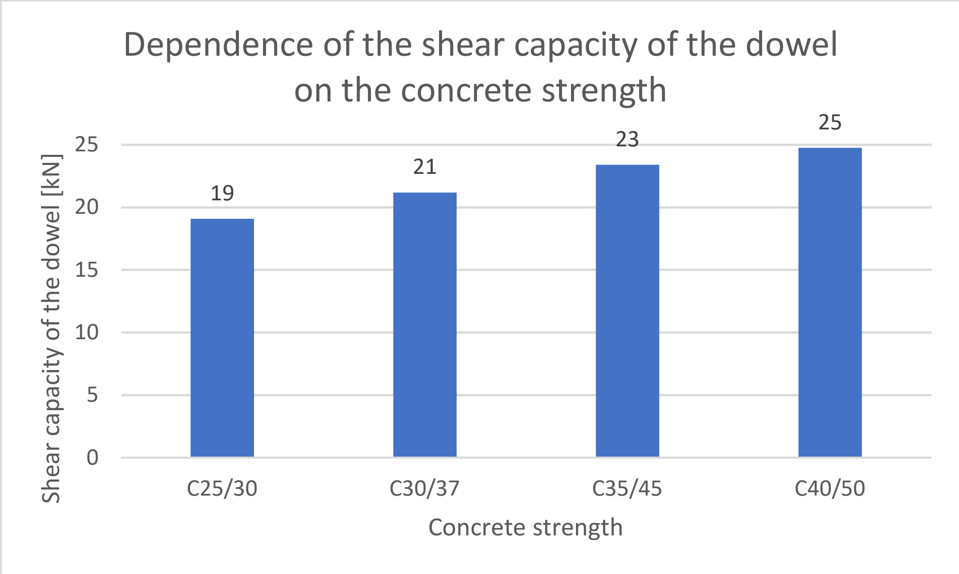


Chart 1. Dependence of the shear capacity of the dowel on the concrete strength

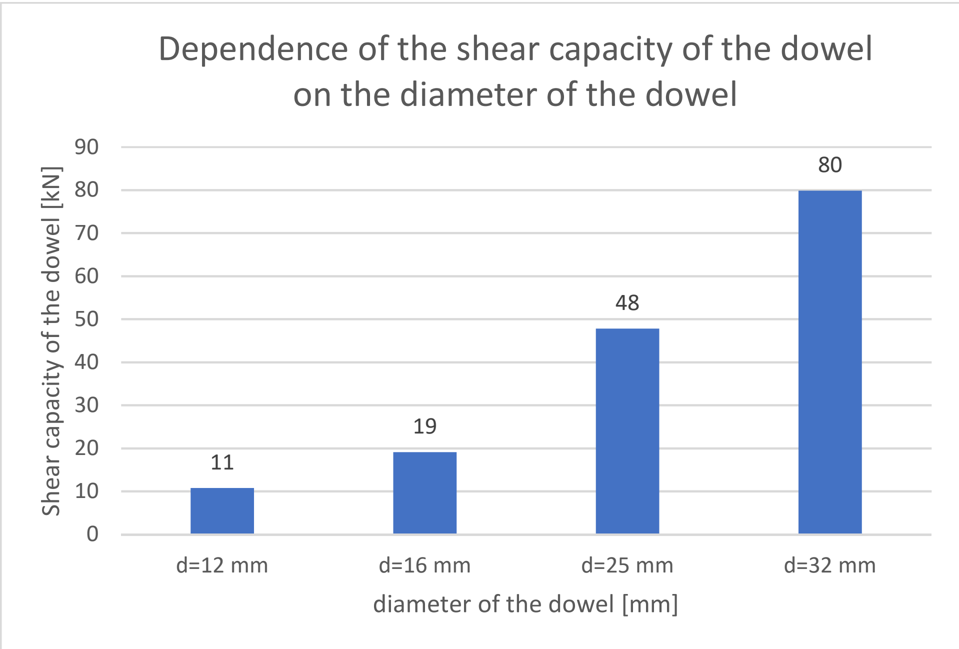


Chart 2. Dependence of the shear capacity of the dowel on the diameter of the dowel

7 Initial data for design

A five-storey precast concrete building was chosen for the further calculations. The overall dimensions of the building are height 17,5 m; length 33,6 m; width 12,0 m. The height of the storey is 2,8 m. The height of the attic is 2,2 m and the height of the parapet is 1,3 m. Also, the building has a cellar. The height of the cellar is 2,3 m. The outer and inner walls are load-bearing. The outer walls are sandwich panels that have the thickness of 460 mm (150 mm - reinforced concrete, 240 mm - insulation, 70 mm - concrete cover layer). The inner walls are 160 mm. The thin-shell slabs have the thickness of 160 mm. All elements are precast concrete elements. Figure 18 shows the plan of the typical floor of the building.

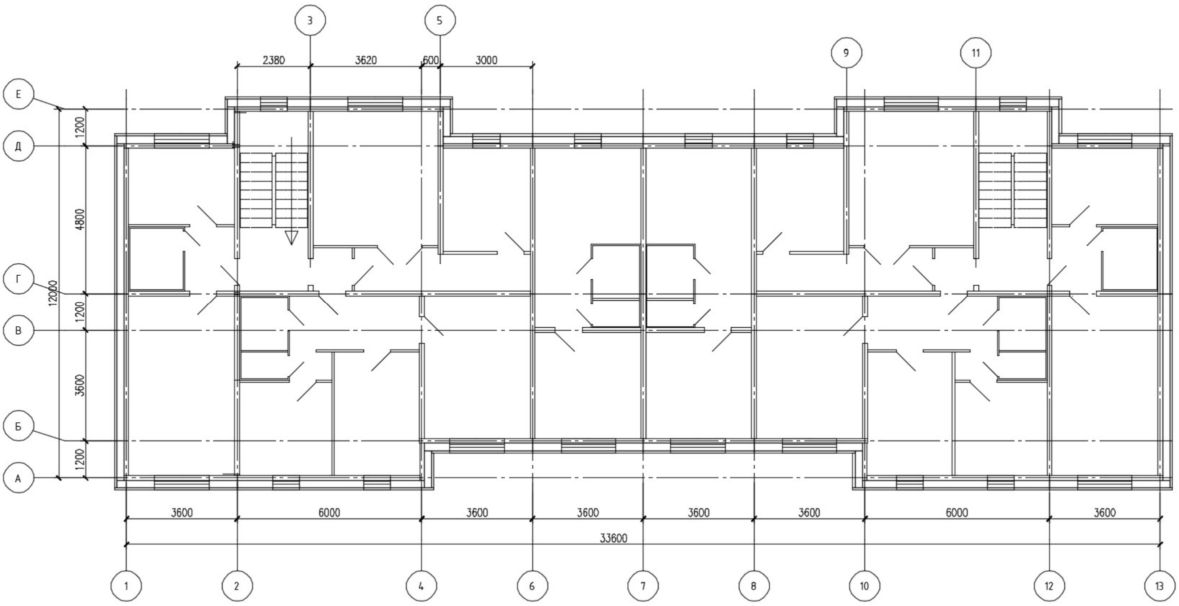


Figure 18. Plan of the typical floor of the building

The stiffening diaphragm was chosen to show the work of fixing devices and the effect of wind load. It is situated at the axis number 2. The stiffening diaphragm consists of one outer wall (WALL 1) and two inner walls (WALL 2 and WALL 3).

8 Load calculations

8.1 Self-weight of the structure

The self-weight includes the weight of walls, floor slabs, roof slabs and staircases. WALL 1 bears the weight of one slab on the right area. The slab is supported on two sides. WALL 2 bears the staircase and two slabs on the right area, they are supported on two sides. At the left area, WALL 2 bears two slabs that are supported on three sides. WALL 3 bears two slabs on the right area, they are supported on three sides and one slab on the left area that is supported on four sides. All slabs are precast. Slabs have simple supporting. Slabs are not continuous. They have hinged connection. Figure 19 shows the chosen stiffening diaphragm and the load distribution of the self-weight.

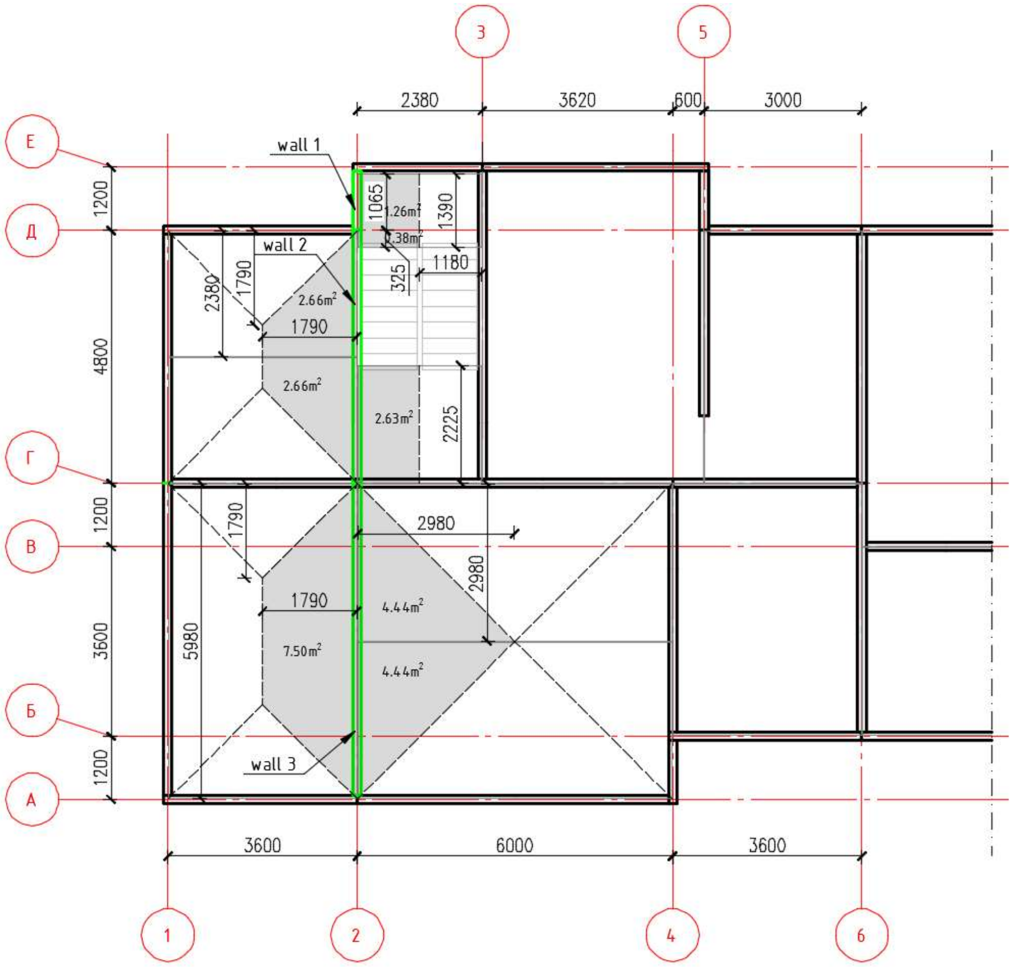


Figure 19. Load distribution of the self-weight

Also, it was chosen the imposed loads on floors, balconies and stairs in the building. The category of uses of the building is A “Areas for domestic and residential activities”. For the category A for floors and stairs the imposed load is q_k =2,0 kN/m^2.

8.2 Snow load

Snow loads on roofs for the persistent / transient design situations.

$$s_1 = \mu_i \cdot c_e \cdot c_t \cdot S_k = 0{,}8 \cdot 1{,}0 \cdot 1{,}0 \cdot 2{,}7 = 2{,}16$$

$$s_2 = \mu_i \cdot c_e \cdot c_t \cdot S_k = 0{,}96 \cdot 1{,}0 \cdot 1{,}0 \cdot 2{,}7 = 2{,}59$$

c_e=1,0 for Normal topography by the table 5.1 EN 1991-1-3

$c_t = 1{,}0$

Coefficient μ_i for Drifting at projections and obstructions $\mu_1 = 0{,}8$ and

$\mu_2 = \gamma \cdot \frac{h}{S_k} = 2 \cdot \frac{1{,}3}{2{,}7} = 0{,}96$. Figure 20 shows the snow loads in case of parapet.

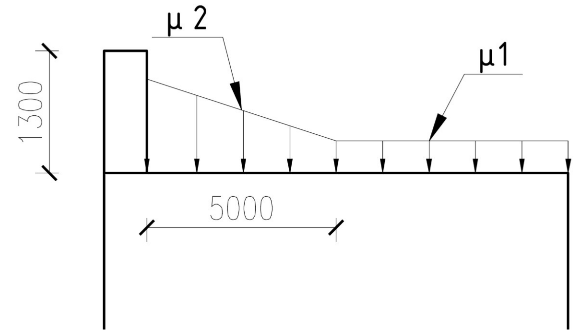


Figure 20. Drifting at projections and obstructions

S_k is 2,7 kN/m^2 for Lappeenranta in accordance with the NA EN1991-1-3.

8.3 Wind load

The basic wind velocity

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} = 1 \cdot 1 \cdot 21 = 21\, m/s$$

Where v_b is the basic wind velocity, defined as a function of wind direction and time of year at 10m above ground of terrain category II,

$v_{b,0}$ is the fundamental value of the basic wind velocity, $v_{b,0} = 21\, m/s$ for Mainland in the entire country in accordance with the NA EN1991-1-4

$c_{dir} = 1$ is the directional factor,

$c_{season} = 1$ is the season factor.

The mean wind velocity

$$v_m(z) = c_r(z) \cdot\ c_0(z) \cdot v_b = 0{,}684 \cdot 1 \cdot 21 = 14{,}36\, m/s$$

Where $c_r(z)$ is the roughness factor, $c_0(z) = 1$ is the orography factor.

Terrain category is IV (Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m) z_0=1 m, $z_{min} = 10$ m.

The roughness factor for $z_{min} \leq z \leq z_{max}$

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) = 0{,}234 \cdot \ln\left(\frac{18{,}6}{1}\right) = 0{,}684$$

Where k_r is terrain factor depending on the roughness length z_o calculated us-ing

$$k_r = 0{,}19\ (\frac{z_0}{z_{0,II}})^{0.07} = 0{,}19\ (\frac{1}{0{,}05})^{0.07} = 0{,}234$$

$z_{0,II}$=0,05 m (terrain category II).

Wind turbulence for $z_{min} \leq z \leq z_{max}$

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{4{,}914}{14{,}36} = 0{,}342$$

where σ_v is standard deviation.

$$\sigma_v = k_r \cdot v_b \cdot k_i = 0{,}234 \cdot 21 \cdot 1 = 4{,}914\ m/c$$

Peak velocity pressure

$$q_p(z) = c_e(z) \cdot q_b = 1{,}6 \cdot 275{,}63 = 441{,}0 \frac{kg}{m \cdot s^2}$$

where $c_e(z) = 1{,}6$ is the exposure factor, $q_b$is the basic velocity pressure.

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 = \frac{1}{2} \cdot 1{,}25 \cdot 21^2 = 275{,}63 \frac{kg}{m \cdot s^2} = 0{,}275\ kPa$$

The wind pressure acting on the external surfaces

$$w_e = q_e(z_e) \cdot c_{pe}$$

Where $q_e(z_e)$ is the peak velocity pressure, c_{pe} is the pressure coefficient for the external pressure.

For walls of rectangular plan buildings w_{e1} is for wind pressure, w_{e2} is for wind suction:

$$w_{e1} = q_e(z_e) \cdot c_{pe1} = 441{,}0 \cdot 0{,}8 = 352{,}80 = 0{,}35\ kN/m^2(+)$$

$$w_{e2} = q_e(z_e) \cdot c_{pe2} = 441{,}0 \cdot 0{,}53 = 233{,}73 = 0{,}23\ kN/m^2(-)$$

Wind forces

$$F_w = F_{we} + F_{fr} = c_s c_d \cdot \sum w_e \cdot A_{ref} + c_{fr} \cdot q_e(z_e) \cdot A_{fr}$$

For walls of rectangular plan buildings:

$$F_{w1} = 1 \cdot 0{,}35 \cdot (33{,}6 \cdot 18{,}6) + 0{,}01 \cdot 0{,}44 \cdot (2 \cdot 18{,}6 \cdot 12{,}0) = 222{,}59\ kN$$

$$F_{w2} = 1 \cdot 0{,}23 \cdot (33{,}6 \cdot 18{,}6) + 0{,}01 \cdot 0{,}44 \cdot (2 \cdot 18{,}6 \cdot 12{,}0) = 145{,}7\ kN$$

8.4 Calculation procedure for the determination of the vertical tension in the horizontal joint

The first step was to determine the actions. There was the self-weight, such as the weight of roof and floors (DL, floor) and the weight of walls (Dl, walls), and the imposed loads, such as snow load on the roof (SL), live load on floor (LL).

Then characteristic value of an action (N_k) and design value (N_d) was calculated. In accordance with EN 1990 (3) the design value

$$F_d = \gamma_f \cdot F_{rep}$$

with

$$F_{rep} = \Psi F_k$$

where F_k is characteristic value of the action,

F_{rep} is the relevant representative value of the action,

γ_f is a partial factor for the action which takes account of the possibility of unfavourable deviations of the action values from the representative values,

Ψ is either 1,00 or Ψ_0, Ψ_1, Ψ_2.

Then was done Ultimate limit states for the walls. The design value of actions was done in accordance to NA EN 1990 for set B.

The calculations are shown in Appendix 1.

Then the extra horizontal load for the wall because of eccentricity (H_k) was calculated as

$$H_k = N_d/150$$

Horizontal load H_k effects at the top of the wall and causes the moment (M_{kh}) at the bottom of the wall.

Also, it is necessary to calculate the wind. It was chosen the wind pressure because of bigger wind action WL1=0,35 kN/m^2 (WL2=0,23 kN/m^2 in case of the wind suction). Wind action has the horizontal load (H_{kw}) and the moment (M_{dw}) that acts at the bottom of the wall.

After that, the position of the wall shoes was decided. Two wall shoes per wall were chosen. The edge distance is R=0,3 m from both edges of the wall

$$z = B - R = B - 0{,}3 \cdot 2$$

where B is length of the wall.

Then the tension/compression load in the location of the wall shoe was determined. Tension/compression have the same value but the different directions.

$$N_{td} = M_{d,tot}/z$$

with

$$M_{d,tot} = M_{dw} + M_{dh}$$

Finally, the total load (tension/compression) in the horizontal joint was calculated. The total load consists of the tension/compression force and the own weight that reduce the tension.

$$N_{tot} = N_{td} + N_d$$

where N_d is the half of own weight.

Then it is necessary to calculate the proportional distribution of the forces.

8.5 Proportional distribution of the forces

The theory of proportional distribution of the forces was shown in Chapter 2 of the thesis. The building is symmetrical, therefore, calculation was done as for a symmetrical system. The walls were marked until the middle of the building as shown in Figure 21. WALL 1 was marked E, WALL 2 was marked D and WALL 3 was marked C.

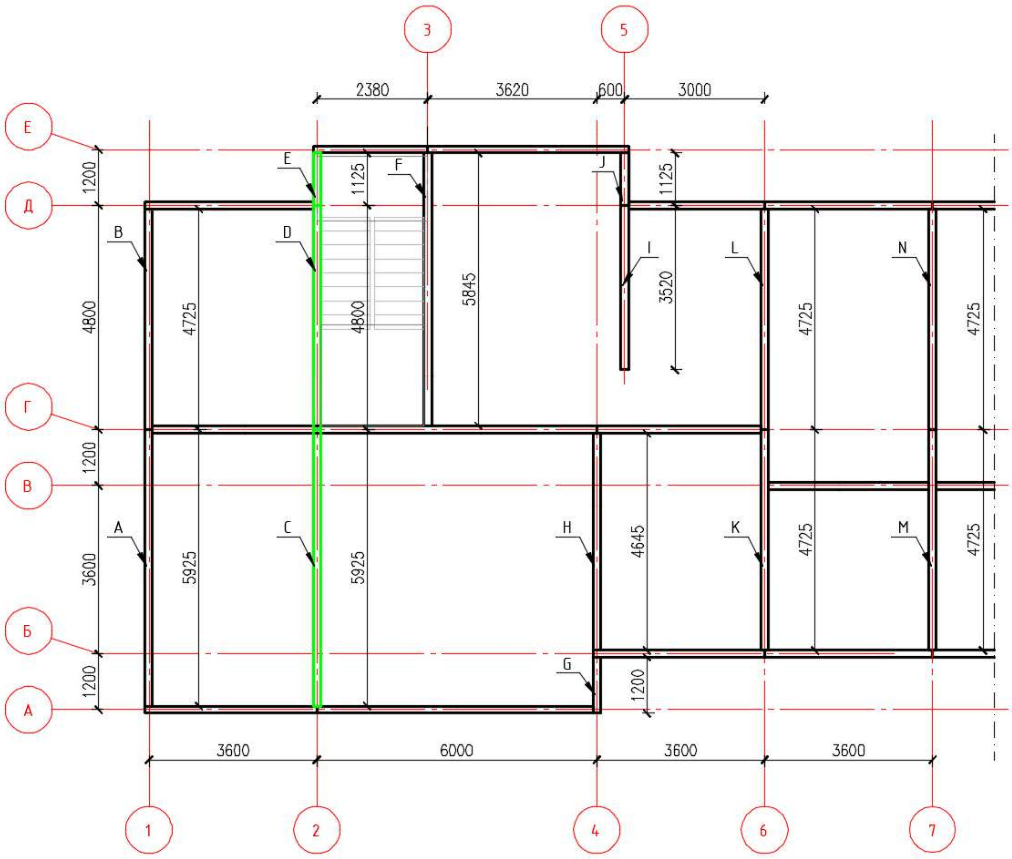


Figure 21. Marking of the walls.

The relative stiffness and the proportion were calculated for the building. Calculations are given in Table 1. The necessary stability walls are marked by yellow.

Table 1. Proportional distribution of the forces.

wall	B,m	t,m	k,i	k,i/Σ k	%
A	5,925	0,15	31,20	0,0760	7,60
B	4,725	0,16	16,88	0,0411	4,11
C	5,925	0,16	33,28	0,0810	8,10
D	4,8	0,16	17,69	0,0431	4,31
E	1,125	0,16	0,23	0,0006	0,06
F	5,845	0,16	31,95	0,0778	7,78
G	1,2	0,16	0,28	0,0007	0,07
H	4,645	0,16	16,04	0,0390	3,90
I	3,52	0,16	6,98	0,0170	1,70
J	1,125	0,16	0,23	0,0006	0,06
K	4,725	0,16	16,88	0,0411	4,11
L	4,725	0,16	16,88	0,0411	4,11
M	4,725	0,16	16,88	0,0411	4,11
N	4,725	0,16	16,88	0,0411	4,11
		Σ :	410,77	1,00	100,00

8.6 Choosing of the wall shoe

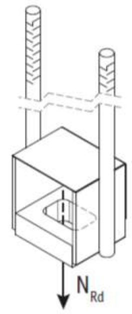
To choose the wall shoes the most unfavourable combinations of actions should be defined. For all three walls, the most unfavourable combination of actions was the combination with the live load as leading variable action and without the wind. The vertical loads reduce the tension that could be caused by the wind. Because of that, all horizontal joints are compressed.

The suitable Bolt shoes were chosen in accordance with Table 2 from Technical manuals SUMO Wall Shoe for bolted wall connections (13). The design value of the total force in a joint per one wall shoe N_{tot} must be smaller than the design value of the resistance of a wall shoe and a suitable anchor bolt N_{Rd}.

$$N_{tot} < N_{Rd}$$

Table 2. Design values of resistances of individual SUMO Wall Shoes for concrete grade C25/30 (13, p. 8)

Wall Shoe	Anchor Bolt	Washer	N_{Rd} [kN]
SUMO 16H	HPM 16	AL 16	62
SUMO 20H	HPM 20	AL 20	96
SUMO 24H	HPM 24	AL 24	139
SUMO 30H	HPM 30	AL 30	220
SUMO 39H	HPM 39	AL 39	383
SUMO 30P	PPM 30	AL 30	299
SUMO 36P	PPM 36	AL 36	436
SUMO 39P	PPM 39	AL 39	521
SUMO 45P	PPM 45	AL 45	697
SUMO 52P	PPM 52	AL 52	938



The most unfavourable combination of actions for all three walls and suitable fixing devices is shown in Appendix 2.

Also, the geometric and other parameters such as the minimum thickness of the wall, the minimum concrete cover, the minimum edge distance and the distance between the bolts shoes should be checked.

The influence of the edge distance R on the design value of the total force N_{tot} is shown in Chart 3. There are three variants of the edge distances R. It is visible that if the edge distance R is longer and the position of wall shoe is closer to the center of the wall, the design value of the total force is smaller than the maximum value.

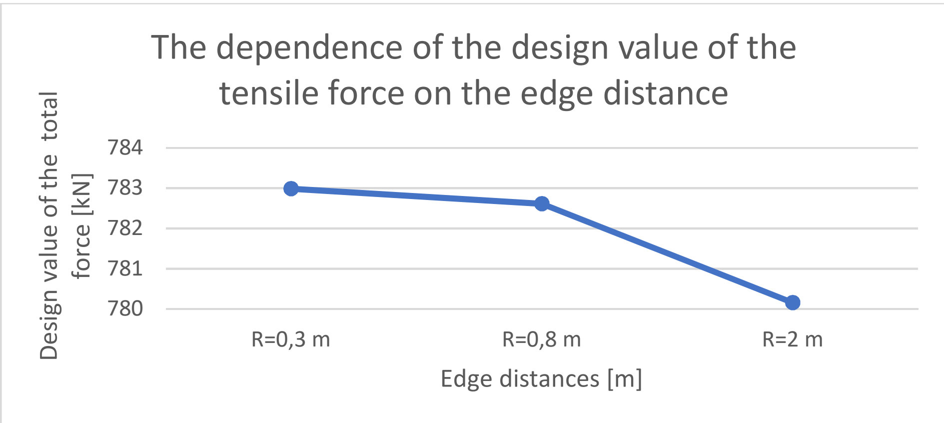


Chart 3. The dependence of the design value of the tensile force on the edge distance

8.7 Calculation procedure for the determination of the vertical shearing forces in vertical joint

The calculation procedure for the determination of the vertical shearing forces is quite the same as the calculation procedure for the determination of the tensile loads in the horizontal joint. The main difference is that the total moment M_{dtot} for the joint between WALL 1 and WALL 2 is calculated as for one wall that consists of two. Also, the proportional distribution of the forces should be recalculated, because of working WALL 1 and WALL 2 as one wall. The proportional distribution of the forces for that wall is 7,57%.

Then tension/compression load N_{td} is determined for the place of the joint. It means that z is determined as the distance between the edge of the wall and joint location. Then the total load V_{ed} is calculated as

$$V_{ed} = N_{td,\%} + N_d$$

Where $N_{td,\%}$ - tension/compression load in the location of the joint in percentage

N_d - half of own weight

8.8 Choosing of the wire loops

To choose the wire loops the most unfavourable combinations of actions should be defined.

The most unfavourable combination of actions was the combination with the live load as leading variable action and with the snow as accompanying variable action.

The design value of the shear force V_{ed} must be smaller than the design value of the shear resistance of wire loops V_{Rd}.

$$V_{Ed} < V_{Rd}$$

The design value of the shear resistance is defined by the spacing between loops and compression strength of the concrete grout of joint. Also, the minimum value of the thickness of the wall panel and joint width should be checked. Table 3 and Table 4 show the design shear resistance of the wire loops.

Table 3. Design Shear resistance VRd [kN/m] of PVL 60, PVL 80, PVL 100 and PVL 120 Wire Loop (14, p. 6)

Concrete strength (EC 2)	Spacing of loops [mm]										
	250	300	350	400	450	500	550	600	650	700	750
C25/30	153	132	116	105	96	89	83	78	74	70	67
C30/37	156	134	119	107	99	91	86	81	77	73	70
C35/45	158	137	122	110	101	94	88	83	79	76	73
C40/50	162	141	126	114	105	98	92	88	83	80	77
C45/55	165	144	128	117	108	101	95	90	86	83	80
C50/60	168	146	131	120	111	104	98	93	89	85	82

Table 4. Design Shear resistance VRd [kN/m] of PVL 140 Wire Loop (14, p. 6)

Concrete strength (EC 2)	Spacing of loops [mm]							
	350	400	450	500	550	600	650	700
C25/30	170	153	137	123	112	103	96	90
C30/37	185	165	148	134	121	111	103	97
C35/45	197	177	158	143	130	119	110	103
C40/50	209	188	167	151	138	128	117	109

Dependence of the design value of the shear resistance of wire loops on spacing of loops and concrete strength is shown in Chart 4 for PVL 60, PVL 80, PVL 100 and PVL 120. The bigger the spacing of loops the smaller the design value of share resistance. The higher the concrete strength the bigger the design value of share resistance. Therefore, wire loops with small spacing of loops and with high concrete strength of the grouting have the biggest design value of share resistance. The same could be said for PVL 140.

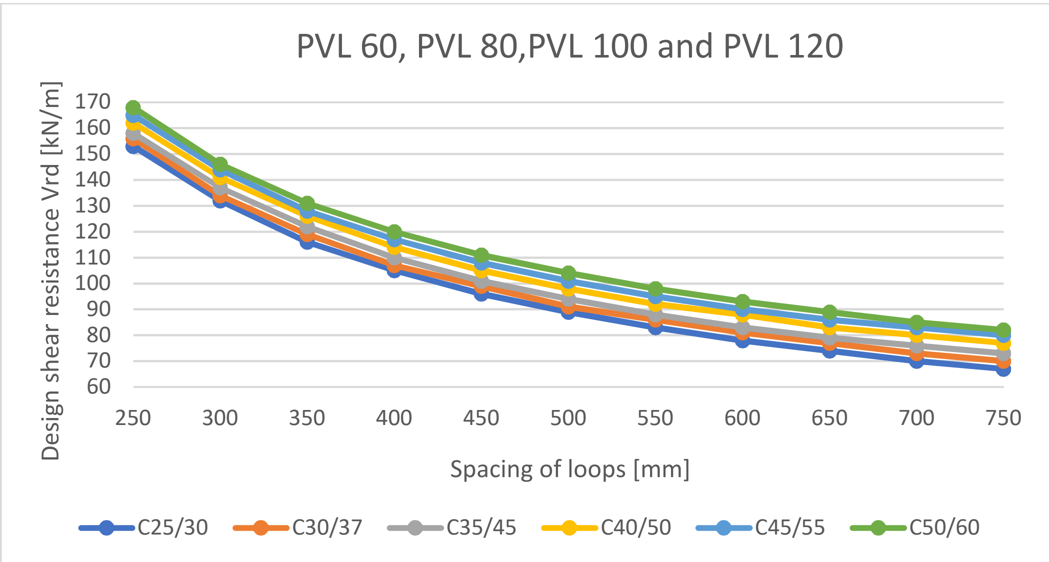


Chart 4. The dependence of the design value of the shear resistance of wire loops on spacing of loops and concrete strength

The most unfavourable combination of actions and suitable fixing devices for the joint between WALL 1 and WALL 2 is shown in Appendix 3.

8.9 Calculation procedure for the determination of the horizontal shearing forces in horizontal joint

The calculation procedure for the determination of the horizontal shearing forces is quite the same as the previous two. The main difference is that the total load F_{tot} was calculated as

$$F_{tot} = F - F_{c,\%}$$

Where F is the sum of the horizontal forces $H_{d,\%}$ in the horizontal joint. To get the horizontal forces H_d, the total moment was divided by the height of the floor.

$F_{c,\%}$ is friction force with proportional distribution

$$F_{c,\%} = N_d \cdot \mu$$

where μ is factor for shear at the interface between concrete cast in different times in accordance with EN 1992.

8.10 Choosing of the reinforced bars

To choose reinforced bars the most unfavourable combinations of actions should be defined.

For WALL 1, WALL 2 and WALL 3 the most unfavourable combination of actions was the combination with wind load as leading variable action.

The design value of the horizontal shear force F_{tot} must be smaller than the design value of the shear resistance of the reinforced bars (capacity of the dowels) F_{VRd}.

$$F_{tot} < F_{VRd}$$

The design value of the shear resistance of the reinforced bars is defined in the Sub-chapter 6.2.

The most unfavourable combination of actions for all three walls and suitable fixing devices is shown in Appendix 4.

9 Summary

During the thesis the effect of wind load was studied. Wind load works as the horizontal load. Wind load with the extra horizontal load causes the tension in vertical joints and shearing forces in vertical and horizontal joints. The wind load is transferred by the shear walls and slabs that transmit force to the shear walls.

Also, different types of fixing devices were studied that are used in Russia and in Finland. Their work, particular qualities, advantages, and disadvantages were studied.

On the example of some fixing devices was shown how to choose the right capacity of the fixing devices. For that it is needed to study the work of the fixing devices and in what cases it could be used. After that you calculate loads, define the proportional distribution of the forces and calculate the necessary loads in the joint with fixing devices in case of the most unfavourable combinations of actions, for example, vertical shearing forces in the vertical joint for wire loops.

Thus, the goals of the thesis were reached.

Figures

Charts

Tables

List of references

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Symbols in tables:

DL – dead load

LL – live load

SL1 – snow load in zone with parapet

SL2– snow load in zone without parapet

WL1– wind load (wind pressure)

WL2– wind load (wind suction)

B – length of the wall

t – thickness of the wall

H– thickness of the floor/roof slab

Table 1. Load calculations for WALL 1 part 1

	FLOOR HEIGHT, m	level mark	length of building	DL, kN/m2	LL,FLOOR kN/m2	SL1, kN/m2	SL2 kN/m2	WL1, kN/m2	WL2, kN/m2
roof	1,3	17,5	34,38	-	-	-	-	0,35	0,23
attic	2,2	16,2		5,5	2,0	2,16	2,592	0,35	0,23
FL5	2,8	14		4	2,0	-	-	0,35	0,23
FL4	2,8	11,2		4	2,0	-	-	0,35	0,23
FL3	2,8	8,4		4	2,0	-	-	0,35	0,23
FL2	2,8	5,6		4	2,0	-	-	0,35	0,23
FL1	2,8	2,8		4	2,0	-	-	0,35	0,23
FL-1	2,3	0		4	2,0	-	-	-	-
GROUND	1,2	-2,3		4	2,0	-	-	-	-
FOUND. TOP	-	-3,5		-	-	-	-	-	-

Table 2. Geometric characteristics for WALL 1

	WALL 1		FLOOR right			FLOOR right			AREA, m²
	B, m	t, m	X	Y	H	X	Y	H	
roof	-	-	-	-	-	-	-	-	-
attic	1,125	0,16	1,18	1,065	0,22	-	-	-	1,26
FL5	1,125	0,16	1,18	1,065	0,16	-	-	-	1,26
FL4	1,125	0,16	1,18	1,065	0,16	-	-	-	1,26
FL3	1,125	0,16	1,18	1,065	0,16	-	-	-	1,26
FL2	1,125	0,16	1,18	1,065	0,16	-	-	-	1,26
FL1	1,125	0,16	1,18	1,065	0,16	-	-	-	1,26
FL-1 (cellar)	1,125	0,16	1,18	1,065	0,16	-	-	-	1,26
GROUND	1,125	0,16	1,18	1,065	0,16	-	-	-	1,26
FOUND. TOP	-	-	-	-	-	-	-	-	-

Table 3. Load calculations for WALL 1 part 2

WALL 1	DL, FLOOR	DL, WALL	LL	SL
roof	-	-	-	-
attic	6,91	8,91	2,51	3,26
FL5	5,03	11,88	2,51	0,00
FL4	5,03	11,88	2,51	0,00
FL3	5,03	11,88	2,51	0,00
FL2	5,03	11,88	2,51	0,00
FL1	5,03	11,88	2,51	0,00
FL-1 (cellar)	5,03	9,63	2,51	0,00
GROUND	5,03	4,68	2,51	0,00
FOUND. TOP	-	-	-	-
total	**42,10**	**82,62**	**20,11**	**3,26**

Table 1. Load calculations for WALL 2 part 1

	FLOOR HEIGHT, m	level mark	length of building	DL, kN/m2	LL,FLOOR kN/m2	SL1, kN/m2	SL2 kN/m2	WL1, kN/m2	WL2, kN/m2
roof	1,3	17,5	34,38	-	-	-	-	0,35	0,23
attic	2,2	16,2		5,5	2,0	2,16	2,592	0,35	0,23
FL5	2,8	14		4	2,0	-	-	0,35	0,23
FL4	2,8	11,2		4	2,0	-	-	0,35	0,23
FL3	2,8	8,4		4	2,0	-	-	0,35	0,23
FL2	2,8	5,6		4	2,0	-	-	0,35	0,23
FL1	2,8	2,8		4	2,0	-	-	0,35	0,23
FL-1	2,3	0		4	2,0	-	-	-	-
GROUND	1,2	-2,3		4	2,0	-	-	-	-
FOUND. TOP	-	-3,5		-	-	-	-	-	-

Table 2. Geometric characteristics for WALL 2

	WALL 2		FLOOR right			FLOOR left			AREA, m^2
	B, m	t, m	X	Y	H	X	Y	H	
roof	-	-	-	-	-	-	-	-	-
attic	4,8	0,16	1,18	4,8	0,22	1,79	4,76	0,22	10,98
FL5	4,8	0,16	1,18	2,55	0,16	1,79	4,76	0,16	8,33
FL4	4,8	0,16	1,18	2,55	0,16	1,79	4,76	0,16	8,33
FL3	4,8	0,16	1,18	2,55	0,16	1,79	4,76	0,16	8,33
FL2	4,8	0,16	1,18	2,55	0,16	1,79	4,76	0,16	8,33
FL1	4,8	0,16	1,18	2,55	0,16	1,79	4,76	0,16	8,33
FL-1 (cellar)	4,8	0,16	1,18	3,15	0,16	1,79	4,76	0,16	9,03
GROUND	4,8	0,16	1,18	4,8	0,16	1,79	4,76	0,16	10,98
FOUND. TOP	-	-	-	-	-	-	-	-	-

Table 3. Load calculations for WALL 2 part 2

WALL 2	DL,STAIRCASE	DL, FLOOR	DL, WALL	LL	SL
roof	-	-	-	-	-
attic	-	60,39	38,02	21,96	28,46
FL5	27	33,30	50,69	16,65	0,00
FL4	27	33,30	50,69	16,65	0,00
FL3	27	33,30	50,69	16,65	0,00
FL2	27	33,30	50,69	16,65	0,00
FL1	27	33,30	50,69	16,65	0,00
FL-1 (cellar)	20,25	36,13	41,09	18,07	0,00
GROUND	-	43,92	19,97	21,96	0,00
FOUND. TOP	-	-	-	-	-
total	**155,25**	**306,95**	**352,51**	**145,24**	**28,46**

Table 1. Load calculations for WALL 3 part 1

	FLOOR HEIGHT, m	level mark	length of building	DL, kN/m2	LL,FLOOR kN/m2	SL1, kN/m2	SL2 kN/m2	WL1, kN/m2	WL2, kN/m2
roof	1,3	17,5	34,38	-	-	-	-	0,35	0,23
attic	2,2	16,2		5,5	2,0	2,16	2,592	0,35	0,23
FL5	2,8	14		4	2,0	-	-	0,35	0,23
FL4	2,8	11,2		4	2,0	-	-	0,35	0,23
FL3	2,8	8,4		4	2,0	-	-	0,35	0,23
FL2	2,8	5,6		4	2,0	-	-	0,35	0,23
FL1	2,8	2,8		4	2,0	-	-	0,35	0,23
FL-1	2,3	0		4	2,0	-	-	-	-
GROUND	1,2	-2,3		4	2,0	-	-	-	-
FOUND. TOP	-	-3,5		-	-	-	-	-	-

Table 2. Geometric characteristics for WALL 3

	WALL 3		FLOOR right			FLOOR left			AREA, m²
	B, m	t, m	X	Y	H	X	Y	H	
roof	-	-	-	-	-	-	-	-	-
attic	5,925	0,16	2,98	2,98	0,22	1,79	1,79	0,22	16,38
FL5	5,925	0,16	2,98	2,98	0,16	1,79	1,79	0,16	16,38
FL4	5,925	0,16	2,98	2,98	0,16	1,79	1,79	0,16	16,38
FL3	5,925	0,16	2,98	2,98	0,16	1,79	1,79	0,16	16,38
FL2	5,925	0,16	2,98	2,98	0,16	1,79	1,79	0,16	16,38
FL1	5,925	0,16	2,98	2,98	0,16	1,79	1,79	0,16	16,38
FL-1 (cellar)	5,925	0,16	2,98	2,98	0,16	1,79	1,79	0,16	16,38
GROUND	5,925	0,16	2,98	2,98	0,16	1,79	1,79	0,16	16,38
FOUND. TOP	-	-	-	-	-	-	-	-	-

Table 3. Load calculations for WALL 3 part 2

WALL 3	DL, FLOOR	DL, WALL	LL	SL
roof	-	-	-	-
attic	90,09	46,93	32,76	42,46
FL5	65,52	62,57	32,76	0,00
FL4	65,52	62,57	32,76	0,00
FL3	65,52	62,57	32,76	0,00
FL2	65,52	62,57	32,76	0,00
FL1	65,52	62,57	32,76	0,00
FL-1 (cellar)	65,52	50,72	32,76	0,00
GROUND	65,52	24,65	32,76	0,00
FOUND. TOP	-	-	-	-
total	**548,75**	**435,13**	**262,09**	**42,46**

Symbols in tables:

N_k - characteristic value of the vertical load

N_d - design value of the vertical load

H_{kw} – characteristic value of the horizontal load from the wind

M_{dw} – design value of the moment from the wind

H_k – extra horizontal load for the wall because of eccentricity

M_{kh} – characteristic value of the moment from the extra horizontal load

M_{dh} – design value of the moment from the extra horizontal load

z – distance between wall shoes

M_{dtot} – total moment from the wind and extra horizontal load

N_{td} – tension/compression load in the location of the wall shoe

% - proportional distribution of the forces in percentage

$N_{td,\%}$ – tension/compression load in the location of the wall shoe in percentage

N_d – half of own weight

$N_{tot,1}$ – the total load on the one side of the wall

$N_{tot,2}$ – the total load on the other side of the wall

N_{Rd} – design value of the resistance of a wall shoe and a suitable anchor bolt

Table 1. Choosing of the wall shoes for WALL 1

WALL 1	vertical load		WIND pressure		horizontal load													
	Nk	Nd	Hkw	Mdw	Hk	shearing	Mkh	Mdh	z	Mdtot	Ntd	%	Ntd,%	Nd	Ntot,1	Ntot,2	wall shoe	NRd
roof	-	-	-	-	-	-	-	-	-	-	-	-	-	-	**-**	**-**	SUMO 16H	62
attic	21,6	25,4	28,88	0,00	0,2	29,0	0,4	0,3	0,525	0,3	-0,6	0,06	0,00	12,7	**12,7**	**12,7**	SUMO 16H	62
FL5	41,0	48,6	30,08	0,00	0,3	59,5	1,3	1,2	0,525	1,2	-2,2	0,06	0,00	24,3	**24,3**	**24,3**	SUMO 16H	62
FL4	60,4	71,8	33,69	0,00	0,5	93,6	2,6	2,4	0,525	2,4	-4,5	0,06	0,00	35,9	**35,9**	**35,9**	SUMO 16H	62
FL3	79,9	95,0	33,69	0,00	0,6	128,0	4,4	4,0	0,525	4,0	-7,5	0,06	0,00	47,5	**47,5**	**47,5**	SUMO 16H	62
FL2	99,3	118,2	33,69	0,00	0,8	162,4	6,6	5,9	0,525	5,9	-11,3	0,06	-0,01	59,1	**59,1**	**59,1**	SUMO 16H	62
FL1	118,7	141,5	33,69	0,00	0,9	197,1	9,2	8,3	0,525	8,3	-15,8	0,06	-0,01	70,7	**70,7**	**70,7**	SUMO 20H	96
FL-1 (cellar)	135,9	162,1	30,68	0,00	1,1	228,8	11,7	10,6	0,525	10,6	-20,1	0,06	-0,01	81,0	**81,0**	**81,0**	SUMO 20H	96
GROUND	148,1	177,0	30,68	0,00	1,2	260,7	13,1	11,8	0,525	11,8	-22,5	0,06	-0,01	88,5	**88,5**	**88,5**	SUMO 20H	96
FOUND. TOP	-	-	-	-	-	-	-	-	-	-	-	-	-	-	**-**	**-**	SUMO 20H	96

Table 2. Choosing of the wall shoes for WALL 2

WALL 2	vertical load		WIND pressure		horizontal load													
	Nk	Nd	Hkw	Mdw	Hk	hearing	Mkh	Mdh	z	Mdtot	Ntd	%	Ntd,%	Nd	Ntot,1	Ntot,2	wall shoe	NRd
roof	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	SUMO 30H	220
attic	148,8	176,0	28,88	0,00	1,2	30,1	2,6	2,3	4,38	2,3	-0,5	4,33	-0,02	88,0	**88,0**	**88,0**	SUMO 30H	220
FL5	276,5	328,6	30,08	0,00	2,2	62,3	8,7	7,8	4,38	7,8	-1,8	4,33	-0,08	164,3	**164,2**	**164,4**	SUMO 30H	220
FL4	404,1	481,2	33,69	0,00	3,2	99,2	17,7	15,9	4,38	15,9	-3,6	4,33	-0,16	240,6	**240,5**	**240,8**	SUMO 36P	436
FL3	531,7	633,8	33,69	0,00	4,2	137,1	29,5	26,6	4,38	26,6	-6,1	4,33	-0,26	316,9	**316,7**	**317,2**	SUMO 36P	436
FL2	659,4	786,4	33,69	0,00	5,2	176,1	44,2	39,8	4,38	39,8	-9,1	4,33	-0,39	393,2	**392,8**	**393,6**	SUMO 36P	436
FL1	787,0	939,1	33,69	0,00	6,3	216,0	61,7	55,6	4,38	55,6	-12,7	4,33	-0,55	469,5	**469,0**	**470,1**	SUMO 45P	697
FL-1 (cellar)	902,6	1078,3	30,68	0,00	7,2	253,9	78,3	70,4	4,38	70,4	-16,1	4,33	-0,70	539,1	**538,4**	**539,8**	SUMO 45P	697
GROUND	988,4	1184,7	30,68	0,00	7,9	292,5	87,7	79,0	4,38	79,0	-18,0	4,33	-0,78	592,3	**591,6**	**593,1**	SUMO 45P	697
FOUND. TOP	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	SUMO 45P	697

Table 3. Choosing of the wall shoes for WALL 3

WALL 3	vertical load		WIND pressure		horizontal load													
	Nk	Nd	Hkw	Mdw	Hk	hearing	Mkh	Mdh	z	Mdtot	Ntd	%	Ntd,%	Nd	Ntot,1	Ntot,2	wall shoe	NRd
roof	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	SUMO 39H	383
attic	212,2	251,3	28,88	0,00	1,7	30,6	3,7	3,3	5,325	3,3	-0,6	8,14	-0,05	125,6	**125,6**	**125,7**	SUMO 39H	383
FL5	373,1	447,7	30,08	0,00	3,0	63,6	12,0	10,8	5,325	10,8	-2,0	8,14	-0,17	223,9	**223,7**	**224,0**	SUMO 39H	383
FL4	533,9	644,2	33,69	0,00	4,3	101,6	24,1	21,7	5,325	21,7	-4,1	8,14	-0,33	322,1	**321,8**	**322,4**	SUMO 39H	383
FL3	694,8	840,6	33,69	0,00	5,6	140,9	39,8	35,8	5,325	35,8	-6,7	8,14	-0,55	420,3	**419,8**	**420,9**	SUMO 39P	521
FL2	855,6	1037,1	33,69	0,00	6,9	181,5	59,1	53,2	5,325	53,2	-10,0	8,14	-0,81	518,5	**517,7**	**519,4**	SUMO 39P	521
FL1	1016,5	1233,5	33,69	0,00	8,2	223,4	82,1	73,9	5,325	73,9	-13,9	8,14	-1,13	616,8	**615,6**	**617,9**	SUMO 52P	938
FL-1 (cellar)	1165,5	1416,3	30,68	0,00	9,4	263,6	103,9	93,5	5,325	93,5	-17,6	8,14	-1,43	708,2	**706,7**	**709,6**	SUMO 52P	938
GROUND	1288,4	1569,2	30,68	0,00	10,5	304,7	116,4	104,8	5,325	104,8	-19,7	8,14	-1,60	784,6	**783,0**	**786,2**	SUMO 52P	938
FOUND. TOP	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	SUMO 52P	938

Symbols in tables:

z – the distance between the edge of the wall and joint location

M_{dtot} – total moment for the joint between WALL 1 and WALL 2

N_{td} – tension/compression load

% – proportional distribution of the forces in percentage

$N_{td,\%}$ – tension/compression load in percentage

N_d – half of own weight

V_{ed} –design value of the shear force (the total load)

V_{Rd} – design value of the shear resistance of wire loops for the whole joint

V_{Rd}, kN/m – design value of the shear resistance of wire loops

Table 1. Choosing of the wire loops for the joint between WALL 1 and WALL2

JOINT BETWEEN WAL 1 AND WALL 2												
z	**Mdtot**	**Ntd**	**%**	**Ntd,%**	**Nd**	**Ved**	**VRd,kN**	**wire loops**	**VRd, kN/m**	**Spacing of loops**	**Concrete strength**	**FLOOR HEIGHT, m**
-	-	-	-	-	-	**-**	210,6	PVL80	162	350	C40/50	1,3
1,065	2,6	-2,5	7,57	-0,19	100,2	**100,0**	356,4	PVL80	162	350	C40/50	2,2
1,065	8,9	-8,4	7,57	-0,64	187,6	**186,9**	453,6	PVL80	162	350	C40/50	2,8
1,065	18,2	-17,1	7,57	-1,29	274,9	**273,6**	453,6	PVL80	162	350	C40/50	2,8
1,065	30,4	-28,5	7,57	-2,16	362,2	**360,0**	453,6	PVL80	162	350	C40/50	2,8
1,065	45,5	-42,7	7,57	-3,23	449,5	**446,3**	453,6	PVL80	162	350	C40/50	2,8
1,065	63,5	-59,6	7,57	-4,51	536,9	**532,3**	585,2	PVL140	209	350	C40/50	2,8

Symbols in tables:

z – height of the floor

$M_{d\,tot}$ – total moment from the wind and extra horizontal load

H_d – horizontal forces in the joint

% – proportional distribution of the forces in percentage

$H_{d,\%}$ – horizontal forces in the joint in percentage

F – friction force

$F_{c,\%}$ – friction force in percentage

F_{tot} – total horizontal load in the joint

$F_{vRd,tot}$ – total shear capacity of the dowels for the wall

F_{vRd} – shear capacity of the one dowel

N – quantity of the reinforced bars

Table 1. Choosing of the reinforcement bars for WALL 1

WALL 1												
z	**Mdtot**	**Hd**	**%**	**Hd,%**	**F**	**Fc,%**	**F tot**	FvRd, tot	**FvRd, kN**	**N**	**spacing between dowels, m**	**length of wall,m**
1,3	-	-	-	-	-	-	-	19	19	-	-	-
2,2	95,6	43,4	0,06	0,03	0,03	0,01	**0,02**	19	19	1	0,6	1,125
2,8	222,7	79,5	0,06	0,05	0,07	0,01	**0,06**	19	19	1	0,6	1,125
2,8	365,3	130,5	0,06	0,08	0,13	0,02	**0,11**	19	19	1	0,6	1,125
2,8	508,3	181,5	0,06	0,11	0,19	0,02	**0,17**	19	19	1	0,6	1,125
2,8	651,7	232,8	0,06	0,14	0,25	0,03	**0,22**	19	19	1	0,6	1,125
2,8	795,5	284,1	0,06	0,17	0,31	0,03	**0,28**	19	19	1	0,6	1,125
2,3	903,5	392,8	0,06	0,24	0,41	0,04	**0,37**	19	19	1	0,6	1,125
1,2	960,0	800,0	0,06	0,48	0,72	0,04	**0,6757**	19	19	1	0,6	1,125
-	-	-	-	-	-	-	-	19	19	1	-	-

Table 2. Choosing of the reinforcement bars for WALL 2

WALL 2												
z	**Mdtot**	**Hd**	**%**	**Hd,%**	**F**	**Fc,%**	**F tot**	FvRd, tot	**FvRd, kN**	**N**	**spacing between dowels, m**	**length of wall,m**
-	-	-	-	-	-	-	-	19	19	-	-	-
2,2	97,2	44,2	4,33	1,91	1,91	2,42	**-0,50**	19	19	1	2,4	4,8
2,8	228,4	81,6	4,33	3,53	5,45	4,96	**0,49**	19	19	1	2,4	4,8
2,8	377,4	134,8	4,33	5,84	9,37	7,50	**1,87**	19	19	1	2,4	4,8
2,8	528,9	188,9	4,33	8,18	14,01	10,04	**3,97**	19	19	1	2,4	4,8
2,8	682,8	243,9	4,33	10,56	18,74	12,58	**6,16**	19	19	1	2,4	4,8
2,8	839,3	299,8	4,33	12,98	23,54	15,12	**8,42**	19	19	1	2,4	4,8
2,3	959,4	417,1	4,33	18,06	31,04	17,43	**13,61**	19	19	1	2,4	4,8
1,2	1022,8	852,3	4,33	36,91	54,97	19,18	**35,79**	38	19	2	1,6	4,8
-	-	-	-	-	-	-	-	38	19	-	-	-

Table 2. Choosing of the reinforcement bars for WALL 3

WALL 3												
z	**Mdtot**	**Hd**	**%**	**Hd,%**	**F**	**Fc,%**	**F tot**	FvRd, tot	**FvRd, kN**	**N**	**spacing between dowels, m**	**length of wall,m**
-	-	-	-	-	-	-	-	19	19	-	-	-
2,2	98,0	44,5	8,14	3,62	3,62	6,42	**-2,79**	19	19	1	3,0	5,925
2,8	230,9	82,5	8,14	6,71	10,34	12,51	**-2,17**	19	19	1	3,0	5,925
2,8	382,2	136,5	8,14	11,11	17,82	18,60	**-0,78**	19	19	1	3,0	5,925
2,8	536,8	191,7	8,14	15,60	26,72	24,69	**2,02**	19	19	1	3,0	5,925
2,8	694,5	248,0	8,14	20,19	35,79	30,79	**5,01**	19	19	1	3,0	5,925
2,8	855,5	305,5	8,14	24,87	45,06	36,88	**8,18**	19	19	1	3,0	5,925
2,3	979,8	426,0	8,14	34,67	59,54	42,54	**17,01**	19	19	1	3,0	5,925
1,2	1045,7	871,4	8,14	70,93	105,61	47,24	**58,37**	76	19	4	1,2	5,925
-	-	-	-	-	-	-	-	76	19	4	-	-