EVG

3D Construction System

Construction System Manual

&

Structural Engineering Handbook

General Notes

The 3D Construction System is a rather new, cost-efficient construction system that has many uses and bases on industrially prefabricated 3D panels. The 3D panels consist of an EPS core with a thickness ranging from 40 to 100 mm sandwiched between two plane-parallel welded wire mesh sheets (cover meshes) and inclined diagonal wires in between that go through the EPS core and that are welded to the cover mesh's line wires.

This results in a light-weight, three-dimensional truss system with a high inherent stiffness. The clear spacing between the EPS core and the cover mesh lies between 13 and 19 mm. 3D panels are high-precision elements produced by fully-automatic EVG Wire Mesh Welders, type 3D/48. The standard width of 3D panels is 1.20 m (1.00 m) while the element's length is variable (in steps of 10 cm) and depends on the corresponding field of application. 3D panels are delivered as positioned elements to the site where they are connected easily to wall and slab structures. Splice meshes serve to seal the joints between the 3D panels. This brings about a continuous mesh structure (reinforcement) over the entire construction.

Afterwards an approx. 40 to 60 mm thick cement-mortar layer (concrete layer) is applied either manually or, preferably, mechanically onto both sides of this “dry” structure. During this process the EPS core of the 3D panels serves as a shuttering and plaster base. As soon as the concrete has hardened the 3D construction achieves its structural and functional strength. The result is a composite sandwich system in which the two reinforced concrete shells on the outside are connected by the wire diagonals arranged in a truss-like pattern with sufficient shear strength.

In order to be able to prove the structural effectiveness of 3D constructions, renowned research centers in Austria and abroad carried out a great number of compression, bending and shear tests. All these tests showed that the principle theories and the resulting calculation methods of reinforced concrete construction systems apply without restrictions also to 3D constructions. However, this implies that, by taking into consideration minor characteristic features, all national and international regulations on standards for reinforced concrete construction systems are valid also for 3D constructions.

The physical characteristics of 3D construction such as thermal resistance, sound reduction index and fire resistance were determined in large-scale laboratory tests or extensive theoretical calculations according to the above-mentioned structural stability check calculations, as well.

In summary, the 3D construction system is an economical alternative to traditional construction systems and meets all structural and physical requirements relating to building physics. Besides, it is possible to reduce the weight of the buildings by as much as 40 %. The Manual on hand aims to introduce the engineer to the state-of-the-art knowledge of 3D construction design and to provide him with efficient and easy design aids. However, the sufficient knowledge of the traditional design rules regarding reinforced concrete constructions are a prerequisite. Nevertheless this manual gives a detailed survey on the specific 3D characteristics and provides for design tables basing on various standards (e.g. ÖNORM, DIN, ACI). This Manual does not claim to be complete, and EVG reserves the right to change parts of it without prior notification if the latest test results or developments require to do so.

Dr. Klaus Matz

1. EVG 3D System

1.1. Construction System

3D CROSS SECTION

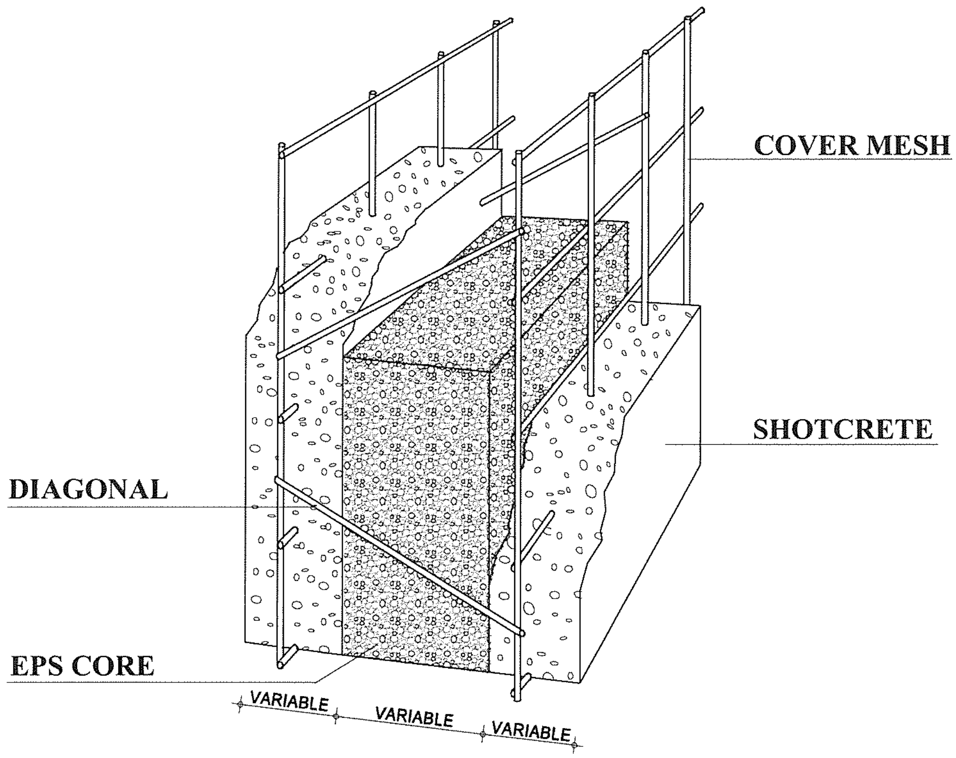


figure 1.1.a Section of a 3D wall

Components of 3D elements can generally be dimensioned as other reinforced concrete components. All theoretical reflections and standards are applicable also for 3D components if it is taken into consideration that the effective concrete cross section is reduced by the EPS core and that shear forces can only be transferred via the diagonals. The following chapters shall deal with the dimensioning of 3D components according to different standards such as DIN (Germany), ACI (USA) and ÖNORM (Austria), shown in examples, always emphasizing on the particularities of the 3D system.

3D components are thin-walled reinforced concrete sandwich elements which can mainly transfer compression- and shear forces in the plane of wall. The flexural resistance perpenticular to the plane of wall is limited. Therefore buildings erected as 3D constructions are composed in a "box-like" way where 3D elements are connected with each other. Thereby the joints between slab and wall or wall and wall in general do not transfer any or only minor bending moments. However, a flexurally rigid connection of the individual slabs to form a continuous slab system is usually common.

Forces acting horizontally on 3D buildings such as wind- or earth quake forces are absorbed most effectively by the 3D shear walls. The respective "box-like" infilling with 3D panel walls in X- and Y direction of 3D buildings has to be provided. For dimensioning, 3D slabs and 3D walls can be considered independently from each other. A frame-like design of 3D buildings is possible to a limited extent only and is not recommended.

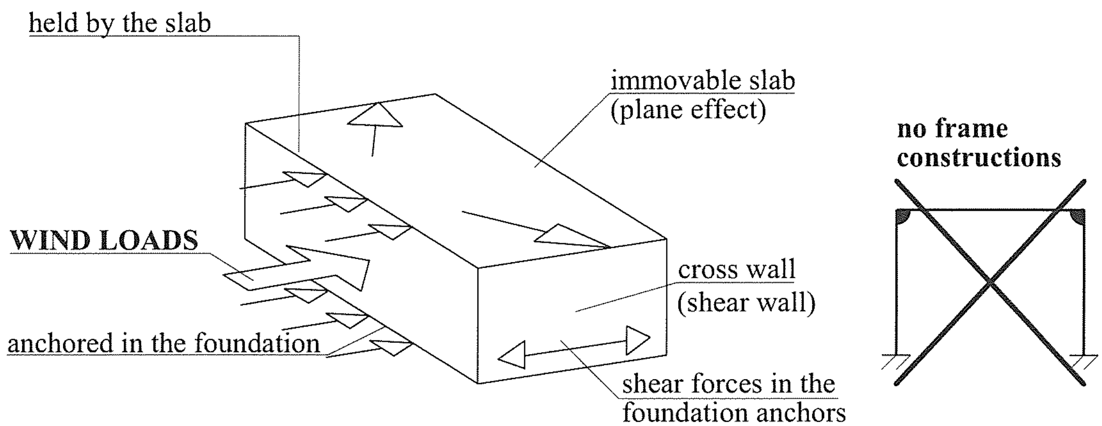


figure 1.1.b Transfer of horizontal loads

3D slab systems are mainly designed as one-way slabs. In transverse direction, a 3D panel can absorb very low shear forces only and hence only very small moments. These one-way 3D slabs can be calculated in case of a continuous effect as continuous girders with constant cross section.

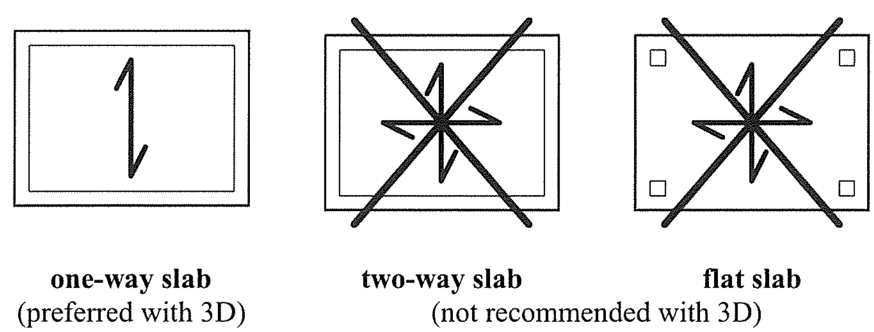


figure 1.1.c Slab systems

1.2. 3D Components

A 3D component consists of an EVG 3D panel and of concrete layers of at least 40 mm thickness applied on both sides. For load-bearing 3D components the minimum thickness of concrete is 50 mm. Only for the walls of one-storey buildings 40 mm of concrete are sufficient. The 3D panels are composed of an EPS core with a thickness of 40 - 100 mm, of two parallel reinforcing mesh sheets (cover mesh) and of diagonal truss wires inclined in longitudinal direction. The diagonal truss wires puncture the EPS core and are galvanized due to the possible risk of corrosion. The cover mesh need not be galvanized if the concrete layer is sufficiently thick (see figure 1.1.a).

The concrete that is applied on both sides, has got the following functions:

- transfer of compression forces
- protection against corrosion of the reinforcement

For a useful protection against corrosion a min. concrete thickness of 40 mm is required inside a building. For outside parts, this value rises to 50 mm. As a rule 3D slabs are covered with a concrete topping of approx. 50 - 60 mm.

The concrete on the walls and bottom side of slabs is generally applied as shotcrete. However there is also the possibility of manual application especially for the thin second layer. The top side of the slab is usually covered with pumped concrete. In all cases the cement content must not be less than 250 - 300 kg/m³.

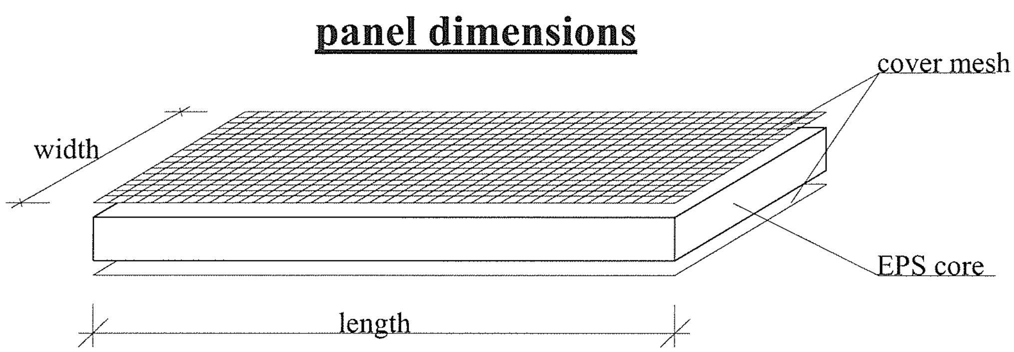


figure 1.2.a The EVG 3D panel

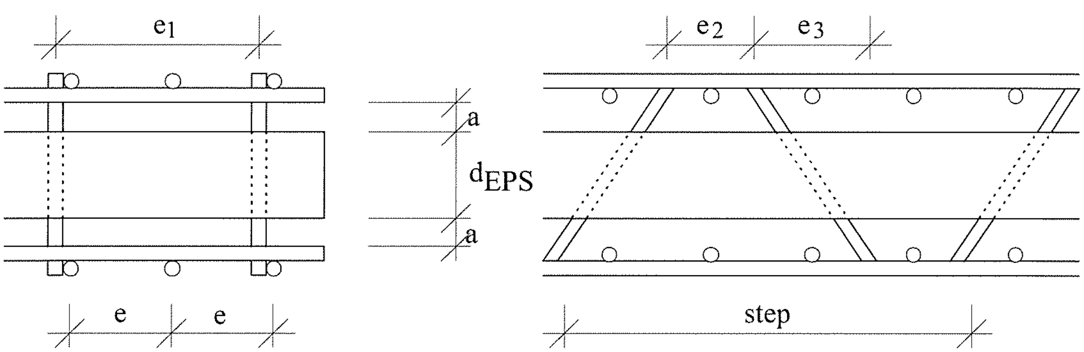


figure 1.2.b Arrangement of the diagonals and the cover mesh

Standard dimensions (see figure 1.2.b):

Panel size:

Length .... min. 2.00 m. Furthermore every length in steps of 10 cm is possible. The max. length is usually 6.00 m. Theoretically longer panels can be produced as well.

Width...... 1.20 m (1.00 m)

EPS......... expanded polystyrene according to ÖNORM B6050 with a density of approx. 15 kg/m³. The thickness varies between 40 and 100 mm in steps of 10 mm.

Cover mesh :

Diameter.......................... 3.0 mm; BST500 acc. to ÖNORM B4200, part 7.

Mesh size (e)................... 50 × 50 mm

Distance mesh - EPS (a) .. 13, 16 or 19 mm; the most common distance is 13 mm.

Diagonals :

Diameter............ 3.8 mm; galvanized steel of group BST500. A max. diameter of 4.5 mm is possible (also stainless steel is used).

Cross distance .... 100 or 200 mm (= e_1)

Step .... 100 mm or 200 mm; resulting in 67 - 200 diagonals per m²

Inclination .... The inclination of the diagonals depends on the distances e_2 and e_3 in figure 1.2.b. For reasons of production, the value for e_2 must not drop below a certain minimum. Currently panels with 2 different arrangements of diagonals are produced.

quantity [pcs./m²]	step [mm]	e_3 [mm]
100	200	60
200	100	40

table 1.2.a Arrangement of diagonals

Hence the inclination angle results from

$$\alpha = \arctan\left(\frac{d_{EPS} + 2 \cdot a}{e_3}\right)$$

Since the value of e_3 is uncertain – it may vary by several millimeters – in the structural calculations a value of 20 mm can be assumed for "a" independently from the distance between mesh and EPS.

1.3. Erection of Wall Panels

The walls of a 3D building start from the upper edge of the foundation, preferably a load bearing slab, or foundation strips as well. Starter bars with a ø of 10 mm set at a distance of approx. 50 cm are necessary on one side of the wall (for outside walls mostly inside) in order to put up the wall. On account of the exactness it is recommended to drill the holes for the starter bars later. Then fill up the holes with cement slurry. These starter bars mainly serve for an easy erection of the panel walls and do not serve to carry horizontal forces or moments. For special structural needs only (e. g. wind loads on cantilever walls) these rebars have to be inserted on both sides at smaller distances according to the structural requirements. Then it is recommended to fill up the drilling holes with chemical binding agents (epoxy resin). It is only in case of considerable loads (e.g. earth quake loads) that this connecting reinforcement has to be placed with the foundation slab already.

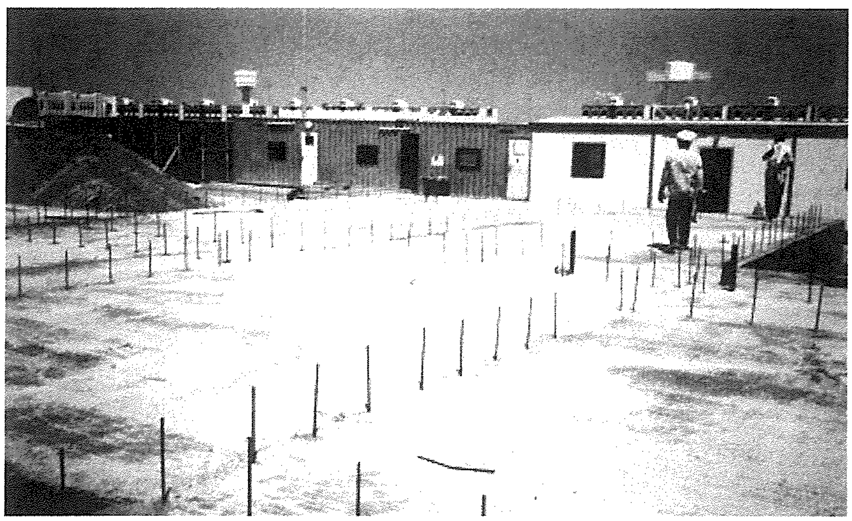


photo 1: Foundation slab with starter-bars

Foundations without connecting reinforcement are practicable as well. This may be, for instance, U-channels of the width of the EPS core that are fixed on the foundation slab. For that end, U-channels of at least 30 mm height and a steel thickness of 0.6 - 0.8 mm are sufficient. In this case, the use of small pieces of splice mesh (e.g. 30 × 30 cm) in the wall corners is recommended. This splice mesh is L-shaped and is clamped below the U-channel. Then the wall panels can be tied to them.

However, the foundation slab has to be covered with a waterproof protective layer before setting up the panels. This is done most effectively with a bituminous paint film.

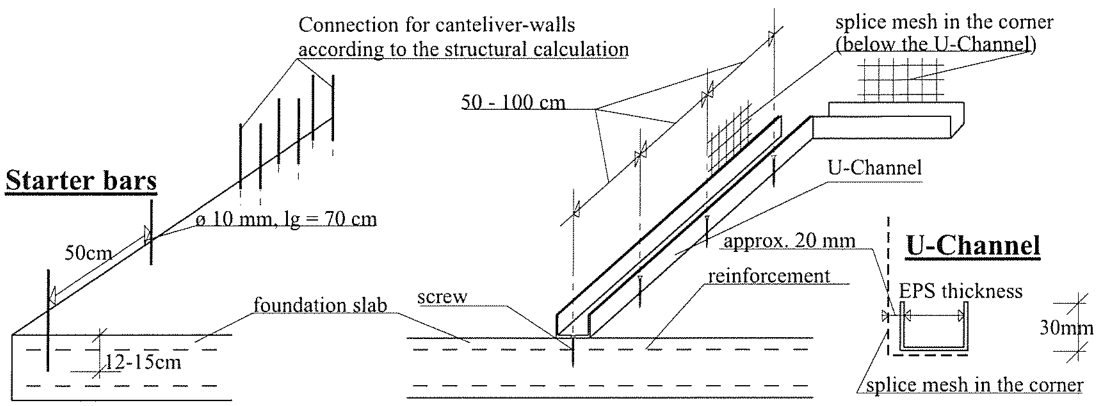


figure 1.3.a Connection to the foundation

Erecting the walls starts always in a corner. This is necessary to give the construction enough rigidity already from the beginning because only then it is possible to arrange the wall panels vertically and to permanently fix them to the bars or to the splice mesh. Fixing is done by tying the panels to the bars by means of tie wire.

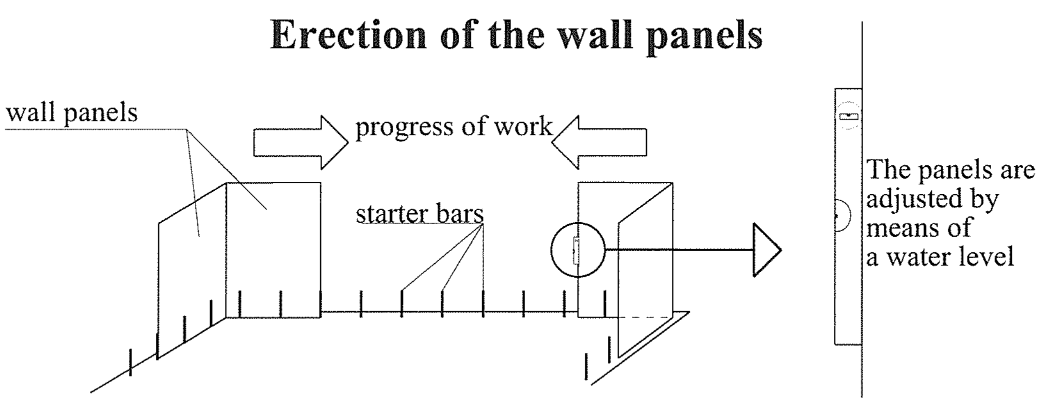


figure 1.3.b Erecting the wall panels

1.3.1. Cutting Method of Wall Panels

Figure 1.3.1.a shows a solution with relatively low waste. Thereby the windows of usual dimensions for living rooms are taken into consideration from the beginning. The panels remaining from a window opening are used in the following wall segment. This results generally in 2 filler panels per segment. For parapets, the same procedure is used in principle. In most of the cases, a wall panel (2.80 - 3.00 m) can be cut into 3 parapet pieces (80 - 90 cm) with low waste. Only small windows (bath room, A/C) are cut out after erection of the wall panel. Up to a panel height of 1.20 m the lintels above the openings are made of panels, where the truss configuration is horizontally (generally the truss configuration is vertically).

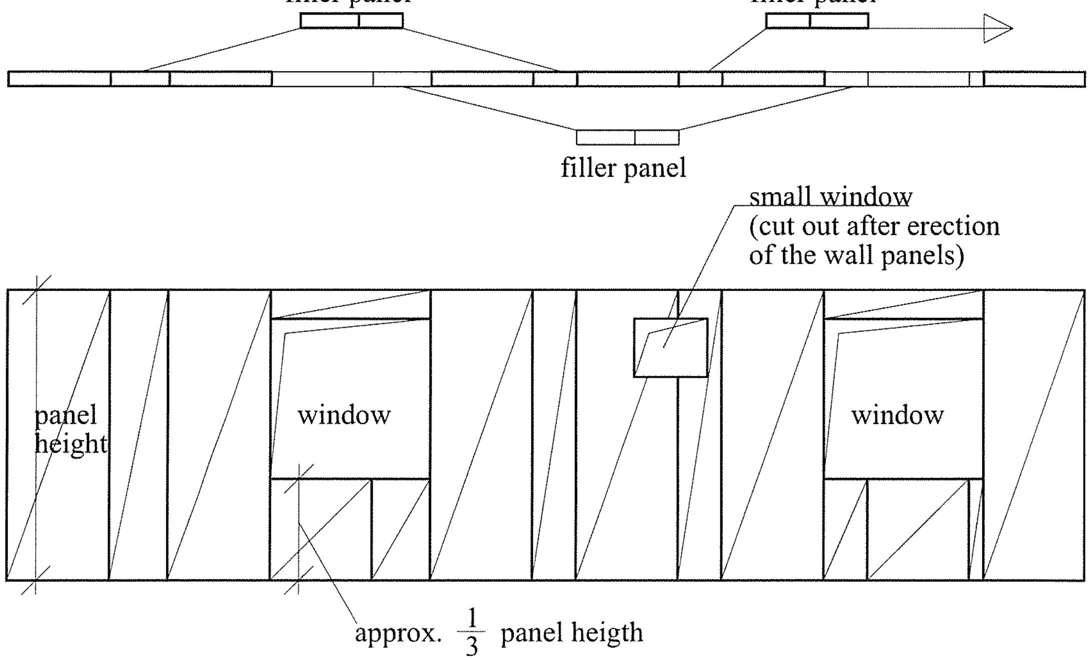


figure 1.3.1.a Typical layout of wall panels

1.3.2. Door and Window Frames

The surface of a finished 3D wall corresponds to the surface of a usual concrete wall. The windows and doors can be set in the same way as in a brick- or concrete building. If the reveal is not made of concrete, a frame of the same width as the wall (e.g. of wood) will have to be used that is fixed by a compression-proof foam.

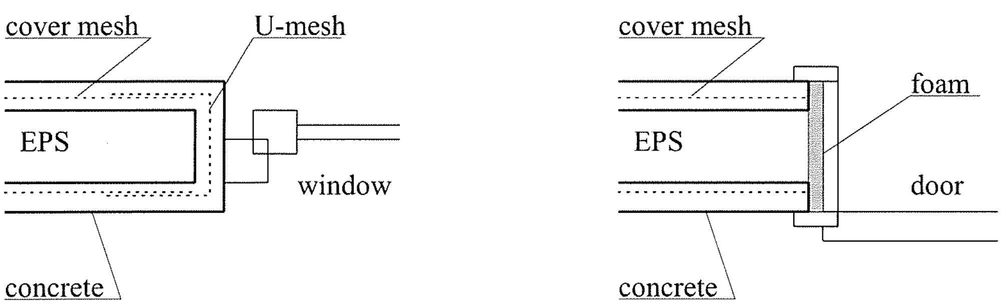


figure 1.3.2.a Openings with and without concrete reveal

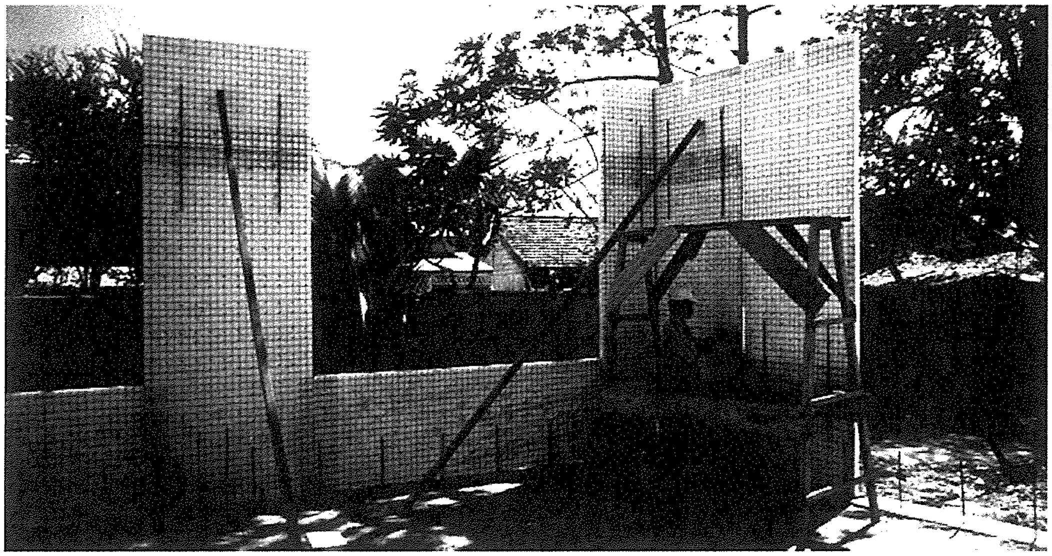


photo 2: Walls with bracings

The pipes for water and electric installations will be laid between EPS and cover mesh after the erection of the panels. Before laying thicker pipes possibly a groove in the EPS core has to be burned out by a torch.

1.4. Panel Splices

After erecting the walls, all splices are overlapped by means of splice mesh. The splice mesh has the same dimensions as the cover mesh of the panel (ø 3 mm, e = 50 mm). The idea behind is to create a continuous mesh reinforcement (cover mesh). This is done most effectively by means of a so-called hogring gun. For this purpose the following areas have to be covered by means of the splice mesh:

- straight panel joints — straight splice mesh, b = 30 or 45 cm
- external corners — L-shaped bent splice mesh, b = 15 + 30 = 45 cm
- internal corners — L-shaped bent splice mesh, b = 2 × 15 = 30 cm
- window and door reveals — U-shaped bent splice mesh, b = 45 cm
- corners of wall openings — straight splice mesh, b = 30 cm, fixed under 45°

As a rule the following quantity of splice mesh will be required :

- splice mesh with 30 cm — 45 - 65 % area of panel surface
- splice mesh with 45 cm — 15 - 30 % area of panel surface

The 45 cm wide splice mesh is mainly required for U-shaped stirrup cages. Therefore the quantity largely depends on the size and number of wall openings. Furthermore this quantity is considerably influenced by the length of free edges to be bordered with U-mesh (e.g.

cantilever slabs or free-standing walls). Components in which a wide overlapping joint is required for structural reasons, are rather the exception and increase the total requirement only slightly.

In order to guarantee a continuous reinforcement, the overlapping joint has to be designed so as to assure that the tensile forces are transferred by at least 2 welding joints. In case of components subject to flexure, the overlapping length has to be doubled for reasons of safety. In the transverse direction of the slab 30 cm are generally sufficient. It is only below single- or line loads that bending moments occur in transverse direction which require larger overlapping length.

connection area	overlapping length
slab in transverse direction, walls	2 welding joints
components subject to flexure	4 welding joints

Table 1.4.a Overlapping lengths of splice mesh

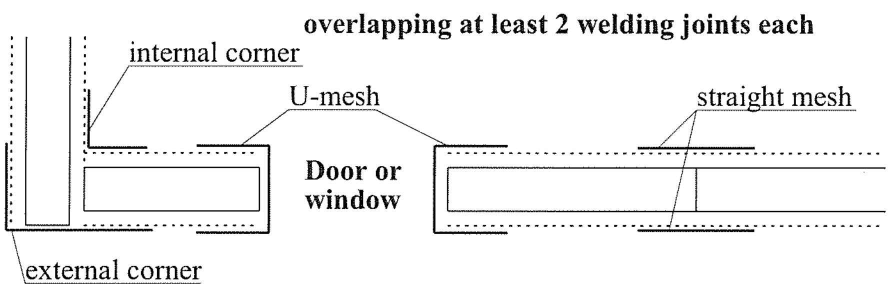


figure 1.4.a Arrangement of splice mesh

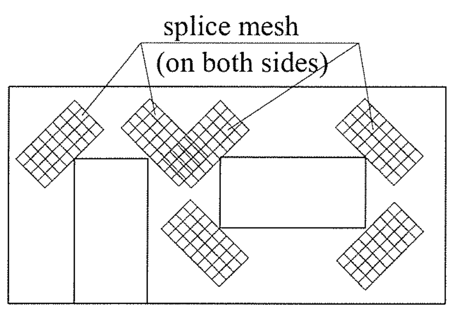


figure 1.4.b Arrangement of splice mesh. To avoid 45° cracks in corners of doors and windows, attach the splice mesh at both sides.

photo 3: Fixing of splice mesh with a hogring gun

1.5. Alignment of Walls

1.5.1. Construction without 3D Slab

The horizontal alignment can be achieved by means of wooden beams (e.g. 10 × 10 cm) or steel beams arranged in a horizontal position. These are tied to the panel with tie wire at a height of approx. 2/3 of the total height (max. 2.0 - 2.5 m) and removed as soon as the first layer of concrete becomes hardened (approx. 2 - 3 days after applying the concrete). Vertical arrangement is done with inclined bracings, e. g. shuttering props whose length can be adjusted independently. These bracings are either anchored in the floor by means of tension-proof connections or they will be attached to both sides of the wall. They will be removed together with the horizontal stiffenings after applying the first layer of concrete.

An other possibility of stiffening the panel wall in transverse direction consists in the use of U-channels which are clamped onto the EPS on the topside of the panel and fastened with each other with rivets. For that end the same U-channels can be used as for anchoring in the foundation (see figure 1.3.a). In addition such a stiffening on topside could be made at the building site from sawn timber.

These topside constructions are subsequently tied to each other at some points. Especially in long walls an inclined bracing as shown in figure 1.5.1.a is required as well.

wall bracing

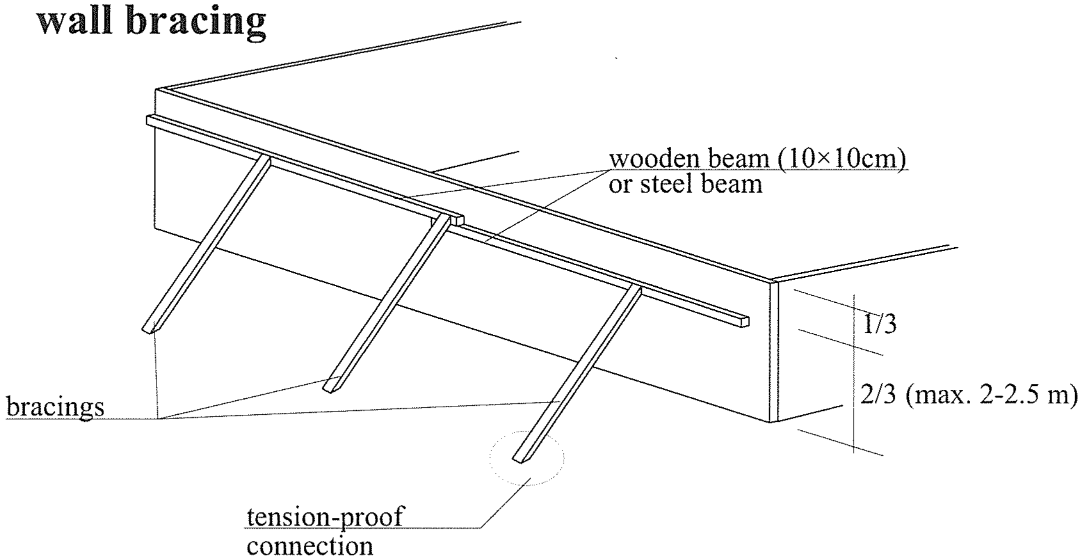
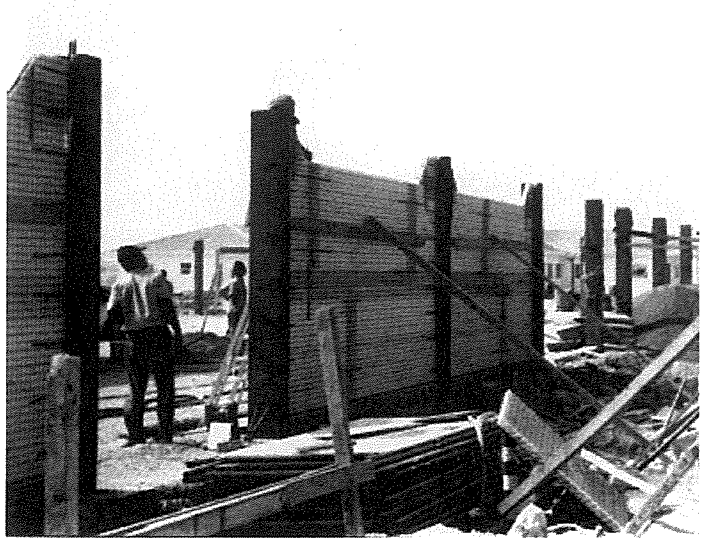


figure 1.5.1.a Alignment of a 3D-Wall

photo 4: Braced wall



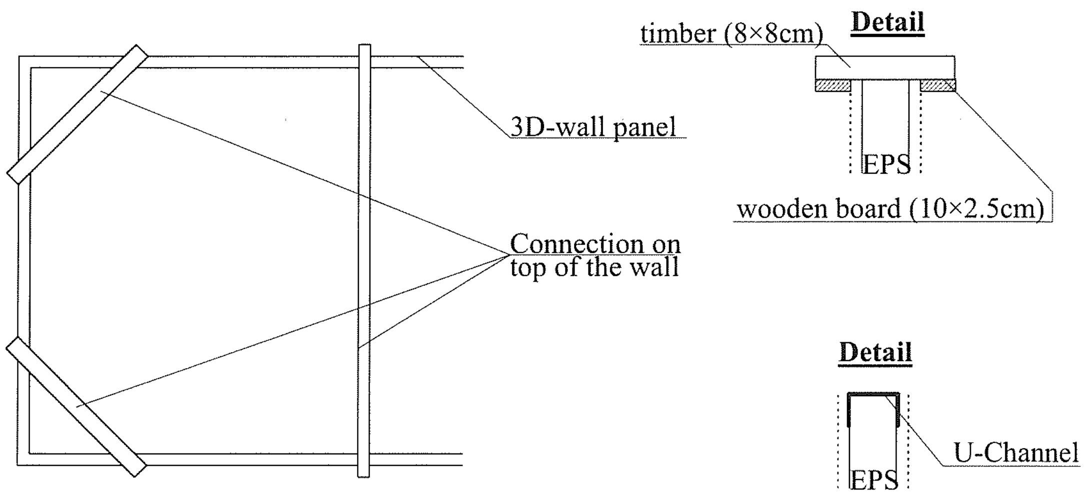


figure 1.5.1.b Stiffening and alignment of a 3D wall with a topside construction

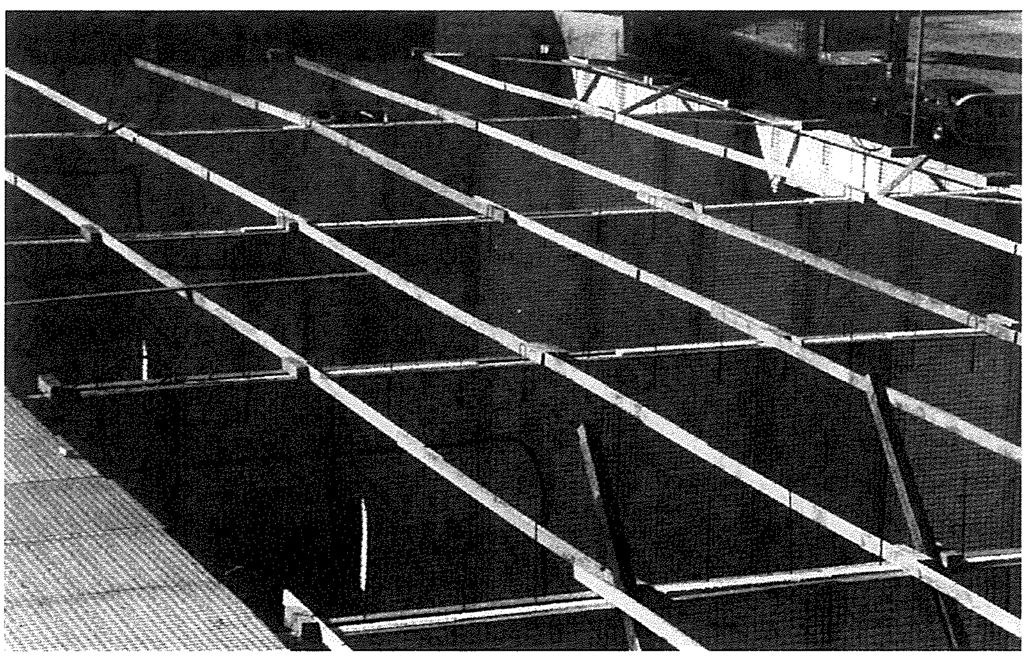


photo 5 Topside construction

1.5.2. Stiffening and Alignment by means of a 3D Slab

If there is a slab made of 3D panels, the wall can be mounted to it conveniently and aligned. To this end the L-shaped rebars (ø 10 mm, lg = 2 × 50 cm) are fixed first to the walls at a distance of 50 cm. Then the walls are aligned by means of a cord and mounted to the slab with these rebars. L-shaped bent splice meshes (b = 2 × 15 cm) are used instead of the rebars for non-load bearing inner walls above which the panels are located.

fixing the wall panels

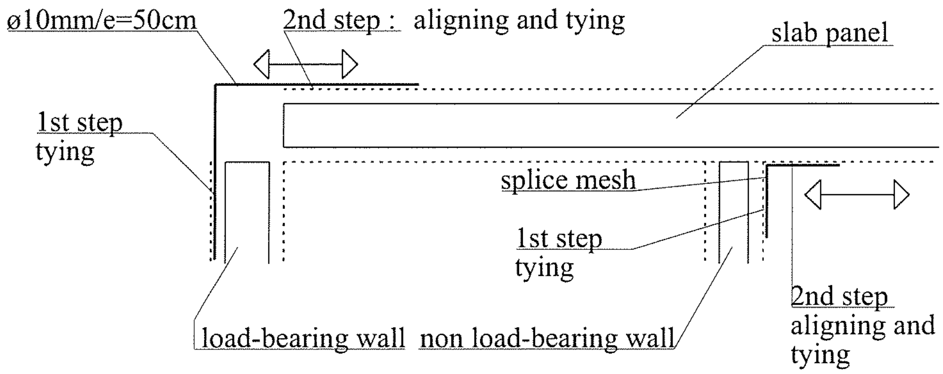


figure 1.5.2.a Stiffening and alignment of a 3D-wall by means of a 3D-slab

photo 6: Aligning the wall with a cord

1.6. Erection of the Slab

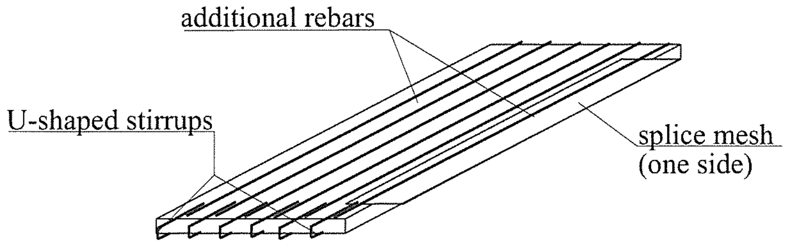
Props whose height is infinitely variable are preferably used to shore the slab. These props have to be put up with girders that run at right angles to the slab panels. When choosing the distance between the support rows it is necessary to take into account the flexural resistance of the panels (see below).

To facilitate work, the panels are reinforced with the necessary items already on the floor.

- additional reinforcement (bars) at the bottom
- splice mesh at the bottom (on one side)
- U-shaped stirrups at the support

slab panel

figure 1.6.a Preparation of the slab panels before placing



Afterwards the panels are lifted manually onto the slab and tied with tie wire. To avoid cracks on the top side of the slabs above non-load bearing walls, the panels are connected with splice mesh on the top side as well. In addition, the required reinforcement at the support (see figure 1.6.b) is attached to the panel above the load bearing inner walls.

After laying the panels, rebars on the top side of the slab have to be mounted above the load-bearing inner walls, to provide for a continuous behaviour of the slab.

At least 2 ø 8 mm rebars are placed at all outer edges (ring beam) as the last reinforcement element after finishing the panel slab.

The 3D wall panel without concrete can practically not resist the vertical forces. Therefore all mounting loads from the slab panel have to be taken by temporarily supports. The slab must not be laid directly on the panel walls. The moment capacity of a standard panel (100 mm EPS, cover mesh ø 3 mm / 50 mm, 200 diagonals ø 3.8 mm per m²) has been determined by tests and results from

$$M_{ADM} = 3.00 \text{ kNm/m}$$

Thereof the following admissible bending moments (buckling) for other panel types under assembling conditions can be calculated:

Moments	100 diag./m²	200 diag./m²
EPS - 50	0.50 kNm/m	1.80 kNm/m
EPS - 100	0.85 kNm/m	3.00 kNm/m

table 1.6.a Admissible buckling moments of 3D panels

Considering a top concrete layer of 50 mm and a live load under assembling conditions of 1.50 kN/m², a theoretic shoring span of approx. 3 m results for a standard slab panel. In fact the distance between struts should, however, been chosen in-between 1.60 and 1.80 m owing to the deflection. Figure 1.6.b shows an optimal arrangement. Thereby the edge support is set up at a distance of 50 - 60 cm from the wall. This allows to make supports by means of tripods and facilitates considerably work for the first concrete layer on the wall. Without a lateral overhang of the slab panel or if this overhang cannot be tied to a 3D wall, it is even recommended to reduce the shoring span to 1.50 m.

As soon as the top concrete layer on the slab and the concrete on the walls are sufficiently hardened to carry slight mounting loads (after 1 - 2 days), these edge supports can be removed and the shoring span can be increased to up to 2.25 m. However make sure that - before removing the edge support - in the area of the reinforcement at the support at least a 50 cm wide strip of the first concrete layer on the bottom side of the slab has been applied to assure the bond between the panel and the reinforcement at the support from the beginning. It is only in this case that the mounting loads can be transferred to the walls correctly.

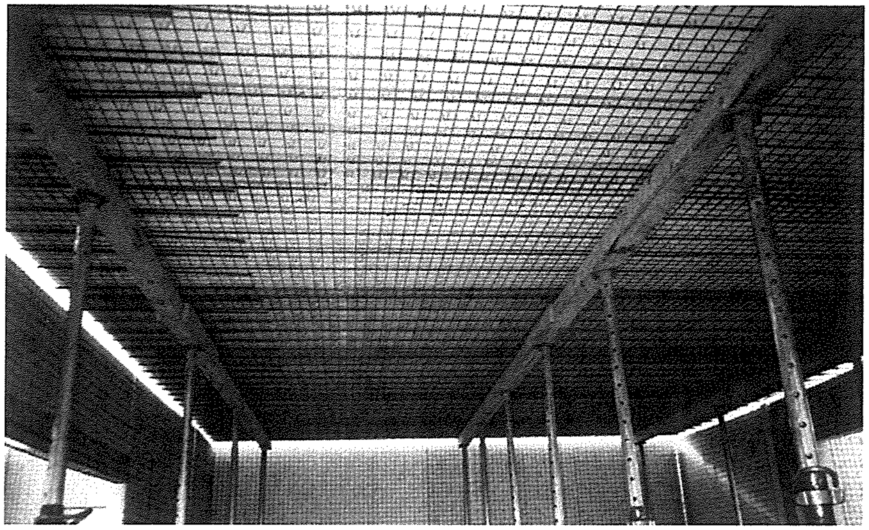


photo 7: Shoring the slab

For obtaining the maximum shoring spans, it is however inevitable that the connection of slab and wall panels at the support is thoroughly made. Otherwise the edges could be lifted and the deflection during concreting would be significantly larger.

shoring the slab panels

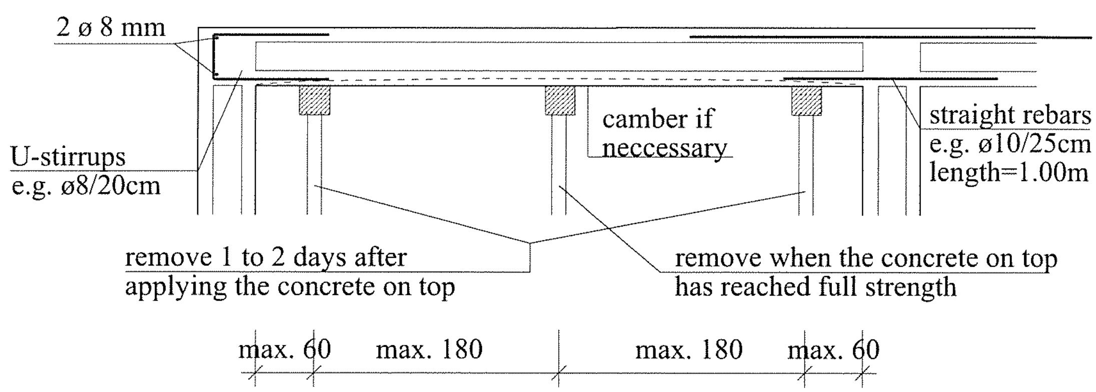


figure 1.6.b Arrangement of shoring

From these considerations result the number of supporting rows in dependence of the panel length for EPS-100 and EPS-50. The given shoring span refers to panels with 200 diagonals/m². In these cases, the shoring span depends on the deflection and not on the flexural resistance of the panel (buckling). In case of 100 diagonals/m², the flexural resistance is decisive. Therefore the shoring span can be calculated directly. As a rule they should be assumed slightly smaller than for panels with 200 diagonals.

shoring span (EPS-100, 200 diag./m², 50 mm top concrete, length in [cm])

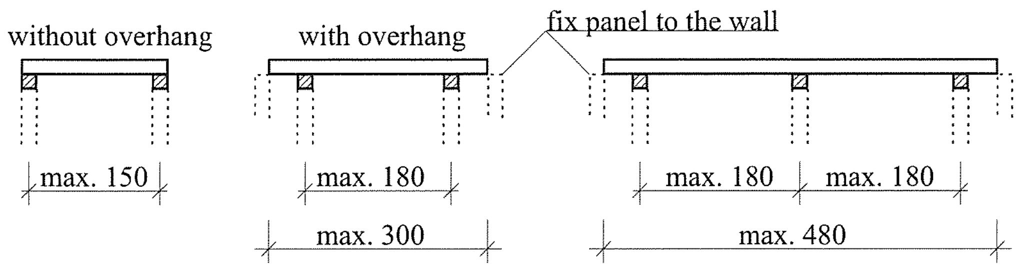


figure 1.6.c Maximum shoring spans for EPS-100 with 200 diagonals per m²

shoring span (EPS-50, 200 diag./m², 50 mm top concrete, length in [cm])

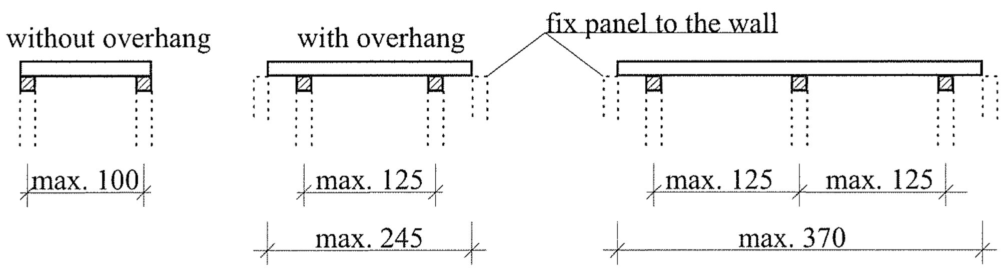
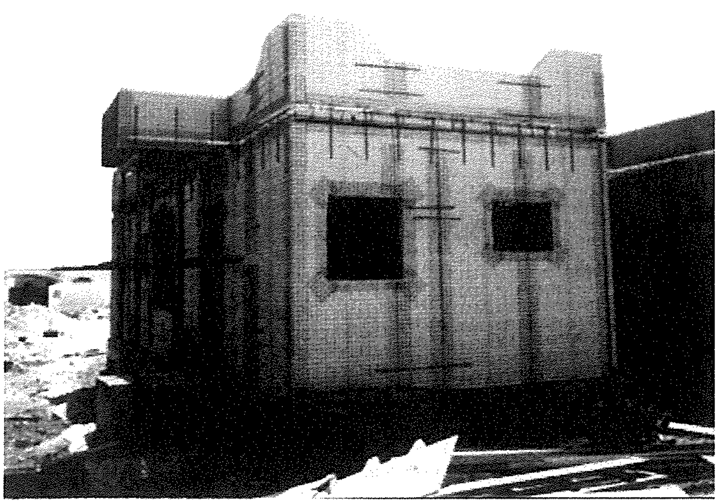


figure 1.6.d Maximum shoring spans for EPS-50 with 200 diagonals per m²

photo 8: Finished panel wall



1.7. Concreting

For details concerning the concreting jobs, see chapter 12.

1.7.1. Procedure

When working with 3D constructions both wet and dry shotcrete pumps may be used. As a rule the following order of the concreting jobs is observed:

- 1st layer of shotcrete on the walls (inside and outside).
- 1st layer of shotcrete on the bottom side of the slab. This concrete layer should be applied at least at the edges of the slab.
- Concrete on the topside of the slab.
- Completion of the shotcrete on walls and on the slab.

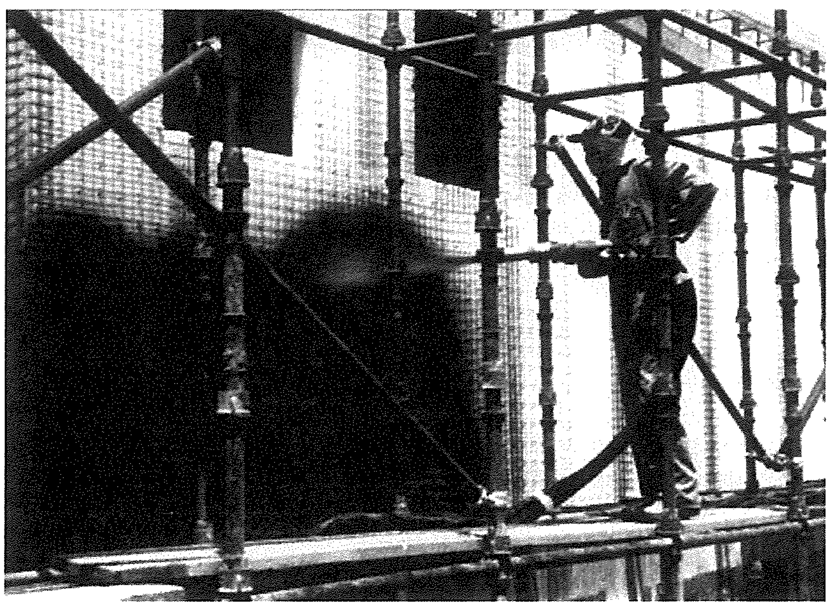


photo 9: Shotcrete jobs

1.7.2. First Shotcrete Layer

The first shotcrete layer on the wall and the lower part of the slab has to be at least 2 cm thick (up to the cover mesh) and must be 1 cm less than the total thickness (that means 5-1=4 cm) at most. This layer is kept rough and has the following tasks:

- it has to transfer the mounting loads from the slab
- it forms a solid surface for the second concrete layer

Stiffenings that may have been attached to the wall, have to be removed 2 to 3 days after applying the first layer. The gaps in the concrete will be filled during the application of the second layer.

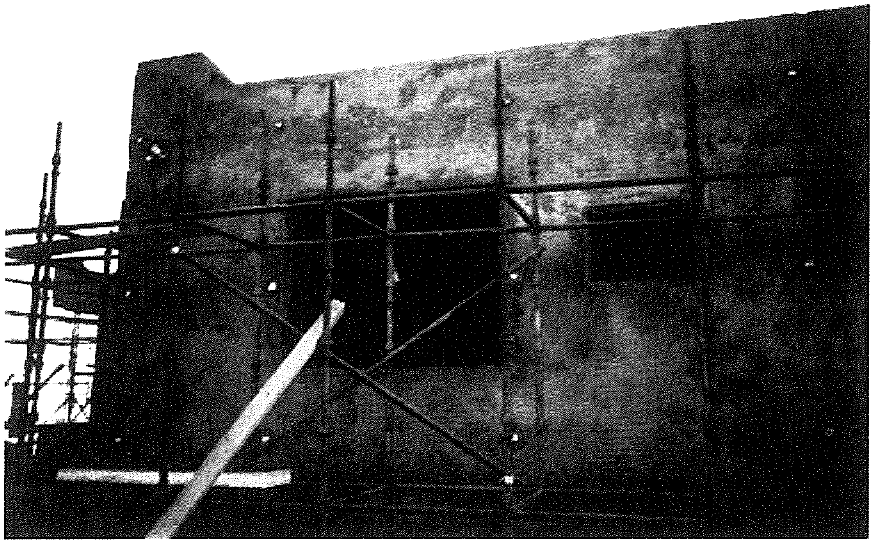


photo 10: Finished 1st layer including measuring marks

1.7.3. Second Shotcrete Layer

The second layer may be applied both manually and by machine. Especially for a thin layer the manual application of concrete can be a more useful alternative. This layer is applied within the measuring marks like regular plaster and smoothed afterwards.

1.7.4. After-treatment

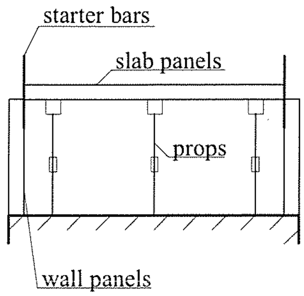
Generally the concrete is produced without admixtures. However, it is necessary to protect the surfaces from early drying out after smoothening. The following methods have proved to be worthwhile:

- covering by a PVC foil
- regular wetting

1.8. Working Sequence for Multi-storey Buildings

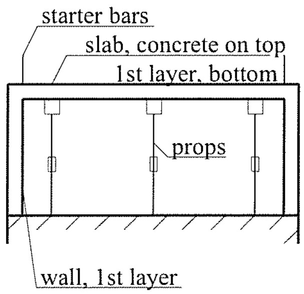
In case of a multi-storey building, the erection loads have to be observed thoroughly. Especially if erection is progressing very quickly, it is possible that the slab is yet too young to bear the entire load of the above storey. Therefore certain minimum delays have to be kept before removing the props of the slab.

1.



In the first step, the wall- and slab panels of the ground floor are erected inclusive all starter bars for the walls in the upper floor. The sketch to the left shows the example of a room with 3 rows of props.

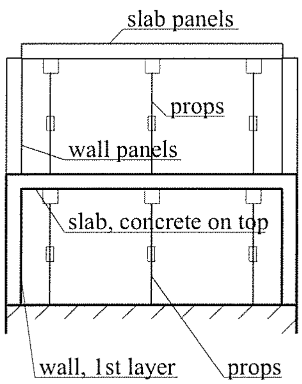
2.



In the next step, the first concrete layer is applied to the walls and the bottom side of the slab. Then the top side of the slab can be concreted.

A group of workers can eventually start to prepare the second concrete layer. However it is recommended to wait until the props are partly removed.

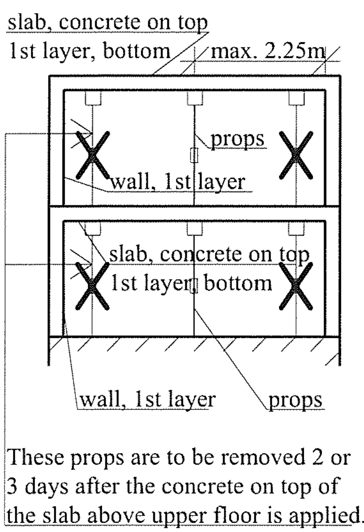
3.



In the third step, the wall- and slab panels of the upper floor are erected. Thereby it is essential that the props in both floors are set up one directly above the other.

The concreting can be continued in the first floor.

4.

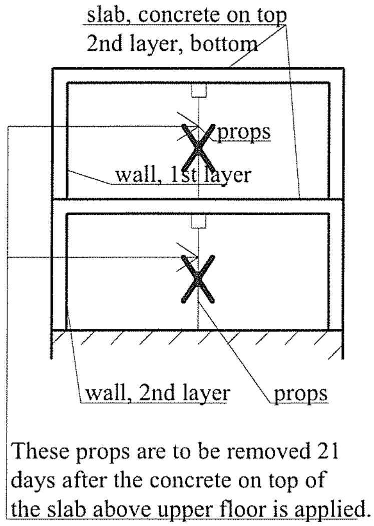


In the next step, apply the first concrete layer on the walls and bottom side of slabs as well as the additional concrete layer on the slab by analogy with the ground floor.

The props at the edges can be removed 2 - 3 days after applying the top concrete layer on the upper floor slab. Remove first the props in the upper floor. Be sure that the distance between the props and the wall does not exceed 2.25 m.

After removing the props at the edges, the 2nd concrete layer can be applied in both stories.

5.



Finally complete the concrete on the walls and the bottom side of slabs. All props can be removed 21 days after applying the top concrete layer on the upper floor slab. Again remove first the props in the upper floor. Then fill the free stripes in the area of the wooden beams with concrete.

In the end apply a final third layer of mortar if necessary.

2. Flexure

NOTATION

a .... depth of the equivalent rectangular compression stress block (area × stress)
A_G...... total area of the gross cross section (= d × b)
A_S ...... area of nonprestressed tension reinforcement
b......... width of cross section
β......... coefficient for the depth of the rectangular stress block according to ACI (0.65-0.85)
d......... effective depth of the cross section
ε .... compressive strain of concrete in ‰
ε_{MAX} .. limit of the range where stress-strain curve is assumed to be parabolic (=2 ‰)
E_S....... modulus of elasticity of steel
F_C....... compression force in concrete
F_T....... tension force in steel
fc........ specified compressive strength of concrete
fc_ε ...... compressive stress of concrete depending on compressive strain ε
f_{W28}.... cube strength of concrete at 28 days
fy .... specified yield strength of steel
M .... moment
M_D ..... moment under service dead load
M_L ..... moment under service live load
M_{MAX} max. admissible moment under service load
M_N ..... nominal flexural strength
M_U ..... ultimate factored moment
ø......... strength reduction factor according to ACI
flexure : ø = 0.90
shear : ø = 0.85
t_2 .... depth of topping concrete layer
x......... depth of compression zone
z .... lever arm of internal forces

2.1. Basics

Principally it is possible to calculate the slabs made of 3D elements according to the same standards that are also used for common reinforced concrete slabs. All rules referring to the calculation of internal forces and to the load-bearing behavior of concrete and steel may apply directly to the calculation of 3D cross sections. However, the reducing of the cross section through the EPS-core has to be taken into consideration when examining the internal forces.

Floor slabs made of 3D panels can be considered generally as one-way slab systems simply or continuously supported. Thus the tensile forces and compression forces are absorbed by the reinforcement elements (cover mesh and additional reinforcement) and by the concrete compression chord. These parts are designed with minor restrictions according to the conventional rules of reinforced construction. The shear forces, however, are transferred solely through the diagonals in case of a 3D slab without additional shear reinforcement (see chapter 3).

2.2. Stress-strain Diagrams

A standard design model makes sense only when it bases on an exact theory of the ratio between stress and strain. Thus it becomes difficult theoretically to apply the method of approximation according to DIN or the ACI method (a method of approximation itself). The following sections deal with an examination of the load bearing behavior of construction materials and the appertaining safety concept.

2.2.1. Stress-strain Diagram of Concrete

The concrete's stress-strain diagram is represented clearly as a non-linear curve not only in the relevant standards but in the entire literature as well. The most frequent mathematical description of this curve is a quadric parabola culminating at a deformation of 2 ‰. This curve is defined in the exact calculations according to DIN. Some recent standards that are supposed to adapt the calculations to the future Eurocode apply this parabolic shape within a range of up to 2 ‰ either.

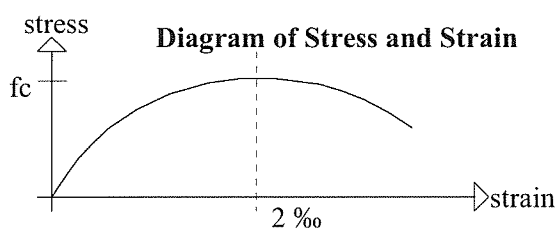
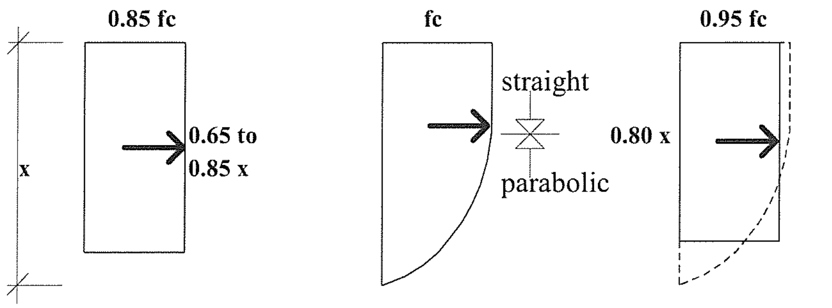


figure 2.2.1.a Typical stress-strain diagram of concrete

While in most of the standards compressive stress remains the same when exceeding this strain, the test results clearly show that compressive stress decreases when the 2 ‰ limit is exceeded. This contradiction between test results and code requirements shall be discussed again in the explanation of the safety concepts.

Compressive Stress Blocks in different Standards



ACI
(rectangle;
up to 3 ‰)

DIN
(parabolic curve
up to 2 ‰; straight
from 2 - 3.5 ‰)

DIN (approx.)
(rectangle)

figure 2.2.1.b Theoretical stress distribution

In ACI calculations the value of depth of the concrete's compressive stress block *a* lies between 0.65 x and 0.85 x.

Concrete grade up to 4000 psi (approx. 28 N/mm²): a = 0.85 x

Concrete grade beyond 4000 psi: the coefficient decreases by 0.05 per 1000 psi (approx. 7 N/mm²) but does not fall below 0.65.

In contrast to the European standards the concrete grades according to ACI indicate the design strength. The designation of concrete in European standards, however, orientates itself to cube strength at 28 days, resulting thus in the following specified compressive strength in the building

$$fc = 0.70\ f_{W28} \quad \text{(DIN)}$$

The cube strength at 28 days is referred to as f_{W28}. For higher concrete grades DIN reduces the coefficient to up to 0.55. This results in the following strengths of the standardized concrete grades:

concrete grade	B15	B25	B35	B45	B55
fc [N/mm²]	10.5	17.5	23.0	27.0	30.0

Table 2.2.1.b Concrete grades according to DIN in [N/mm²]

When designing a 3D cross section the two above-mentioned approximations can be applied in case of a rectangular stress block in the compression zone with restrictions only. These solutions presuppose that the entire theoretical compression zone is really given and, as a consequence, that the neutral axis must not lie within the insulation material. Furthermore, limiting strain lies clearly beyond 2 ‰ - this does not always seem to be recommendable for a 3D cross section. Owing to the above reasons we can recommend the design according to ACI or according to DIN's method of approximation with reservations only. Therefore, flexural design can be made according to DIN or ACI. However, for maximum moment capacity we recommend lower limits.

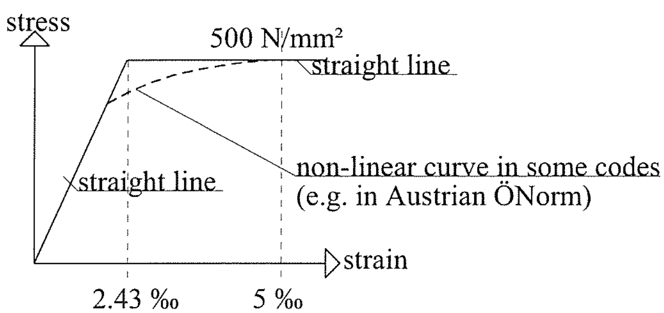
2.2.2. Stress-strain Diagram of Steel

The depth of the entire compression zone results from the ratio between the concrete's compressive strain and strain of steel. It depends thus on strain during which the reinforcement element attains yield stress. For both materials DIN and ACI define uniform limiting strain.

Both, the German DIN and the American ACI deal with a bilinear stress-strain diagram. In general, the stress-strain curve in the lower area is assumed to be straight (E_S = 20,600 kN/cm²). When assuming a constant modulus of elasticity the yield strength of steel with a steel grade of 500 N/mm² (usual panel reinforcement) is attained already at 2.43 ‰. Besides DIN limits maximum strain of steel to 5 ‰.

Diagram of Stress and Strain

figure 2.2.2.a Stress-strain diagram of steel



While concrete attains always maximum compressive strain (3 ‰) and, as a consequence, calculated strain of steel increases considerably at small moments when applying ACI's method of design, DIN limits the strain of steel to 5 ‰ and adapts the compressive strain of concrete to the requirements. The actual differences for the design of a cross section in both methods are small since the length of the lever arm of the internal forces changes only slightly.

According to ACI, the max. reinforcement ratio is equivalent to 75 % of the amount of reinforcement at which the tensile reinforcement reaches its specified yield strength just as concrete reaches its assumed ultimate strain of 3 ‰. The depth of the compression zone implies the same restriction. Owing to these correlation the ratio between compression zone and total depth can be calculated as follows:

strain limitation	**DIN**	**ACI ***
limiting compressive strain of concrete	3.5 ‰	3.0 ‰
limiting strain of steel	5.0 ‰	4.2 ‰
compression zone/effective depth	41.2 %	41.5 %

table 2.2.2.a Max. size of compression zone * According to ACI this value applies to fy = 500 N/mm²

Some standards, e.g. Austrian standard ÖNORM, use a connecting curve instead of the bilinear stress-strain diagram. As a consequence, yield strength is attained only at a higher steel strain (figure 2.2.2.a).

2.3. Models

The following chapter presents briefly the methods of flexure design according to ACI-318 (USA) and DIN-1045 and examines whether they can be used with regard to 3D systems.

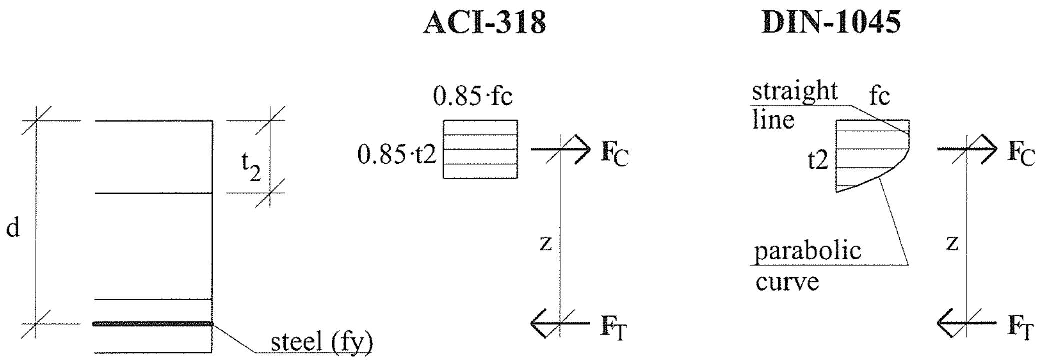


figure 2.3.a Flexure models (max. compressive strain of concrete)

2.3.1. Flexure Design according to ACI-318

The flexural strength according to ACI results from the following formulae

$$M_U = ø \cdot A_S \cdot fy \cdot (d-\frac{a}{2}) \qquad \text{i.e.} \qquad a = \frac{As \cdot fy}{b \cdot 0.85 \cdot fc} \leq \beta \cdot t_2$$

for β see paragraph 2.2.1

2.3.2. Flexure Design according to DIN-1045

The stress flow in the compression zone is assumed to be a parabolic curve up to 2 ‰ and a rectangular stress block between 2 ‰ and 3.5 ‰. Depending on compressive strain the equation for compression stress for the parabolic part is as follows:

$$fc_{\varepsilon} = fc \cdot \frac{\varepsilon}{\varepsilon_{max}} \cdot \left(2 - \frac{\varepsilon}{\varepsilon_{max}}\right)$$

$$\varepsilon max = 2 ‰$$

In addition to this, it is reckoned that the compression stress remains the same up to a compressive strain of 3.5 ‰. This sort of compression zone of concrete is also used in the future Eurocode. The maximum strain of the reinforcement is standardized to be 5 ‰ for all steel grades.

2.3.3. Flexure Design according to DIN-1045 (approximation)

$$M_{max} = \frac{0.80t_2 \cdot 0.95fc \cdot (d - a/2) \cdot b}{1.75} = 0.434 \cdot t_2 \cdot fc \cdot (d - a/2) \cdot b$$

1.75.... global safety factor
t_2........ $\leq d \cdot 0.416$
a......... $= t_2 \cdot 0.80$

According to the DIN methods the required area of reinforcement can be calculated as follows:

$$As = \frac{1.75 \cdot M}{z \cdot fy}$$

1.75.... global safety factor
M....... max. moment under service load
z......... lever arm of internal forces, approx. 0.9 d

2.4. Safety Concepts

The safety concept by ACI on the one hand, and those by DIN on the other hand differ fundamentally from each other. While DIN provides for a design with limiting strains and compares the resultant values with the internal forces factored by a global safety coefficient (=1.75), ACI brings in several safety factors at various positions of the calculation. Firstly, the moments with split safety factors (dead load = 1.4, live load = 1.7) are assessed. Then the area of steel is calculated again by applying strength reduction factors (flexure = 0.9; shear = 0.85). Even the extent of compression force being

$$F_C = ß \cdot fc \cdot 0.85\, x \cdot b$$

i.e. ß......... 0.65 - 0.85

cannot be considered to be only a simplification of geometrical correlation. However, an unequivocal assignment of a safety factor is not possible in this case.

The safety concept for the design of a 3D cross section subject to flexure must take into consideration especially the thin compression layer (concrete topping).

In comparison to the test results the deviation of the concrete's mathematical stress-strain curve represents a safety factor reduction that is not mentioned in the pertaining standards but nonetheless relevant for the design of a 3D cross section. While most standards allowing a compressive strain of concrete beyond 2 ‰ assume that stress in the range beyond 2 ‰ remains the same, the tests clearly show a decrease of compression stress in the upper area (see 2.2.1). In a common reinforced concrete beam this deficit is compensated without major restrictions during the creeping of the concrete and by the accompanying redistribution of stresses. As a consequence the position of the neutral axis is shifted downwards increasing thus the compression zone automatically. Owing to the fact that cross sections of 3D panels subject to flexure have only a thin compression zone this redistribution of stresses cannot be accepted without reservations.

Another restriction of the compression zone's depth is steel strain when yield strength is attained. While yield strength is attained quite early in standards with a bilinear stress-strain curve, other standards use a connecting curve that ends in a higher steel strain (see figure 2.2.2.a).

In the framework of a simple safety concept it is obviously necessary to limit compression force by estimating that the yield strength of steel lies in the area of the maximum strain to be expected. The strain supported by DIN (5 ‰) does not only provide for high safety but it covers local specifications of steel as well. In general, such an exception is supported also by the Austrian standard.

2.5. Flexural Resistance of 3D Cross Sections

For the design of a 3D cross section methods provided in different codes and standards like ACI or DIN can be used. However, to determine the limit of flexural resistance the use of a pure approximation model does not seem to make sense since these models generally start from a concrete cross section stretching over the entire height. Especially the current approximations require a complete existing compression zone that is replaced afterwards by a smaller figure with a rectangular stress block. That’s why a standardized method of design is possible only on account of the assessment of stresses by means of the stress-strain curve. For this end it is necessary to define the limiting strains that take into consideration the local features such as the exact specification of steel on the one hand and that observe a tighter safety concept for concrete on the other hand. These requirements result in the following assumption of strain limits:

- maximum compressive strain of 2.0 ‰ (pure parabolic stress-strain curve)
- yield strength of steel at 5.0 ‰

This limits the depth of the compression zone to 28.6 % of the effective depth when using the steel to the full. In order to avoid restrictions of safety also after the redistribution of stresses on account of creeping, the criteria to be fulfilled is the requirement that the neutral axis has to be located always within the top concrete layer.

design model (maximum strain)

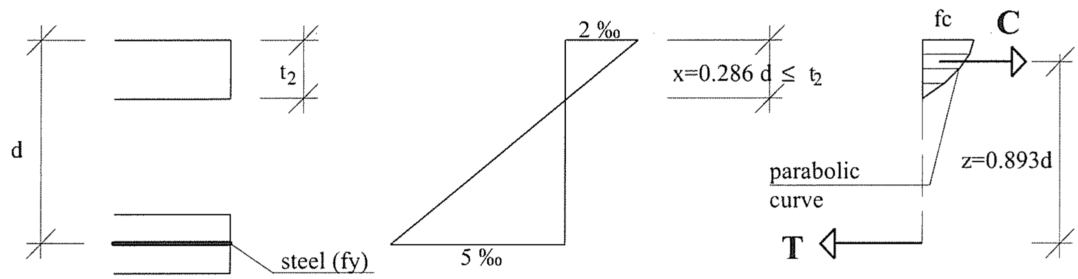


figure 2.5.a Design model

2.5.1. Flexural Resistance under Service Load (DIN)

On these conditions the maximum admissible moment under service load according to DIN (global safety factor is 1.75) can be calculated as follows

$$M = 0.0972 \cdot fc \cdot b \cdot d^2 \quad \leq \quad 0.3810 \cdot fc \cdot t_2 \cdot b \cdot (d - 0.375 \cdot t_2)$$

In the table 2.5.a the dimensions for t_2 and d_{EPS} are indicated in mm and the moments in kNm/m. Moments are calculated under service load including the safety factor according to DIN (= 1.75) at a concrete grade of 17.5 N/mm² (= B25). For other concrete grades these values have to be multiplied with fc/17.5 N/mm². The distance between the lower edge of the EPS and the center of gravity of the reinforcement is assumed to be 20 mm.

concrete layer (compression) [mm]	EPS thickness [mm]						
	40	50	60	70	80	90	100
50	20.6	24.5	28.7	33.3	38.3	43.5	49.1
60	24.5	28.7	33.3	38.3	43.5	49.1	55.1
70	28.7	33.3	38.3	43.5	49.1	55.1	61.4
80	33.3	38.3	43.5	49.1	55.1	61.4	68.0

table 2.5.a Maximum admissible moments M [kNm/m] fc=17.5 N/mm²

Hence the required quantity of reinforcement is

$$As = \frac{1.75 \cdot M}{z \cdot fy}$$

i.e. 1.75.... global safety factor
M....... max. moment under service load
z......... z can be taken from common calculation tables.

The lever arm of the internal forces depends on the degree of loading of the cross section. In general, the approximate value z = 0.9d is sufficiently accurate.

2.5.3. Flexural Resistance under Ultimate Load (ACI)

According to the ACI design model the maximum factored moments result in

$$M_U = 1.4 \cdot M_D + 1.7 \cdot M_L$$
$$M_U \leq ø \cdot M_N \quad \text{(required strength} \leq \text{design strength)}$$
$$M_U \leq 0.1531 \cdot fc \cdot b \cdot d^2 \quad \leq \quad 0.6 \cdot fc \cdot t_2 \cdot b \cdot (d - 0.375 \cdot t_2)$$

In the table 2.5.b the dimensions for t_2 and d_{EPS} are indicated in mm and the moments in kNm/m. Moments are calculated under ultimate load according to ACI at a concrete grade of 17.5 N/mm² (= B25). For other concrete grades these values have to be multiplied with fc/17.5 N/mm². The distance between the lower edge of the EPS and the centroid of tension reinforcement is 20 mm.

concrete layer (compression) [mm]	EPS thickness [mm]						
	40	50	60	70	80	90	100
50	32.4	38.6	45.3	52.5	60.3	68.6	77.4
60	38.6	45.3	52.5	60.3	68.6	77.4	86.8
70	45.3	52.5	60.3	68.6	77.4	86.8	96.7
80	52.5	60.3	68.6	77.4	86.8	96.7	107.1

table 2.5.b Maximum factored moments M_U [kNm/m] fc=17.5 N/mm²

2.6. Design of Slabs

In the following section the flexural reinforcement of a one-way slab shall be designed according to DIN and ACI.

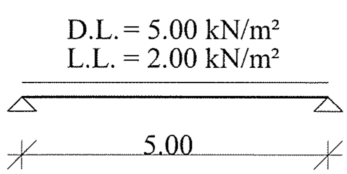


figure 2.6.a System drawing of a 3D slab

Example:

3D cross section with 50 + 100 + 60 mm (d = 180 mm)
B25 (fc = 17.5 N/mm² = 1.75 kN/cm²; approx. 2500 psi)
ST500 (fy = 500 N/mm² = 50 kN/cm²)

According to DIN the moment under service load results in

$$M = \frac{5.00^2 \cdot (5.00 + 2.00)}{8} = 21.9 \text{ kNm/m} < 55.1 \text{ kNm/m (see table 2.5.a)}$$

Hence the required quantity of reinforcement is

$$As = \frac{1.75 \cdot M}{z \cdot fy} = \frac{1.75 \cdot 21.88}{0.9 \cdot 0.18 \cdot 50} = 4.73 \text{ cm}^2/\text{m}$$

3.32 cm²/m of additional reinforcement is required in case of a panel reinforcement of 1.41cm²/m.

According to ACI the moment under ultimate load results in

$$M_U = \frac{5.00^2 \cdot (1.4 \cdot 5.00 + 1.7 \cdot 2.00)}{8} = 32.5 \text{ kNm/m} < 86.8 \text{ kNm/m (see table 2.5.b)}$$

For the first approximation *a* is assumed to be 1.5 cm. Therefore, the lever arm of internal forces results in z = d – a/2 = 17.25 cm. Hence the required quantity of reinforcement is

$$As = \frac{M_U}{z \cdot \phi \cdot fy} = \frac{32.50}{0.1725 \cdot 0.9 \cdot 50} = 4.19 \text{ cm}^2/\text{m}$$

2.78 cm²/m of additional reinforcement is required in case of a panel reinforcement of 1.41cm²/m. For this reinforcement the depth of the rectangular stress block results in

$$a = \frac{As \cdot fy}{0.85 \cdot fc \cdot b} = \frac{4.19 \cdot 50}{0.85 \cdot 1.75 \cdot 100} = 1.4 \text{ cm}$$

Hence the moment capacity follows as

$$\phi \cdot M_N = \phi \cdot As \cdot fy \cdot \left(d - \frac{a}{2}\right) = 0.9 \cdot 4.19 \cdot 50 \cdot \left(0.18 - \frac{0.014}{2}\right) = 32.6 \text{ kNm/m} > 32.5 \text{ kNm/m}$$

The admissible moments by using additional reinforcement for the most commonly used 3D slabs are shown in the tables 2.6.a and 2.6.b. These moments are calculated according to DIN and include already a safety factor of 1.75.

Basic values:

B25 (fc = 17.5 N/mm²)
ST500
concrete layer (compression) = 60 mm

EPS 100	admissible moments [kNm/m] (B25, fc=17.5 N/mm², safety factor=1.75, fy=500 N/mm², t2=60 mm)								
add. rebar ø [mm]	mesh	8	8	8	8	8	8	10	10
spacing [mm]	ø3/50	300	240	200	150	120	100	120	100
As [cm²/m]	1.41	3.09	3.51	3.93	4.76	5.60	6.44	7.96	9.27
M [kNm/m]	7.0	15.1	17.1	19.0	22.9	26.8	30.6	37.4	43.0

table 2.6.a Maximum admissible moments in [kNm/m] (EPS-100) effective depth = 180 mm

EPS 50	admissible moments [kNm/m] (B25, fc=17.5 N/mm², safety factor=1.75, fy=500 N/mm², t2=60 mm)								
add. rebar ø [mm]	mesh	8	8	8	8	8	8	10	10
spacing [mm]	ø3/50	300	240	200	150	120	100	120	100
As [cm²/m]	1.41	3.09	3.51	3.93	4.76	5.60	6.44	7.96	9.27
M [kNm/m]	5.0	10.8	12.2	13.6	16.3	19.0	21.7	26.3	28.7

table 2.6.b Maximum admissible moments in [kNm/m] (EPS-50) effective depth = 130 mm

The spacing between the rebars were chosen on the basis of a panel width of 1.20 m. As a consequence, an integral number of additional rebars per panel is required. The additional rebars should be as thin as possible and must not exceed a diameter of 10 mm.

Basing on the same data a design according to the American standard ACI results in the following ultimate factored moments:

EPS 100	ultimate moments [kNm/m] (B25, fc=17.5 N/mm², fy=500 N/mm², t2=60 mm)								
add. rebar ø [mm]	mesh	8	8	8	8	8	8	10	10
spacing [mm]	ø3/50	300	240	200	150	120	100	120	100
As [cm²/m]	1.41	3.09	3.51	3.93	4.76	5.60	6.44	7.96	9.27
Mu [kNm/m]	11.3	24.2	27.3	30.4	36.6	42.6	48.5	58.8	67.4

table 2.6.c Ultimate factored moments in [kNm/m] (EPS-100) effective depth = 180 mm

EPS 50	ultimate moments [kNm/m] (B25, fc=17.5 N/mm², fy=500 N/mm², t2=60 mm)								
add. rebar ø [mm]	mesh	8	8	8	8	8	8	10	10
spacing [mm]	ø3/50	300	240	200	150	120	100	120	100
As [cm²/m]	1.41	3.09	3.51	3.93	4.76	5.60	6.44	7.96	9.27
Mu [kNm/m]	8.1	17.2	19.4	21.6	25.9	30.0	34.0	40.9	46.6

table 2.6.d Ultimate factored moments in [kNm/m] (EPS-50) effective depth = 130 mm

3. Shear

NOTATION

α .... inclination of the diagonals
A_S .... area of reinforcement [cm²]
a_S .... area of reinforcement per m [cm²/m]
b........... width
d_{DIAG} ... diameter of the diagonals
d_{EPS}...... thickness of the EPS
d........... effective depth of the cross section
F_{DIAG} ... force per diagonal under service load
$f_{k,adm}$.. admissible buckling stress under service load
fy .... specified yield strength of steel
λ........... slenderness
lg.......... length
lg_e .... buckling length
ν........... safety factor
ν_k .... safety factor in case of buckling
n_{DIAG} ... number of diagonals per m²
n_R......... number of rows of diagonals per meter
ø........... strength reduction factor according to ACI
flexure : $ø = 0.90$
shear : $ø = 0.85$
r .... radius of gyration
S........... horizontal shear force
τ........... shear stress
τ_{02}, τ_{03} . limits of shear stress according to DIN
V.......... shear force
V_{DIAG} .. shear strength of the panel under service load
V_{ADM} ... shear strength of the cross section under service load
Vn........ nominal shear strength
Vu........ ultimate factored shear force
z........... lever arm of internal forces ($0.90 \cdot d$ - $0.95 \cdot d$)

3.1. Basics

The shear strength of 3D slabs is provided by the panel's diagonals and by possibly added shear reinforcement. Shear strength of 3D panels is limited both by the diagonals' buckling strength and by the strength of the welding joint.

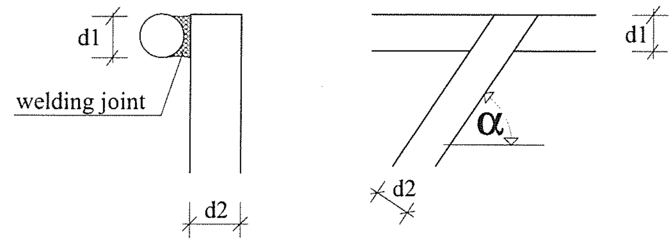


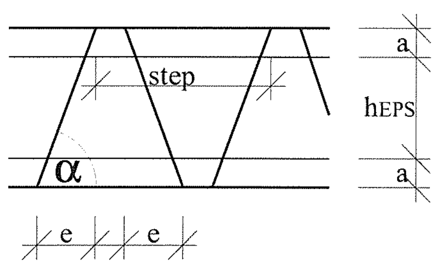
figure 3.1.a Welding joint

Tests allow to check the admissible shear force in the welding joint (including safety factor) that must correspond to at least 30 % of the greatest possible force in the diagonal. This force in the diagonal refers to the yield strength (fy) and results in a minimum force of

$$F_{DIAG} = 0.3 \cdot fy \cdot \frac{d_{DIAG}^2 \cdot \pi}{4}$$

This correlation results in a limitation of the admissible maximum stress in the diagonals of 15 kN/cm². As a general recommendation the ratio between the cover mesh's diameter and the diameter of the diagonals shouldn't be lower than 0.60.

The admissible stress in case of buckling is ascertained according to Austrian standard ÖNORM B4600 with a safety factor of 2.05. Slenderness can be calculated assuming partial fixing in the concrete with a buckling length of 75 % of the free length of the diagonal. Tests have proved that this partial fixing of the diagonals at both ends lies on the safe side when it comes to the tested 3D cross sections. Currently (1999) panels with 2 different diagonal configurations are being produced.



$$\alpha = \arctan\left(\frac{h_{EPS} + 2a}{e}\right)$$

Typ 1 : e = 40 mm

Typ 2 : e = 60 mm

figure 3.1.b Inclination of diagonals

For these panel types the clearance “a” between mesh and EPS is 13, 16 or 19 mm. The most commonly used value is 13 mm. The distance between the EPS and the reinforcing element’s center of gravity can be determined to be about 20 mm. The inclination of the diagonals is, of course, only a mean value since scattering of values for “e” is, generally speaking, a couple of millimeters.

panel type	step	diag./m²	e [mm]
type 1	100 mm	200 pcs.	40
type 2	200 mm	100 pcs.	60

table 3.1.a standard panels

In most cases panel type 1 is used as standard panel for slabs.

However, this transfer of shear forces has to be ruled out in case of the cross direction of a 3D slab. Owing to the angle between the cover mesh and the diagonals - 90° in this case - no shear forces and, as a consequence, no moments can be transferred.

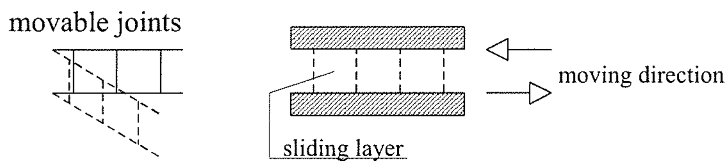


figure 3.1.c Panel in cross direction

In cross direction the diagonals and the EPS form just a sliding layer between the two concrete slabs. These act separately and lead to a considerably decreased rigidity. For a slab with a 50 mm concrete layer each on the top and bottom sandwiching a 100 mm EPS panel, there is a moment of inertia of 58.333 cm^4 per meter in main direction, and 2.083 cm^4 per meter in cross direction. Therefore, by approximation a 3D slab corresponds to a joist construction having beams in one direction and a thin slab in the other direction.

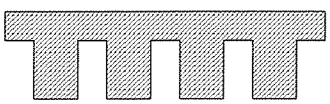


figure 3.1.d Structural equivalent system including beams in main direction and a thin slab in cross direction

Thus it is possible to design also a square slab as a one-way 3D slab. In case of major deformations only, noticeable compression forces form in the EPS. This results in a considerably improved distribution of concentrated loads and, as a consequence, it leads to additional safety in case of local overstressing that was not taken into consideration for structural analyses.

3.2. Admissible Forces in the Diagonals

Buckling load of the diagonals is determined for an effective length of 75 % of the free length.

Assumption:
angle of diagonals according to figure 3.1.b.
steel grade ST 500

buckling length $lg_e = 0.75 \cdot lg = 0.75 \cdot \frac{d_{EPS}}{\sin \alpha}$

$$\lambda = \frac{lg_e}{r} = \frac{4 \cdot lg_e}{d_{DIAG}} \xrightarrow{\text{table}} f_{k,\,adm}$$

$$F_{DIAG} = f_{k,adm} \cdot As$$

$$f_{k,adm} \leq 0.3 \cdot fy = 0.3 \cdot 50 = 15 \text{ kN/cm}^2$$

The different inclination of the diagonals for both standard panel types results in slightly different buckling forces, too. The following tables show the diagonal forces for panel type 1 and 2 basing on table 3.1.a. The pitch between mesh and EPS is 13 mm. For values beyond 13 mm the results will be slightly improved.

EPS	ø of diagonals [mm]					
[mm]	3.0	3.5	**3.8**	4.0	4.3	4.5
≤ 70	1.06	1.44	**1.70**	1.88	2.18	2.39
80	0.97	1.44	**1.70**	1.88	2.18	2.39
90	0.78	1.44	**1.70**	1.88	2.18	2.39
100	0.64	1.19	**1.65**	1.88	2.18	2.39

table 3.2.a Admissible forces in the diagonals in [kN] (panel type 1, 200 diagonals/m²)

EPS	ø of diagonals [mm]					
[mm]	3.0	3.5	**3.8**	4.0	4.3	4.5
≤ 70	1.06	1.44	**1.70**	1.88	2.18	2.39
80	0.85	1.44	**1.70**	1.88	2.18	2.39
90	0.70	1.30	**1.70**	1.88	2.18	2.39
100	0.58	1.08	**1.50**	1.84	2.18	2.39

table 3.2.b Admissible forces in the diagonals in [kN] (panel type 2, 100 diagonals/m²)

In view of the limitation of stress to 15 kN/cm² the values of line 1 apply also to all EPS thicknesses below 70 mm. The columns in bold-faced type show the currently used diameter.

The values of the admissible stress in case of buckling $f_{k,adm}$ can be found in table 3.3.a.

3.3. Admissible Stresses in Case of Buckling

Buckling of ST 500 (steel grade 500) according to Austrian standard ÖNORM B 4600
Safety factor for steel $\nu = 1.70$
Safety factor for buckling $\nu_k = 2.05$

Within the elastic range ($\lambda \geq 75$) the following applies to admissible stress in case of buckling:

$$f_{k,\,adm} = \frac{\pi^2}{\lambda^2} \cdot \frac{E}{\nu_k} \quad [\text{kN/cm}^2] \quad \text{when} \quad E = 20{,}600\ \text{kN/cm}^2$$

λ	0	1	2	3	4	5	6	7	8	9	λ
0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	0
10	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	10
20	28.4	28.3	28.2	28.1	28.0	27.9	27.8	27.7	27.6	27.5	20
30	27.3	27.2	27.1	27.0	26.8	26.6	26.5	26.3	26.2	26.1	30
40	26.0	25.9	25.7	25.5	25.3	25.1	25.0	24.8	24.6	24.4	40
50	24.3	24.1	23.9	23.7	23.5	23.3	23.1	22.9	22.6	22.4	50
60	22.1	21.9	21.7	21.4	21.1	20.8	20.6	20.3	20.0	19.7	60
70	19.4	19.0	18.7	18.3	18.0	17.6	17.2	16.7	16.3	15.9	70
80	15.5	15.1	14.7	14.4	14.1	13.7	13.4	13.1	12.8	12.5	80
90	12.2	12.0	11.7	11.5	11.2	11.0	10.8	10.5	10.3	10.1	90
100	9.9	9.7	9.5	9.3	9.2	9.0	8.8	8.7	8.5	8.3	100
110	8.2	8.0	7.9	7.8	7.6	7.5	7.4	7.2	7.1	7.0	110
120	6.9	6.8	6.7	6.6	6.5	6.3	6.2	6.1	6.1	6.0	120
130	5.9	5.8	5.7	5.6	5.5	5.4	5.4	5.3	5.2	5.1	130
140	5.1	5.0	4.9	4.8	4.8	4.7	4.7	4.6	4.5	4.5	140
150	4.4	4.3	4.3	4.2	4.2	4.1	4.1	4.0	4.0	3.9	150
160	3.9	3.8	3.8	3.7	3.7	3.6	3.6	3.6	3.5	3.5	160
170	3.4	3.4	3.4	3.3	3.3	3.2	3.2	3.2	3.1	3.1	170
180	3.1	3.0	3.0	3.0	2.9	2.9	2.9	2.8	2.8	2.8	180
190	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.5	2.5	190
200	2.5	2.5	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	200
210	2.2	2.2	2.2	2.2	2.2	2.1	2.1	2.1	2.1	2.1	210
220	2.0	2.0	2.0	2.0	2.0	2.0	1.9	1.9	1.9	1.9	220
230	1.9	1.9	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.7	230

table 3.3.a Admissible stress in case of buckling of ST500 [kN/cm²]

In view of the limitation of stress in case of buckling due to the limeted joint strength the values relevant for the practical calculation of the force in the diagonals are fully within the elastic range ($\lambda \geq 75$). Thus they are defined independently of a standard through the exact theory of Euler. As a consequence it is possible to use these values also for calculations with any other standard (e.g. ACI). Actual buckling strength is ascertained by multiplying the values of the table by the safety factor 2.05. Joint strength, too, includes a safety factor. Thus the limitation to 15 kN/cm² does not apply to the actual buckling strength.

3.4. Truss Model

If the pitch between the welding joints is small, a 3D cross section can be seen as a truss. Analogously to the calculation of a truss, shear force V can be considered a vertical component of diagonal bars and can be formulated as

$$V_{DIAG} = n_R \cdot F_{DIAG} \cdot \sin \alpha$$

In the only case of a panel with 200 diagonals/m², a top concrete layer of at least 60 mm and welding joints that are very close to each other (max. 10 mm), it will be possible to compensate the impairment of the idealized system in view of the pitch between the welding joints by assuming a safe buckling length and, thus, to neglect it for a rough analysis. If there is a wider step (e.g. 200 mm), the actual load bearing behavior will differ too much. Since panels with a wider step are used for slabs, too, it is advisable to examine them more closely.

Owing to the position of the diagonals it is not possible to assign unequivocal joints to the shear if the pitch between the welding joints is wide. While, in case of a small pitch between the diagonals (step = 100 mm, panel type 1 according to table 3.1.a) or a topping slab of the corresponding thickness, there is - at least theoretically - a point of intersection between the diagonals and the compression chord, such a point of intersection with the tension chord cannot be presupposed, not even in case of an idealized analysis. Therefore, bending moments have to be transferred in the tension chord anyway. If the pitch between the diagonals is wider (step = 200 mm), the point of intersection with the compression chord's axis is not given anymore. Another difficulty is created through the fact that the compression chord's center of gravity depends on the thickness of the top concrete layer, and, in the extreme case, it is higher than the point of intersection of the diagonal.

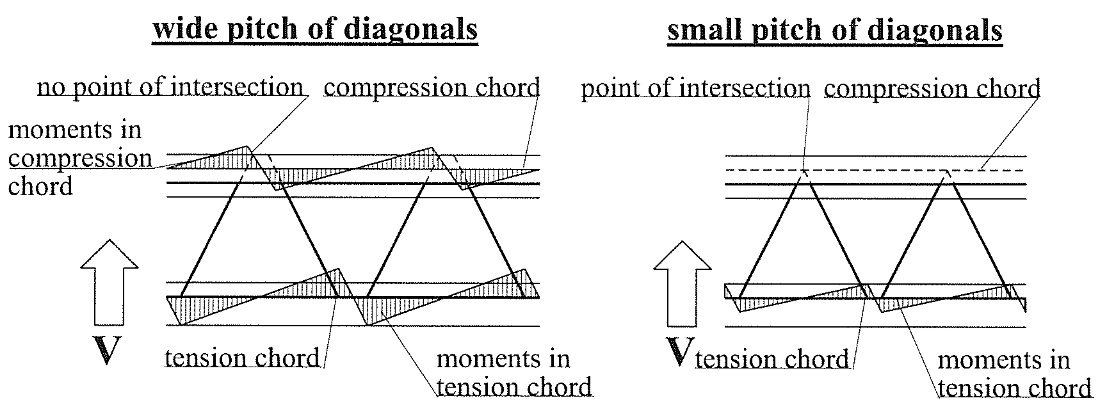
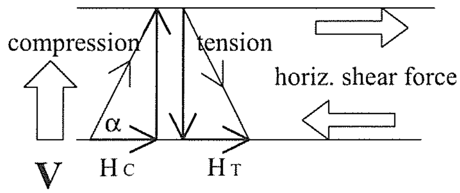


figure 3.4.a Internal forces

3.5. Horizontal Shear Force

A generally valid approach hardly seems feasible for all panel types in case of a truss. However, it is possible to consider shear force as a product of the horizontal shear force and the lever arm of the internal forces as it is done in case of common reinforced concrete beams. Horizontal shear force *S* seen as the change of tensile force per length unit corresponds exactly to the horizontal component of the diagonal.



$H_C = F_C \times \cos\alpha$

$H_T = F_T \times \cos\alpha$

i.e. F_C = compression force

F_T = tension force

figure 3.5.a Shear force model

The total horizontal shear force is formed by the sum of the horizontal components of the diagonals H_C and H_T. Thus horizontal shear force is

$$S = \Sigma(H_C + H_T)$$

Basing on this observation admissible shear force can be produced directly:

$$V_{DIAG} = S \cdot z = \Sigma(H_C + H_T) \cdot z = F_{DIAG} \cdot \cos\alpha \cdot n_{DIAG} \cdot z$$

i.e. z......... lever arm of the internal forces. This value is generally assumed to be 0.95 d. In case of very thick concrete slabs this value may become high accordingly. Since the cross section acts like a vault, the theoretical lever arm in the vicinity of the support, however, cannot be chosen to be higher than the diagonals' theoretical point of intersection.

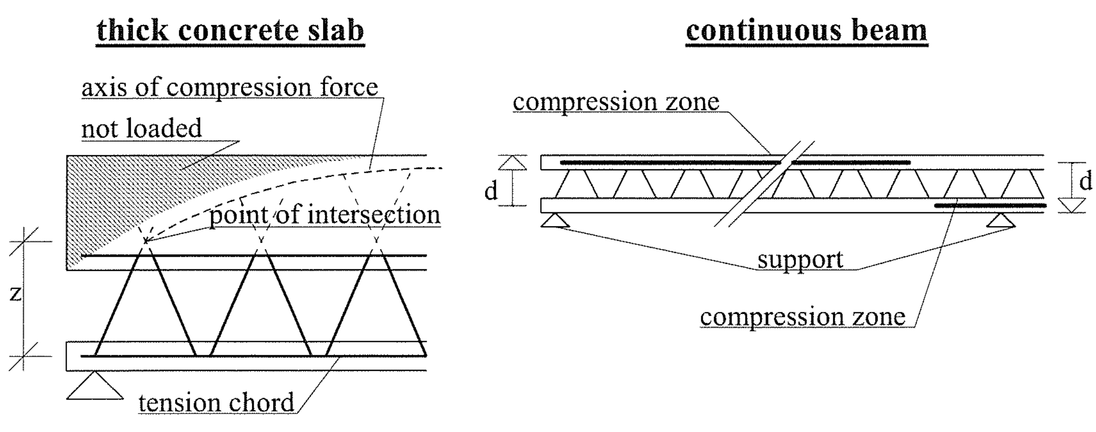


figure 3.5.b Chord forces

3.6. Standard Floor Slabs

Theoretical correlations are evaluated in the following for both panel types. These panel types correspond to table 3.1.a.

type 1 : 200 diagonals per m^2
type 2 : 100 diagonals per m^2

Stresses in case of buckling were assumed for $lg_e = 0.75$ lg and ø 3.8 mm for ST500. The effective depths apply to 50 and 100 mm EPS at a pitch between mesh and EPS of 13 mm. A concrete layer of 50 to 80 mm was assumed as a topping slab. The most interesting values for continuous girders are the two lowest ones since the bottom concrete layer generally is manufactured thinner. However, the latter is decisive at the internal support of a continuous girder. In the tables 3.6.a and 3.6.b the lever arm of the internal forces is worked out up to the theoretical point of intersection of the diagonals at most.

EPS 100		
concrete layer (compression)	diagonals per m^2	
	200	100
50 mm	14.3	9.8
60 mm	14.3	10.3
70 mm	14.3	10.9
80 mm	14.3	10.9

table 3.6.a Admissible shear forces for EPS 100 in [kN/m]

EPS 50		
concrete layer (compression)	diagonals per m^2	
	200	100
50 mm	13.8	11.0
60 mm	13.8	11.0
70 mm	13.8	11.0
80 mm	13.8	11.0

table 3.6.b Admissible shear forces for EPS 50 in [kN/m]

Shear force remains the same for thicker concrete layers. The values of both tables are admissible shear forces under service load. For the American Standard (ACI) that multiplies dead load by 1.4 and live load by 1.7, a safety factor of 2.05 has to be taken into consideration.

According to ACI for 100 mm EPS and 200 Diagonals per m^2 the nominal shear strength Vn results in

$$Vn = 2.05 \times 14.3 = 29.3 \text{ kN}$$

The max. factored shear force results in

$$Vu \leq øVn = 0.85 \times 29.3 = 24.9 \text{ kN}$$

For other standard panels the factored shear force can be found in tables 3.6.c and 3.6.d.

EPS 100		
concrete layer (compression)	diagonals per m² 200	100
50 mm	24.9	17.0
60 mm	24.9	18.0
70 mm	24.9	19.0
80 mm	24.9	19.1

table 3.6.c Factored shear forces øVn for EPS 100 in [kN/m]

EPS 50		
concrete layer (compression)	diagonals per m² 200	100
50 mm	24.0	19.2
60 mm	24.0	19.2
70 mm	24.0	19.2
80 mm	24.0	19.2

table 3.6.d Factored shear forces øVn for EPS 50 in [kN/m]

3.7. Additional Shear Reinforcement

In contrast to practical conventional reinforced construction, the shear force that is decisive for the design of the slab can be found right next to the inner edge of the support and not after a spacingof d/2 (sometimes also d).

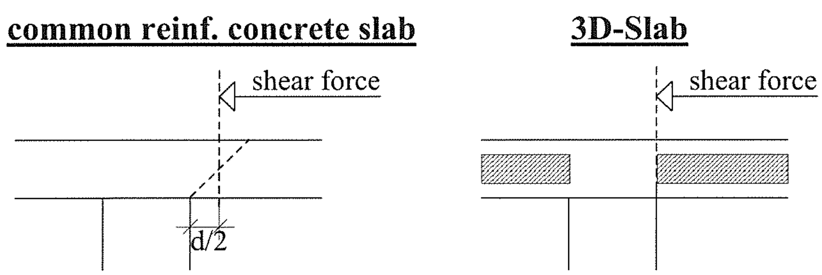


figure 3.7.a Decisive shear force

If the existing shear force exceeds the shear resistance of the panel, it will be necessary to provide for additional shear reinforcement. The following proposals make a good approach to solve this problem:

- prefabricated lattice girders
- reinforced girders cast in situ
- shear girders made of U-mesh
- special shear reinforcement elements (shear truss elements)

All the following examples are carried out as reinforced concrete constructions according to German standard (DIN). However, also the design according to any other code or standard is possible.

3.7.1. Prefabricated Shear Girders

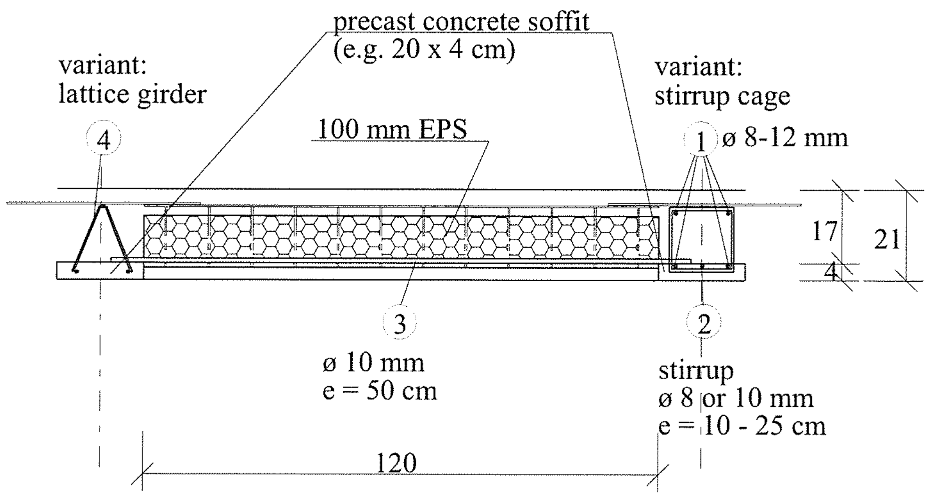


figure 3.7.1.a Prefabricated shear girders

Figure 3.7.1.a shows a shear girder with an additional longitudinal reinforcement within the girder for heavy floor loads. Prefabricated reinforcing elements, such as lattice girders, may also be used instead of stirrup cages. Example 3.7.2.b shows more or less the same girder. However, it is a girder cast in situ. Both girders are to be designed according to the rules of conventional reinforced construction. In case of a prefabricated concrete soffit it is recommended to put the 3D panels on the concrete soffits by inserting rebars (ø at least 10 mm) in spacings of 50 cm (item 3 in figure 3.7.1.a). Then the remaining concrete can be applied to the bottom conventionally by means of a shotcrete gun.

3.7.2. Shear Girders cast in situ

In general calculation of shear reinforcement follows the below scheme:

1. material : steel grade = 500 N/mm²
   diameter = 8 or 10 mm

2. shear force $V_{ADM} = \dfrac{a_s \cdot fy \cdot z}{1.75}$

i.e. 1.75.... global safety factor according to DIN
z......... approx. 0.95 d

The results are shown in table 3.7.2.a and table 3.7.2.b. These tables take into account the share of reinforcement by stirrups only. The share of concrete depends on concrete quality and width of the girder. In accordance with German code, the entire shear force has to be absorbed by reinforcement if shear stress is high ($\geq\tau_{02}$). Effective depths of 130 and 180 mm were assumed. This corresponds to panels with a 50 and 100 mm EPS core and a concrete topping of 60 mm.

An individual stirrup may also be arranged in parallel to the panels in case of minor overhangs of 3D panel's shear resistance.

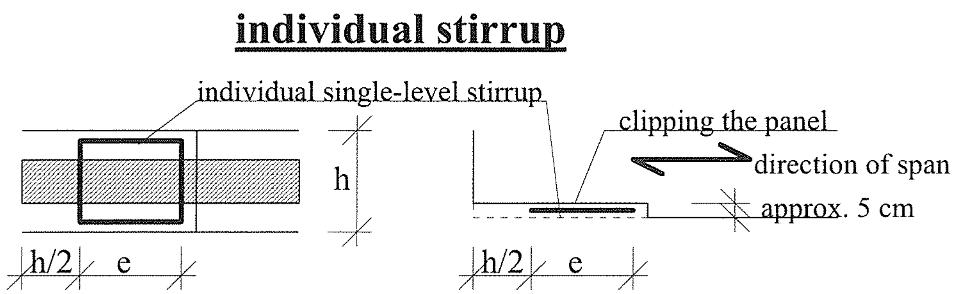


figure 3.7.2.a Individual additional stirrup

stirrup [mm]	pitch [cm]	area of steel [cm²/m]	shear strength [kN] EPS-50	shear strength [kN] EPS-100
8	25	2.01	7.09	9.82
8	20	2.51	8.87	12.28
8	15	3.35	11.82	16.37
8	10	5.03	17.74	24.56
10	25	3.14	11.09	15.35
10	20	3.93	13.86	19.19
10	15	5.24	18.48	25.58
10	10	7.85	27.71	38.37

table 3.7.2.a Admissible shear forces for individual stirrups
h = 50 + EPS + 60 mm, ST500

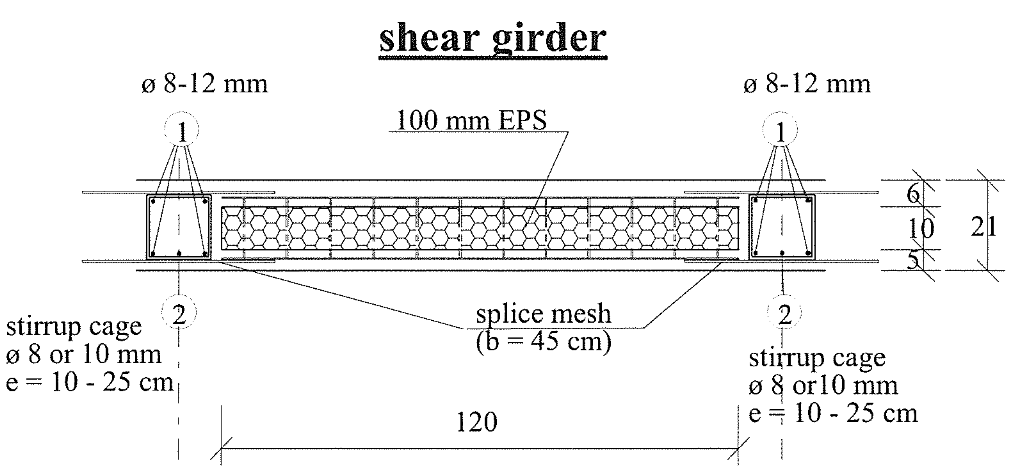


figure 3.7.2.b Shear girder

stirrup [mm]	pitch [cm]	area of steel [cm²/m]	shear strength [kN] EPS-50	shear strength [kN] EPS-100
8	25	4.02	13.44	18.61
8	20	5.03	16.80	23.27
8	15	6.70	22.40	31.02
8	10	10.05	33.61	46.53
10	25	6.28	21.00	29.08
10	20	7.85	26.25	36.35
10	15	10.47	35.01	48.47
10	10	15.71	52.51	72.71

table 3.7.2.b Admissible shear forces for stirrup cages
h = 50 + EPS + 60 mm, ST500

Example:

(panel type 1, 100 mm EPS; shear resistance of panels according to table 3.6.a)

The width of the shear girder should be 20 cm. This results in a unit spacing of 1.40 m. If actual shear force is assumed to be 24.0 kN/m, additional reinforcement will be as follows:

required shear force V_{max}	= 24.0 kN/m	=	33.60 kN/1.40m
admissible shear force of panel V_{Panel}	= 14.3 kN/m	=	-17.20 kN/1.40m
remaining shear force ΔV		=	**16.40 kN/1.40m**

chosen reinforcement = ø 8 mm, e= 25 cm (ΔV = 18.61 kN/1.40m)

Figure 3.7.2.c shows the internal forces in a slab with lattice girders. Horizontal shear force *S* results directly from tension force *T* in the diagonal and the angle of inclination α. The influence of angle β can be neglected.

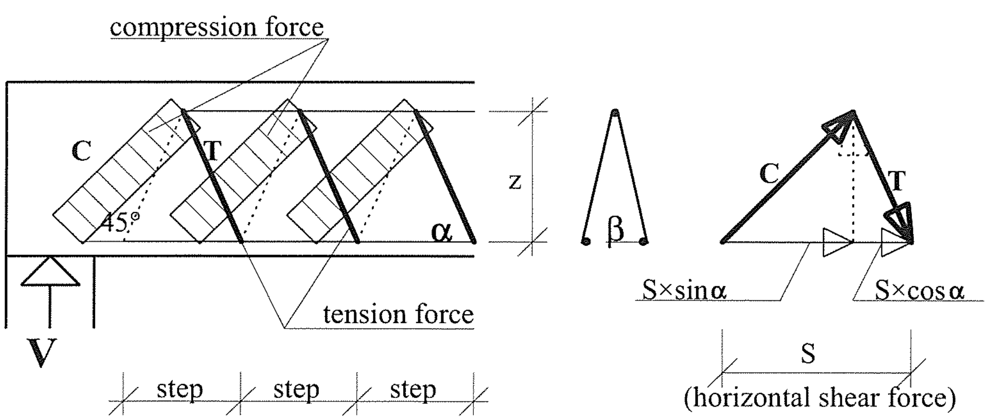


figure 3.7.2.c Internal forces in a lattice girder

By assuming a strut in the concrete inclined by 45°, the horizontal shear force *S* (= change of chord force per meter) results in the following when using a lattice girder with 2 diagonal wires:

$$S = T \cdot (\sin\alpha + \cos\alpha) = 2 \cdot \frac{a_s \cdot fy}{step} \cdot (\sin\alpha + \cos\alpha)$$

i.e. T........ tension force in a diagonal
a_s........ area of cross section of one diagonal
step.... pitch of diagonals

On the assumption of a strut inclined by 45°, $\sin\alpha$ refers to the share of the concrete's compression diagonal and $\cos\alpha$ represents the share of the lattice girder's tension diagonal.

Thus vertical shear force V results in

$$V = \frac{S \cdot z}{1.75}$$

i.e. 1.75.... (according to DIN)
z......... approx. 0.95 d

In any case the concrete strut, too, has to be checked according to the shear stress.

$$\tau = \frac{\Delta V}{b \cdot z} \leq \tau_{03} \text{ (according to DIN)}$$

i.e. ΔV ..... shear force without the panel's share
b......... width of the concrete's cross section. For a lattice girder this corresponds to the width between the panels (about 10 - 12 cm) only.

3.7.3. Splice Mesh

In order to use U-shaped bent splice mesh for the absorption of additional shear forces, it is necessary to fix these mesh stirrups to one or both edges of the slab panel. Load bearing capacity is assured only if the mesh is encased by concrete sufficiently. The concrete's effect on tension is not taken into account generally since the width of the concrete zone is mostly a couple of centimeters only.

Designing is done analogously to shear design of ordinary reinforced beams. The shear strength provided by the splice mesh results in

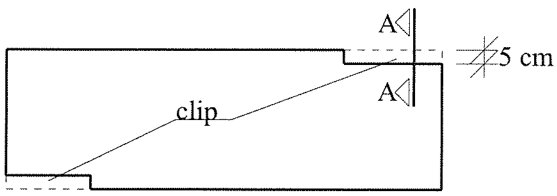
$$V_{ADM} = \frac{a_s \cdot fy \cdot z}{1.75}$$

i.e. z......... approx. 0.95 d
1.75.... global safety factor according to DIN

For a standard slab (EPS-100, 60 mm topping concrete) with as = 1.41 cm²/m and fy = 500 N/cm², admissible shear force per U-mesh is V = 6.5 kN. It is possible to fix the splice mesh to the edge of the panel or to clip a small section of the panel (about 5 cm wide). Shear stress in this 5 cm wide concrete girder is

$$\tau = \frac{V}{b \cdot z} = \frac{6.5}{5 \cdot 0.95 \cdot 18} = 0.08 \text{ kN/cm}^2 = 0.8 \text{ N/mm}^2$$

and, as a consequence, it lies clearly within the admissible range (valid for all concrete grades). According to DIN 1045 the admissible maximum shear stress is τ_{03}. When fixing the mesh between the panels this additional space requirement has to be taken into consideration for panel layout.



For section A-A see figure 3.7.3.b

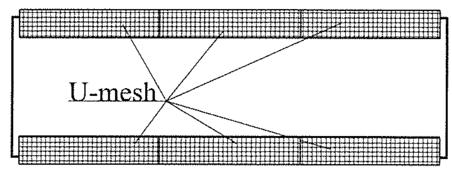


figure 3.7.3.a Additional splice mesh

Owing to the fact that in most cases the additional shear reinforcement is needed only for a minor area of the slab, the variant foreseeing the clipping of a panel's section and inserting a U-mesh seems to be the most reasonable approach. For cases that require a shear reinforcement over a major area of the panel only, the most useful solution will be to provide for a continuous stirrup cage.

The additional shear strength per meter in the previous example (EPS-100, panel type 1 with 200 diag/m², 60 mm topping concrete) is:

V_{ADM} = 6.5 kN/piece / 1.20 m = 5.4 kN/m

panel + 1 U-shaped mesh	= 14.3 + 5.4	= **19.7 kN/m**
panel + 2 U-shaped meshes	= 14.3 + 2·5.4	= **25.1 kN/m**

This results in the following values in case of EPS-50 and 60 mm topping concrete:

V_{ADM} = 4.7 kN/piece / 1.20 m = 3.9 kN/m

panel + 1 U-shaped mesh	= 13.8 + 3.9	= **17.7 kN/m**
panel + 2 U-shaped meshes	= 13.8 + 2·3.9	= **21.6 kN/m**

splice mesh between panels

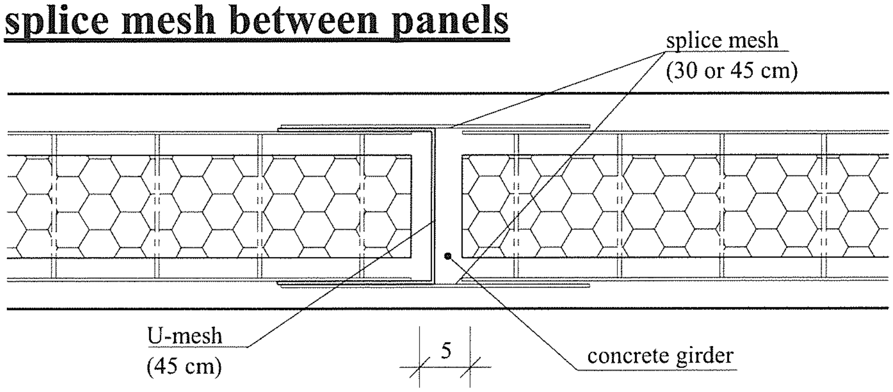


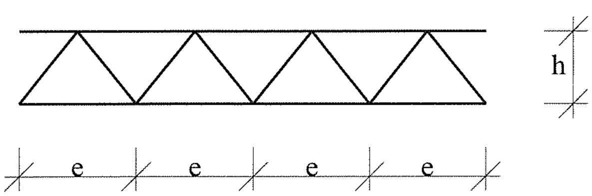
figure 3.7.3.b Splice mesh between panels

3.7.4. Special Shear Reinforcement Elements (Shear Truss Elements)

Special shear reinforcement elements are probably the most reasonable form of shear reinforcement. The following tables show shear reinforcement elements that can be produced by EVG. Admissible shear force has to be determined for 3 cases.

1. Shear reinforcement elements are made of anticorrosive steel (stainless or galvanized) and are placed without concrete encasement. The compression diagonals are subject to buckling stress. In contrast to the panel's diagonals the limitation of admissible stress to 15 kN/cm² does not apply in view of the continuous diagonal wire.
2. Shear reinforcement elements are encased by a thin concrete layer. If the concrete is not able to absorb shear forces, it must be considered only as a protection against corrosion and against buckling of the compression diagonal. Calculation follows the example of item 1.
3. The concrete girder is wide enough to be calculated as a shear girder. Calculation is done analogously to the one for lattice girders according to section 3.7.2. Shear stress in the concrete is to be proved.

shear reinforcement element



e pitch between diagonals
h height of elements (EPS thickness + 40 mm)

figure 3.7.4.a Shear reinforcement element

The distance *e* depends on the diameter of the diagonal.

ø 3.8 - ø 4.2 mm	e = 150 mm
ø 5.0 - ø 12.0 mm	e = 200 mm

This becomes necessary since the different elements are produced on two different machines. In all cases height of the used shear reinforcement elements is EPS thickness + 40 mm. The values in the table were calculated for a slab with a top concrete layer of 60 mm. The spacing between the bottom reinforcement and the EPS was assumed to be 20 mm.

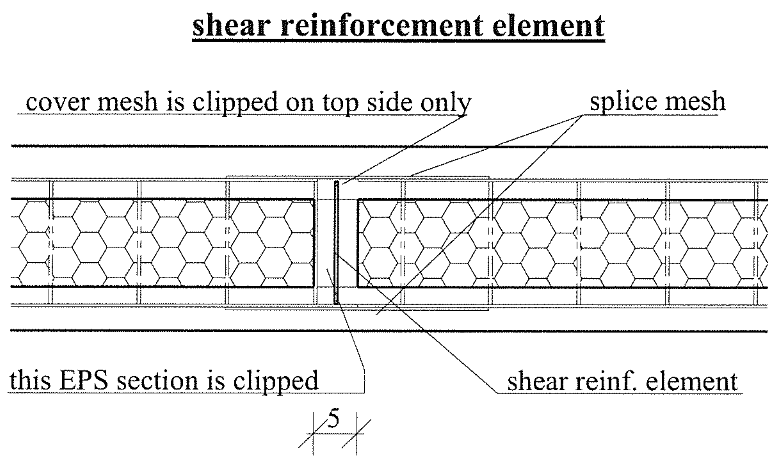


figure 3.7.4.b Arrangement of a shear reinforcement element

3.7.4.1. Anticorrosive Shear Reinforcement Elements

Table 3.7.4.a shows the admissible shear force for anticorrosive shear reinforcement elements that can be placed without concrete encasing right next to the panel. Buckling strength was taken from section 3.3.

ø [mm]	3.8	4.0	4.2	5.0	6.0	8.0	10.0	12.0
EPS 50	2.1	2.4	2.7	3.3	5.0	9.3	14.8	21.6
EPS 100	1.2	1.5	1.8	2.9	5.0	10.2	16.8	25.0

table 3.7.4.a Admissible shear forces in [kN/piece] for shear reinforcement elements not encased with concrete

3.7.4.2. Shear Reinforcement Elements with Non-bearing Concrete Encasement

Safety factor of table 3.7.4.b was assumed to be 1.75 (DIN). Concrete serves only as encasement and has no structural function.

ø [mm]	3.8	4.0	4.2	5.0	6.0	8.0	10.0	12.0
EPS 50	2.5	2.8	3.0	3.8	5.4	9.6	15.0	21.6
EPS 100	2.9	3.2	3.5	4.6	6.6	11.7	18.3	26.3

table 3.7.4.b Admissible shear forces in [kN/piece] for shear reinforcement elements with non-bearing concrete encasement

3.7.4.3. Shear Reinforcement Elements with Bearing Concrete Encasement

The values of table 3.7.4.c start out from shear reinforcement elements that are encased with concrete, either. In this case, however, concrete has to fulfill a structural function. Therefore it is necessary to prove shear stress that, according to DIN, must lie below τ_{03}. In general, a 4-5 cm wide concrete strip is not sufficient anymore for thicker diameters.

ø [mm]	3.8	4.0	4.2	5.0	6.0	8.0	10.0	12.0
EPS 50	3.6	3.9	4.3	4.6	6.7	11.9	18.5	26.7
EPS 100	4.7	5.2	5.8	6.3	9.1	16.2	25.4	36.5

table 3.7.4.c Admissible shear forces in [kN/Piece] for shear reinforcement elements with bearing concrete encasement

4. Compression

NOTATION

A_C...... area of cross section of concrete
A_G...... area of gross cross section of 3D wall
as .... reinforcement area per meter
β_d....... ratio between factored dead load and factored total load
b......... width of wall (mostly 1 m)
δ......... moment magnification factor according to ACI
d......... effective depth
e......... max. eccentricity of load application under service load
e_{max}.... max. admissible eccentricity of load application under service load
E_C...... modulus of elasticity of concrete
fc........ specified compressive strength of concrete
fy .... specified yield strength of steel
h......... overall thickness of wall ($concrete_{INSIDE}$+EPS+$concrete_{OUTSIDE}$)
I .... moment of inertia of wall cross section
λ......... slenderness
lg_e...... effective length of compression members (buckling length)
lg_U...... unsupported length of compression members
m........ eccentricity of load application under service load referring to the mid third *x*
M .... bending moment
ν......... safety factor
N........ admissible compression force under service load
N_0...... admissible compression force under service load without considering buckling
ø......... strength reduction factor according to ACI
compression : ø = 0.70
shear : ø = 0.85
P_C....... Euler's buckling load
P_U...... max. factored axial load
r .... radius of gyration
s .... distance of center of gravity to the compression edge
t_1 .... thickness of external concrete shell (tension edge)
t_2 .... thickness of internal concrete shell (compression edge)
Vc ...... nominal shear strength provided by concrete
Vn...... nominal shear strength
Vs ...... nominal shear strength provided by shear reinforcement
Vwu... ultimate factored shear force
W....... section modulus of wall cross section referring to the compression edge
x......... mid third of wall cross section referring to the compression edge

4.1. Basics

All 3D walls can be designed as load-bearing walls. In the following chapter the admissible vertical force is calculated according to DIN's method of approximation that we recommend. This method was chosen on account of its ease of operation and reliable results proven by several tests in different countries. Nevertheless it is also possible to calculate the structural resistance of 3D walls according to conventional methods such as those defined by DIN and ACI for reinforced concrete walls. Therefore the method according to ACI is also shown in a brief way. These methods take into account the slenderness of the wall through an additional lever arm for the vertical load. Furthermore the calculations are made with an additional eccentricity that bases on the inaccuracies during the execution of the building project, unconsidered deformations on account of creep and shrinkage or on different influences of temperature. In addition, eccentricity takes into consideration the inclination of the slab at the point of support caused by deflection. For 3D walls it is possible to reckon with minimum eccentricity between 20 and 40 mm. An instruction how to estimate the minimum eccentricity is to find in section 9.1.1.

Wall openings

Those wall parts that are located above an opening are calculated as beams or deep beams. This case given it will be necessary to overlap the joints between the wall and the slab on both sides with splice mesh along the entire length. For the sake of simplicity most of the door and window lintels are designed as simply supported beams.

Transfer of lateral forces - earthquake forces

In addition to the vertical loads the 3D walls are also subject to shear in plane of wall through lateral forces (wind or earthquake) especially in the first few floors of a building. For this case the walls are designed as cantilever shear walls (ACI).

Lateral loads that may act simultaneously in X- and Y direction are transferred to the foundation via the 3D shear walls by means of foundation anchors. Earthquake loads are simulated by applying lateral loads at the level of the floor slabs. The overall lateral load follows from the weight of the storey and a standardized value for the occurring horizontal acceleration. As a minimum load 1/100 of the storey weight can be assumed as lateral load.

4.2. Calculation according to DIN 1045

4.2.1. Approximation Method

In general, mesh reinforcement of 3D walls is of less importance (a_S=1.41cm²/m) so we could take the view that the amount of reinforcement is negligible for the first approximation. To determine the load bearing capacity of slender non-reinforced concrete cross sections DIN 1045 offers a simple method of approximation with increased safety factor. In a somehow modified form this method can also be applied to the double-shell system of 3D walls. It can be assumed that the concrete's effect on tension is not taken into consideration and that a gaping joint occurring under service load must reach until the center of gravity of the overall cross section at most. This would imply that compression force has to be taken by one wall shell only in the extreme case. By approximation, the admissible axial compression force N_0 (without considering buckling) of the unreinforced double-shell wall cross section can be determined according to the following equations. The concrete shells, however, may also have different thicknesses:

$$F_o = \frac{1}{v} \cdot b \cdot fc \cdot k_l \qquad (1)$$

with $k_l = t_1 \cdot \left(1 - \frac{e}{e_{max}}\right) + t_2$ and $e_{max} = s - \frac{t_2}{2}$

$$s = \frac{t_2 \cdot \frac{t_2}{2} + t_1 \cdot \left(h - \frac{t_1}{2}\right)}{(t_1 + t_2)}$$

i.e.

v .... safety factor consisting of partial safety factors e.g. $v = 3.0$

The value of k_l describes an approximation that takes into account the nonlinear deformation behavior of the concrete.

Since the concrete layers are very small and, as a consequence, the usual inaccuracies during the execution of the building project have rather strong effects we recommend to generally take safety factor 3.0 when using 3D walls.

In addition to the determination of admissible load of unreinforced cross sections it is essential to prove safety against buckling of slender compressed members by means of reduction factor k_2 according to equation (2). This factor takes into account by approximation the influence of unintentional eccentricity and wall bending according to the theory of the second order.

$$k_2 = \left[1 - \frac{\lambda}{140} \cdot \left(1 + \frac{m}{3}\right)\right] \qquad (2)$$

i.e.:

$m = \frac{e}{x}$ — eccentricity of load application referring to the mid third under service load

$e = \frac{M}{F}$ — biggest eccentricity of load application in the central third of buckling length under service load

$x = \frac{W}{A_C}$ — mid third of wall cross section referring to the compression edge

$A_C = (t_1+t_2) \cdot b$ — concrete cross section of 3D wall

$I = b \cdot \left[t_1 \cdot \left(h - s - \frac{t_1}{2}\right)^2 + t_2 \cdot \left(s - \frac{t_2}{2}\right)^2 + \frac{t_1^3 + t_2^3}{12} \right]$ — moment of inertia of wall cross section

$W = \frac{I}{s}$ — section modulus of wall cross section referring to compression edge (see equation (1))

lg_e — effective length of 3D wall (mostly height of wall)

$\lambda = \frac{lg_e}{r}$ — slenderness

$r = \sqrt{\frac{I}{A_C}}$ — radius of gyration

Thus the admissible compression force for a 3D wall is

$$N = k_2 \cdot N_0 \qquad (3)$$

This simple method of approximation for the determination of the admissible compression force in case of different 3D walls (thickness of concrete shells) and concrete grades is applied also by other standards (e.g. Austrian standard ÖNORM) and provides for workable approximate values that can be considered to be on the safe side. Using the global safety factor $\nu = 3.0$ corresponds very well with Austrian code ÖNORM B3350, either.

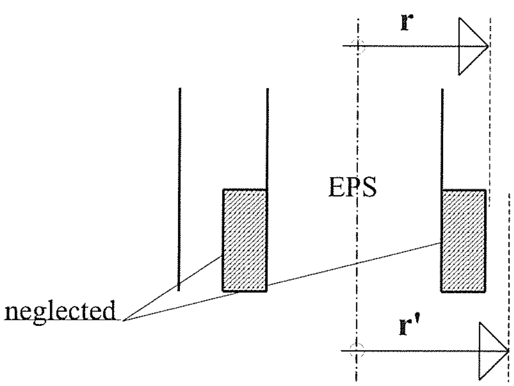
As a consequence, compression force is:

$$N = \frac{1}{3} \cdot b \cdot fc \cdot k_1 \cdot k_2 \qquad (4)$$

The method of approximation must be used for a wall slenderness of $\lambda \leq 70$ only. In chapter 9 several methods are shown how to adapt the buckling length to this requirement.

4.2.2. Buckling in Case of Minor Loads

If the load of a wall is very small, it will be possible to increase slenderness to values between 70 and 100. This can be achieved by neglecting a part of the concrete shells and the accompanying increase of the radius of gyration. However, eccentricity for determination of k_1 refers to the original cross section of the wall. Owing to the fact that the mathematical cross section of the concrete decreases rather quickly, this method can be applied for very small loads only (e.g. roof loads).



r radius of gyration

r' radius of gyration of the reduced cross section

figure 4.2.2.a Theoretical decrease of the cross section

The diagrams of section 4.6 took this correlation for slenderness beyond 70 into account.

Maximum buckling length of a 3D wall with a reduced cross section must be smaller than

$$lg_e \leq 70 \cdot h/2 = 35 \cdot h$$

For stability reasons this slenderness can be recommended also for unloaded walls, e.g. for free-standing boundary walls.

buckling length	EPS 50		EPS 100	
concrete thickness	40 mm	50 mm	40 mm	50 mm
$\lambda = 70$	3.25	3.64	4.97	5.35
$lg_e = 35$ h	4.55	5.25	6.30	7.00

table 4.2.2.a Recommended maximum effective length of 3D walls in [m]

If these values of slenderness are exceeded, a more accurate method will have to be applied. This case given it will be necessary to take into consideration also the wall's additional deformation through the diagonals that may make up quite a considerable portion of the total deformation.

4.2.3. Walls with Asymmetrical Cross Section

Especially prefabricated walls may have quite different concrete shell thicknesses in parts - in contrast to walls cast in situ with shotcrete. In this case the outer concrete shell has the required minimum thickness. The inner concrete shell constitutes in most cases the load-bearing part of the wall and needs to be designed also with regard to thermal and acoustic insulation. For a continuous thermal insulation as shown in figure 11.3.b a thicker inner concrete layer may have more structural advantages.

The method according to section 4.2.1 may be applied by analogy. For the transfer of compression forces the inner concrete shell only can be used. If the inner concrete shell is thicker than 10 cm, it will have to be reckoned with an additional eccentricity. At least an unintended eccentricity of $t_2/10$ is advisable, i.e. t_2 refers only to the thickness of the compressed concrete shell. In contrast to a 3D wall having very thin concrete shells the stresses in this concrete wall are to be assumed to be triangular or trapezoidal.

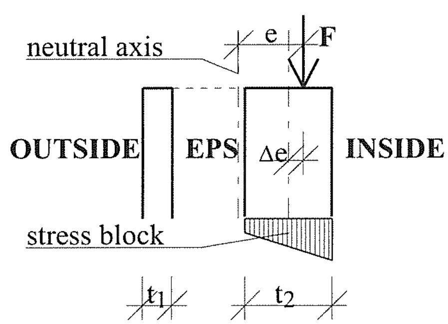


figure 4.2.3.a Wall with asymmetrical cross section

Intermediate values can be interpolated linearly. While thickness t_2 always refers to the thickness of the inner concrete shell, slenderness λ can be determined with the radius of gyration of the total cross section.

4.3. Calculation according to ACI 318-89

Effects of slenderness can be taken into account by means of the "moment magnification method" of ACI 318-89, section 10.11.5. In this case, the occurring moment is increased in dependence of slenderness and amount of vertical load by factor δ.

Thus factor δ is

$$\delta = \frac{1}{1 - \frac{P_U}{øP_C}}$$

i.e.

P_U ...... max. factored load
P_C ...... buckling load acc. to the theory of Euler
ø......... strength reduction factor = 0.70

Thus buckling load according to the theory of Euler is

$$P_C = \frac{\pi^2 \cdot E_C \cdot I_E}{(lg_e)^2 \times (1 + \beta_d)}$$

i.e.

I_E .... effective moment of inertia of the cross section. This effective moment of inertia corresponds to 1/5 of the moment of inertia of the gross cross section (for calculation see explanation of formula 2, paragraph 4.2.1).

The moment is multiplied by the resultant value for δ. Afterwards the internal forces (moment and vertical load) are compared with the load bearing capacity of the wall cross section. In general, the influence of the reinforcement can be neglected in this context. Owing to the complex calculation the evaluation of the mathematical correlation is done by means of a computer program (see section 4.5).

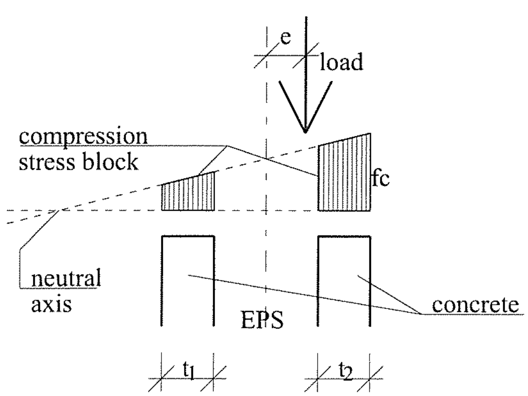


figure 4.3.a Stress pattern in a 3D wall. The stress and expansion flow can be assumed to be linear.

4.4. Shear Strength of 3D Walls according to ACI 318-89

The transfer of lateral loads such as wind loads or earthquake loads requires to take into account the shear design of walls. Especially in seismic regions it is necessary to adapt design to the respective transfer of lateral loads. In this case the use of connecting reinforcement elements in the foundation on both sides of the wall is vital. The wall's shear design can then be executed with minor restrictions analogously to the shear design of reinforced beams. This chapter will deal with design according to the provisions of the American standard ACI. The only modification with regard to the shear design of a beam is the reduction of the effective depth of the cross section to 80% of the actual wall length. When taking this reduction into consideration it is possible to design shear also according to any other standard.

Shear design of walls in plane of wall follows the below formulae

$$Vwu \leq ø \cdot Vn$$
$$Vn = Vc + Vs$$

The upper limit of Vn bases on tests and results in

$$Vn \leq \frac{5}{6} \cdot \sqrt{f_c} \cdot (t_1 + t_2) \cdot d$$

Hence

ø......... strength reduction factor = 0.85
d......... effective depth = 0.80 · horizontal length of the wall

The nominal shear strength Vc provided by concrete results in

$$Vc = \frac{1}{6} \cdot \sqrt{f_c} \cdot (t_1 + t_2) \cdot d$$

fc in [N/mm²]

If shear force Vwu exceeds the shear strength ø·Vc, an additional reinforcement has to be added.

$Vs = as \cdot fy \cdot d$ shear strength provided by reinforcement

i.e. as .... area of reinforcement per meter. This reinforcement has to be placed on both faces of the wall in longitudinal (vertical) and transverse (horizontal) direction.

Example:

Shear strength of a 3D wall (length: lw = 1 m)

cover mesh:	(3/50/50)	$A_s = 2 \times 1.41\ cm^2/m = 282\ mm^2/m$
steel grade:	ST 500	$fy = 500\ N/mm^2$
concrete grade:	B15	$fc = 10.5\ N/mm^2$

thickness of both concrete layers $t_1+t_2 = 2 \times 50 = 100$ mm

$$Vc = \frac{1}{6} \cdot \sqrt{fc} \cdot d \cdot b = \frac{1}{6} \cdot \sqrt{10.5} \cdot 100 \cdot 0.8 \quad = \quad 43.2\ kN$$

$$Vs = As \cdot fy \cdot d = 282 \cdot 500 \cdot 0.8 \quad = \quad 112.8\ kN$$

$$Vwu \le ø \cdot Vn = 0.85 \cdot (43.2+112.8) \quad = \quad \underline{132.6\ kN}$$

In case of earthquake loads the influence of concrete on total shear strength has to be neglected. During an earthquake cracks form in the concrete and reduce shear strength.

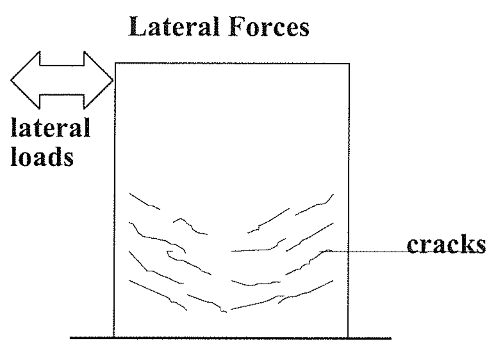


figure 4.4.a Cracks in a wall after an earthquake.

4.5. Computer Printouts

The following chapter shows the printouts of EVG programs. These programs run according to DIN and ACI and indicate also a comparative calculation according to ÖNORM for central loading.

4.5.1. 3D Wall with 50 mm EPS (DIN-1045)

```
EXAMPLE 1

CHARACTERISTICS OF 3D PANELS
----------------------------

THICKNESS OF EPS                         e =  50    mm
WIRE DIAMETER USED IN COVER MESH         d =   3.0  mm
MESH SIZE                                a =  50    mm
STEEL AREA PER 1m OF COVER MESH        As1 = 141    mm²/m
DISTANCE EPS - COVER MESH (13;16;19)    de =  13    mm
WIRE DIAMETER OF TRUSS WIRES            dd =   3.8  mm
QUANTITY OF TRUSS WIRES PER m²           n = 100
WEIGHT OF COVER MESH (ONE LAYER)        gg =   2.22 kg/m²
WEIGHT OF TRUSS WIRES PER m²            g2 =   1.08 kg/m²
WEIGHT OF 3D PANEL PER m²               g3 =   6.52 kg/m²



APPROXIMATE METHOD FOR DESIGNING COMPR. MEMBERS (DIN 1045)
=========================================================
DESIGN YIELD STRENGTH OF STEEL          fy = 500    N/mm²
DESIGN COMPR. STRENGTH OF CONCRETE      fc =  10.5  N/mm²

THICKNESS OF CONCRETE PLASTER tens.     t1 =  50    mm
THICKNESS OF CONCRETE PLASTER compr.    t2 =  50    mm
UNSUPPORTED LENGTH OF COMPR.MEMBER      WH =   2.80 m

ECCENTRICITY   AXIAL LOAD    BENDING MOMENT
     e        (permissible)  (permissible)
    (mm)         (kN/m)         (kNm/m)
============================================

    20            153            3.1
    25            138            3.5
    30            125            3.7
    35            112            3.9
    40             99            4.0
    45             88            4.0
    50             77            3.8

APPROXIMATE AXIAL LOAD ÖNORM B 3350     Po =  133 kN/m
```

4.5.2. 3D Wall with 100 mm EPS (DIN-1045)

EXAMPLE 2

CHARACTERISTICS OF 3D PANELS

THICKNESS OF EPS	e =	100	mm
WIRE DIAMETER USED IN COVER MESH	d =	3.0	mm
MESH SIZE	a =	50	mm
STEEL AREA PER 1m OF COVER MESH	As1 =	141	mm²/m
DISTANCE EPS - COVER MESH (13;16;19)	de =	13	mm
WIRE DIAMETER OF TRUSS WIRES	dd =	3.8	mm
QUANTITY OF TRUSS WIRES PER m²	n =	100	
WEIGHT OF COVER MESH (ONE LAYER)	gg =	2.22	kg/m²
WEIGHT OF TRUSS WIRES PER m²	g2 =	1.48	kg/m²
WEIGHT OF 3D PANEL PER m²	g3 =	7.92	kg/m²

APPROXIMATE METHOD FOR DESIGNING COMPR. MEMBERS (DIN 1045)

DESIGN YIELD STRENGTH OF STEEL	fy =	500	N/mm²
DESIGN COMPR. STRENGTH OF CONCRETE	fc =	10.5	N/mm²
THICKNESS OF CONCRETE PLASTER tens.	t1 =	50	mm
THICKNESS OF CONCRETE PLASTER compr.	t2 =	50	mm
UNSUPPORTED LENGTH OF COMPR.MEMBER	WH =	2.80	m

ECCENTRICITY e (mm)	AXIAL LOAD (permissible) (kN/m)	BENDING MOMENT (permissible) (kNm/m)
30	194	5.8
38	179	6.7
45	164	7.4
53	150	7.9
60	136	8.2
68	123	8.3
75	110	8.2

APPROXIMATE AXIAL LOAD ÖNORM B 3350   Po = 231 kN/m

4.5.3. 3D Wall with 50 mm EPS (ACI 318)

```
EXAMPLE 3

CHARACTERISTICS OF 3D PANELS
----------------------------

SPECIFIED YIELD STRENGTH OF STEEL       fy = 500.0  N/mm²
SPECIFIED COMPR. STRENGTH OF CONCRETE   fc =  10.5  N/mm²

WIRE DIAMETER USED IN COVER MESH         d =   3.0  mm
MESH SIZE                                a =  50    mm
DISTANCE EPS - COVER MESH (13;16;19)    de =  13    mm
WIRE DIAMETER OF TRUSS WIRES            dd =   3.8  mm
QUANTITY OF TRUSS WIRES PER m²           n = 100
WEIGHT OF COVER MESH (ONE LAYER)        gg =   2.22 kg/m²
WEIGHT OF TRUSS WIRES PER m²            g2 =   0.99 kg/m²
WEIGHT OF 3D PANEL PER m²               g3 =   6.43 kg/m²

THICKNESS OF EPS                         e =  50    mm
THICKNESS OF CONCRETE PLASTER tens.     t1 =  50    mm
THICKNESS OF CONCRETE PLASTER compr.    t2 =  50    mm
STEEL AREA PER 1m OF COVER MESH        As1 = 141    mm²/m
DISTANCE FROM COMPR.FIBER TO STEEL      d1 = 118    mm


DESIGN STRENGTHS OF 3D ELEMENTS ACCORDING TO ACI 318-95
========================================================
FLEXURE STRENGTH OF 3D SLABS            Mu =   6.9  kNm/m
SHEAR STRENGTH OF 3D SLABS              Vu =  17.4  kN/m
SHEAR STRENGTH OF 3D WALLS (INPLANE)   Vwu = 143.5  kN/m


DESIGN OF 3D WALLS UNDER COMPRESSION
(Moment Magnification Method, Chapter 10.11.5 )
===============================================
UNSUPPORTED LENGTH OF COMPR. MEMBER (m)      lu =   2.8
RATIO OF MAX.AXIAL D.L. TO MAX.AXIAL L.L.    Bd =   0.7

   MOMENT         FACTORED       ACTUAL       ECCENTRICITY
MAGNIFICATION    AXIAL LOAD   ALLOWED MOMENT
   FACTOR            Pu           Mmax             e
   delta           (kN/m)        (kNm/m)          (mm)
========================================================
   1.252             87           5.7              65
   1.368            116           6.3              55
   1.506            145           6.7              47
   1.676            174           6.9              40
   1.888            203           6.9              34
   2.162            232           6.6              29
```

4.5.4. 3D Wall with 100 mm EPS (ACI 318)

```
EXAMPLE 4

CHARACTERISTICS OF 3D PANELS
----------------------------

SPECIFIED YIELD STRENGTH OF STEEL      fy = 500.0  N/mm²
SPECIFIED COMPR. STRENGTH OF CONCRETE  fc =  10.5  N/mm²

WIRE DIAMETER USED IN COVER MESH        d =   3.0  mm
MESH SIZE                               a =  50    mm
DISTANCE EPS - COVER MESH (13;16;19)   de =  13    mm
WIRE DIAMETER OF TRUSS WIRES           dd =   3.8  mm
QUANTITY OF TRUSS WIRES PER m²          n = 100
WEIGHT OF COVER MESH (ONE LAYER)       gg =   2.22 kg/m²
WEIGHT OF TRUSS WIRES PER m²           g2 =   1.40 kg/m²
WEIGHT OF 3D PANEL PER m²              g3 =   7.83 kg/m²

THICKNESS OF EPS                        e = 100    mm
THICKNESS OF CONCRETE PLASTER tens.    t1 =  50    mm
THICKNESS OF CONCRETE PLASTER compr.   t2 =  50    mm
STEEL AREA PER 1m OF COVER MESH       As1 = 141    mm²/m
DISTANCE FROM COMPR.FIBER TO STEEL     d1 = 168    mm


DESIGN STRENGTHS OF 3D ELEMENTS ACCORDING TO ACI 318-95
=======================================================
FLEXURE STRENGTH OF 3D SLABS           Mu =   9.9  kNm/m
SHEAR STRENGTH OF 3D SLABS             Vu =  24.8  kN/m
SHEAR STRENGTH OF 3D WALLS (INPLANE)  Vwu = 143.5  kN/m


DESIGN OF 3D WALLS UNDER COMPRESSION
(Moment Magnification Method, Chapter 10.11.5 )
===============================================
UNSUPPORTED LENGTH OF COMPR. MEMBER (m)      lu =   2.8
RATIO OF MAX.AXIAL D.L. TO MAX.AXIAL L.L.    Bd =   0.7
```

MOMENT MAGNIFICATION FACTOR delta	FACTORED AXIAL LOAD Pu (kN/m)	ACTUAL ALLOWED MOMENT Mmax (kNm/m)	ECCENTRICITY e (mm)
1.103	87	8.5	98
1.143	116	10.1	88
1.185	145	11.6	80
1.230	174	12.9	74
1.279	203	14.1	70
1.333	232	15.1	65
1.390	261	15.9	61
1.453	289	16.6	57
1.522	318	17.2	54
1.598	347	17.5	50
1.682	376	17.7	47
1.775	405	17.8	44
1.880	434	17.9	41
1.997	463	16.8	36
2.129	492	15.0	30
2.281	521	13.3	25

4.6. Diagrams according to DIN 1045

The following section deals with the admissible vertical loads for walls according to the method of approximation of DIN 1045, chapter 17.9. All diagrams are based on the following scheme:

safety factor	$\nu = 3.0$
concrete grade	$fc = 10.5$ N/mm² (= B15)

The following values are required to determine the admissible vertical load:

$\lambda = lg_e/r$	i.e.	lg_e = effective length of compression member r = radius of gyration
$e = M/N$	i.e.	M = bending moment N = vertical force

The point of intersection between the diagram curve for the eccentricity *e* and slenderness λ results in force *F*. For other concrete grades the admissible wall load results in

$$F' = \frac{F \cdot fc}{10.5}$$

i.e. F .... value of diagram

Example:

B15 (fc = 10.5 N/mm²) safety factor = 3.0
wall height = 2.80 m effective length $lg_e = 1.0 \times$ height = 2.80 m
thickness of wall = 50 + 100 + 50 mm = 200 mm (2 × 50 mm concrete; 100 mm EPS)
internal wall (unintended eccentricity $e_1 = 30$ mm)

radius of gyration r = 76.4 mm; $\lambda = 2800 / 76.4 = 36.7$

loads : $F = 120$ kN/m
$M = 3.00$ kNm/m
$e_0 = M / N = 25$ mm

Total eccentricity results in $e = e_0 + e_1 = 30 + 25$ mm = 55 mm.
According to the diagram the maximum admissible vertical force is approx. 145 kN/m.

$$F_{adm} = 145 \text{ kN/m} > F_{existing}$$

As a consequence, the wall meets the safety requirements.

4.6.1. Wall thickness = 40 + 50 + 40 mm

radius of gyration r = 46.5 mm, maximum eccentricity e_{MAX} = 45 mm

VERTICAL LOADS
(fc = 10.5 N/mm²; safety factor = 3.0)

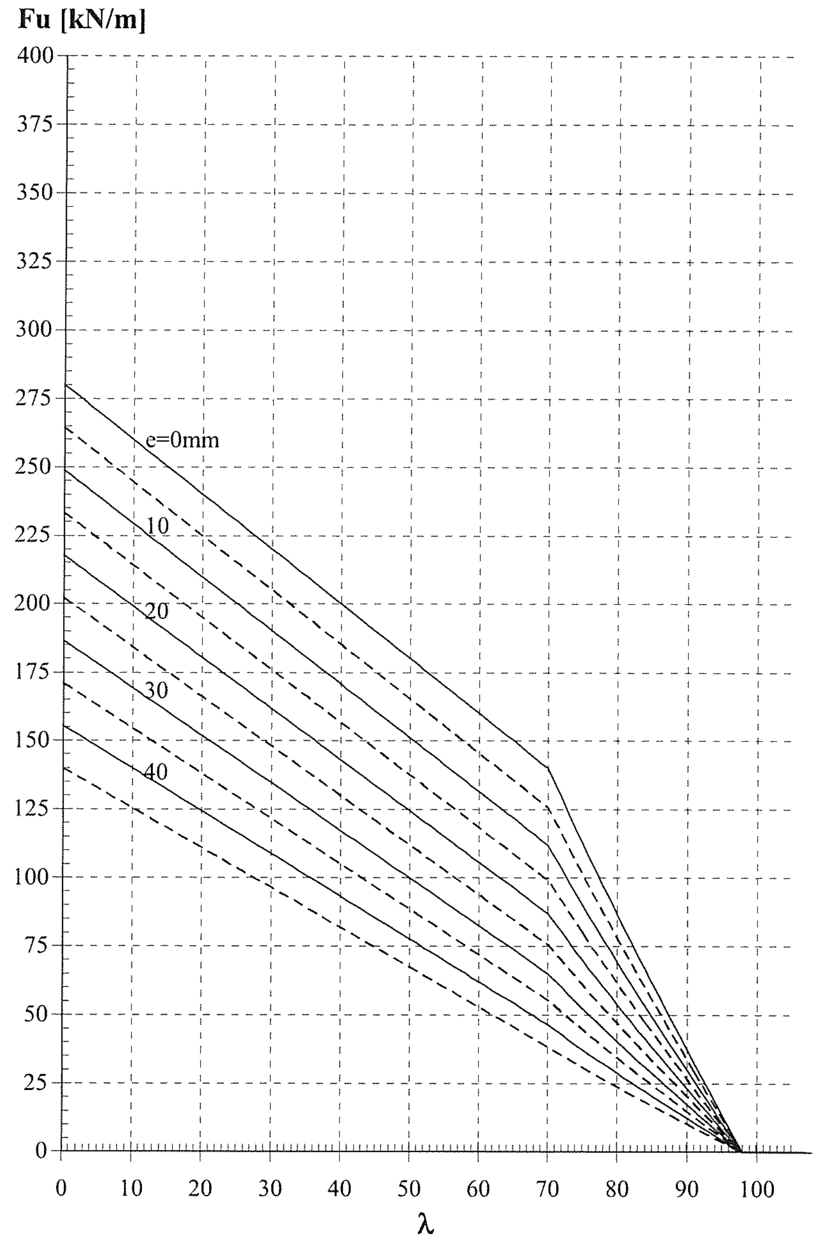


diagram 4.6.1.a Admissible vertical load (40 + 50 + 40 mm)

4.6.2. Wall thickness = 50 + 50 + 50 mm

radius of gyration r = 52.0 mm, maximum eccentricity e_{MAX} = 50 mm

VERTICAL LOADS
(fc = 10.5 N/mm²; safety factor = 3.0)

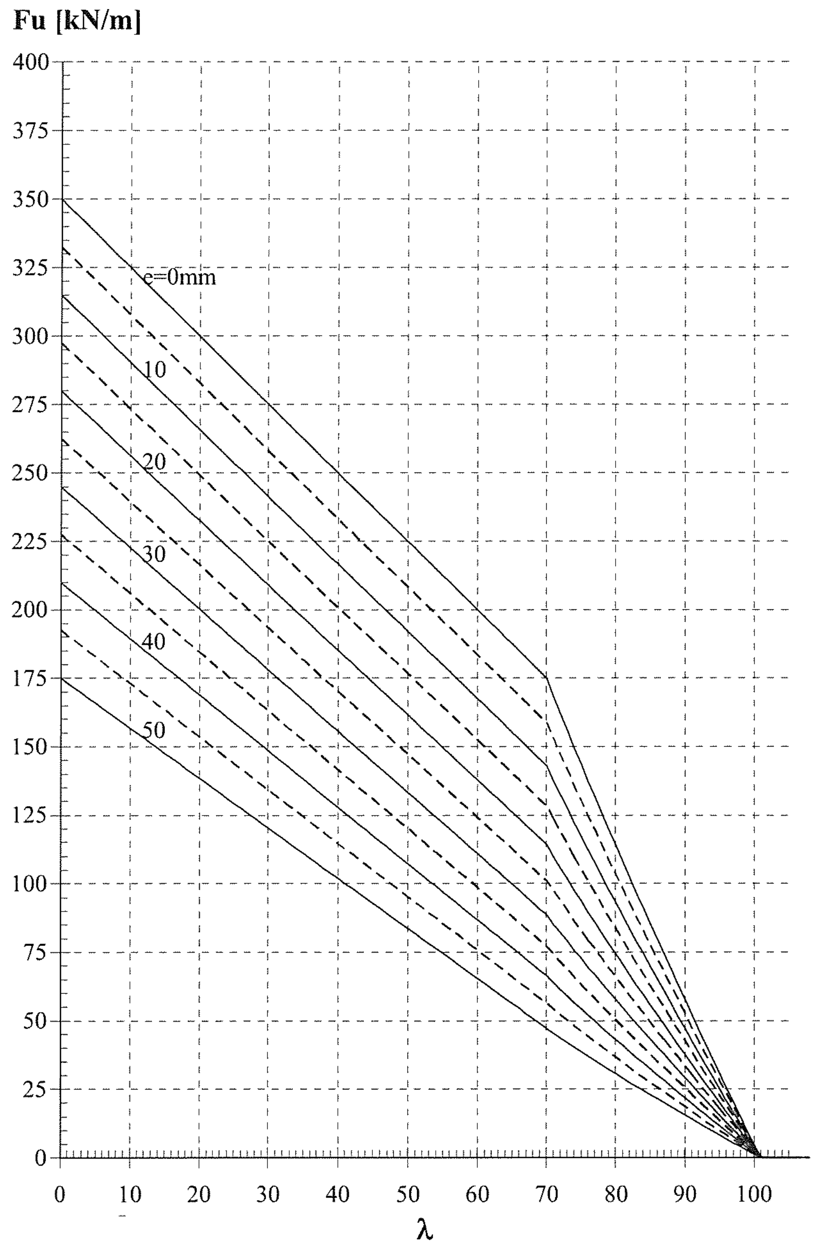


diagram 4.6.2.a Admissible vertical load (50 + 50 + 50 mm)

4.6.3. Wall thickness = 40 + 50 + 90 mm

radius of gyration r = 57.7 mm, maximum eccentricity e_{MAX} = 35 mm

VERTICAL LOADS
(fc = 10.5 N/mm²; safety factor = 3.0)

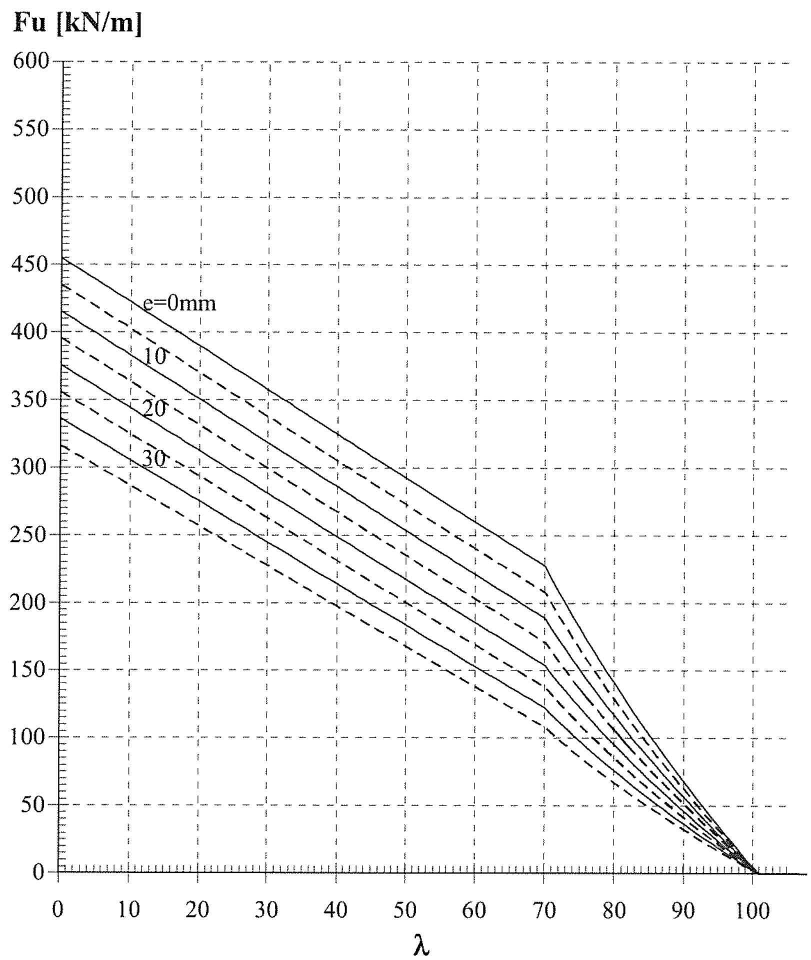


diagram 4.6.3.a Admissible vertical load (40 + 50 + 90 mm)

4.6.4. Wall thickness = 50 + 50 + 100 mm

radius of gyration r = 64.0 mm, maximum eccentricity e_{MAX} = 42 mm

VERTICAL LOADS
(fc = 10.5 N/mm²; safety factor = 3.0)

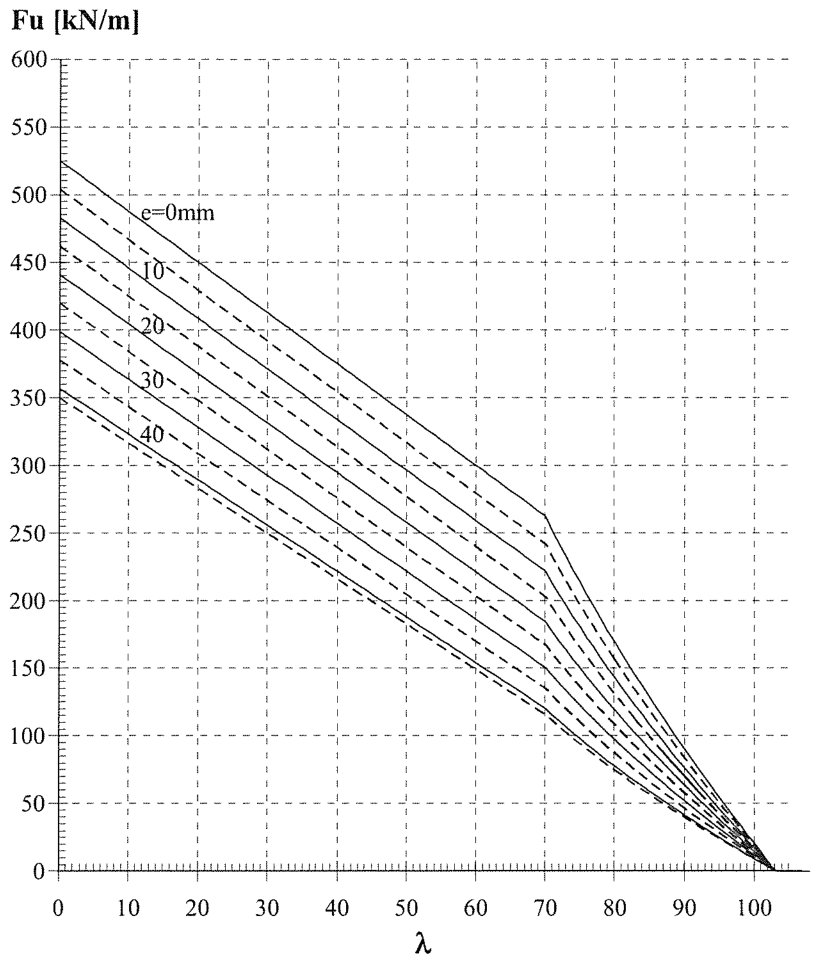


diagram 4.6.4.a Admissible vertical load (50 + 50 + 100 mm)

4.6.5. Wall thickness = 40 + 100 + 40 mm

radius of gyration r = 70.9 mm, maximum eccentricity e_{MAX} = 70 mm

VERTICAL LOADS
(fc = 10.5 N/mm²; safety factor = 3.0)

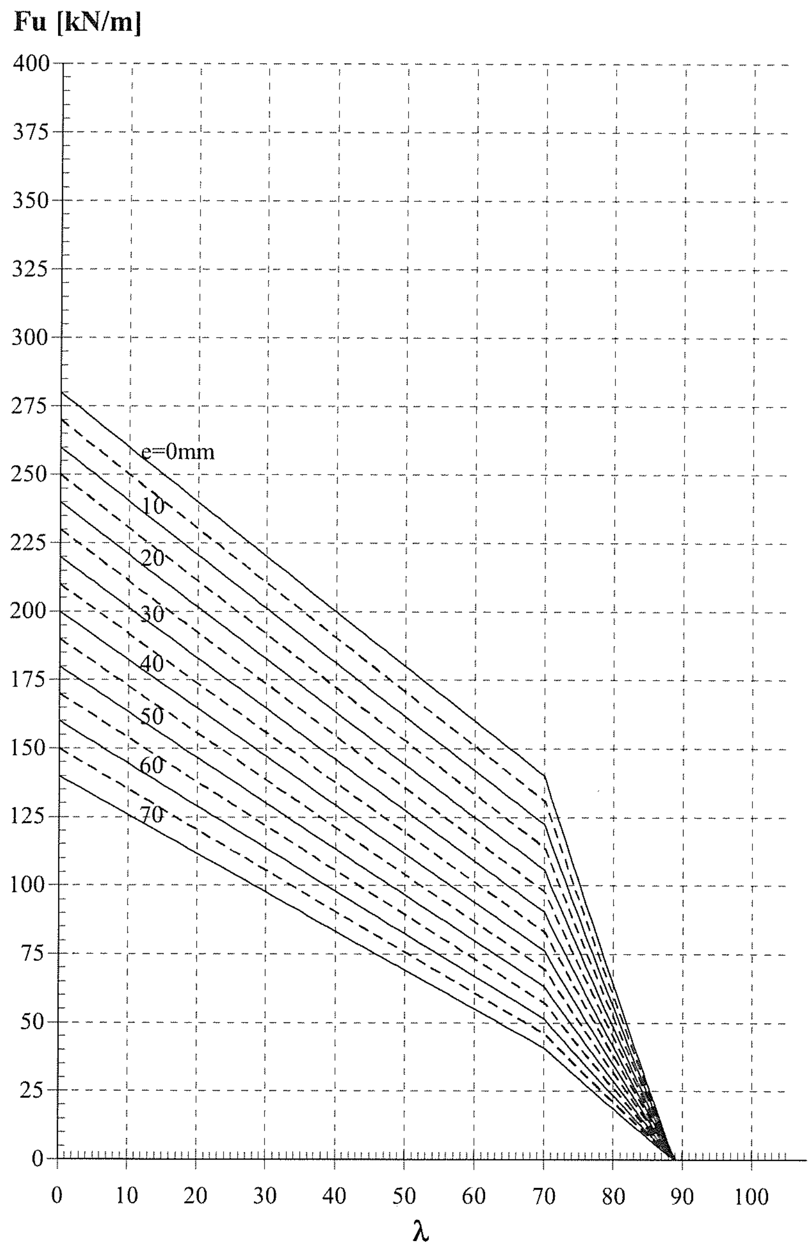


diagram 4.6.5.a Admissible vertical load (40 + 100 + 40 mm)

4.6.6. Wall thickness = 50 + 100 + 50 mm

radius of gyration r = 76.4 mm, maximum eccentricity e_{MAX} = 75 mm

VERTICAL LOADS
(fc = 10.5 N/mm²; safety factor = 3.0)

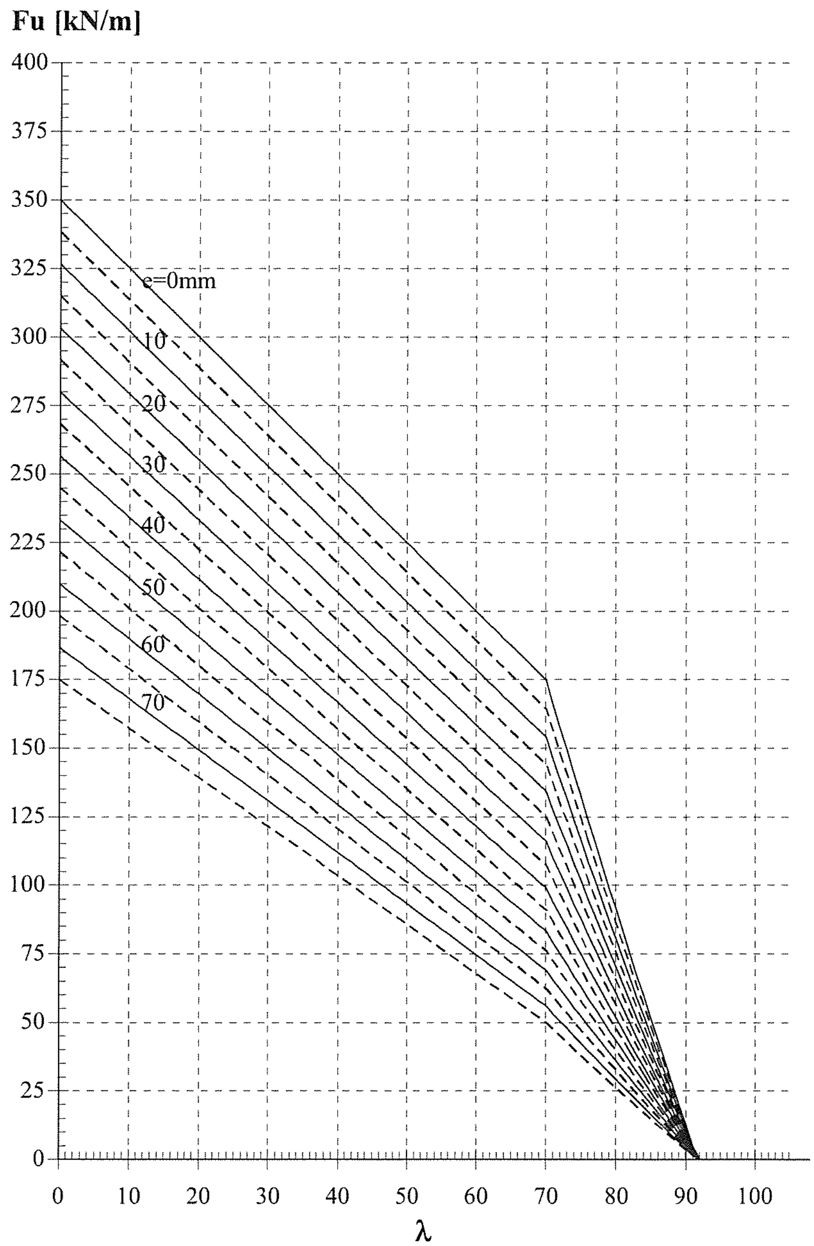


diagram 4.6.6.a Admissible vertical load (50 + 100 + 50 mm)

4.7. Design Tables according to DIN

The following abridged print-out refers to wall tables with an eccentricity of 30 mm. The concrete with a specified compression strength of 14.5 N/mm² doesn't refer to DIN but it is widely used. It corresponds to a concrete grade of B20.

concrete-grade [N/mm²]	**admissible axial load P [kN/m] for 3D walls effective length of wall [m]**													
	2.60	2.80	3.00	3.20	3.40	3.60	3.80	4.00	4.20	4.40	4.60	4.80	5.00	5.20
10.5	90	82	75	67	57	46	36	26	17	7	0	0	0	0
14.5	124	113	103	93	79	64	50	36	23	10	0	0	0	0
17.5	149	137	124	112	95	77	60	44	28	12	0	0	0	0

table 4.7.a 40 mm concrete + 50 mm EPS + 40 mm concrete

concrete-grade [N/mm²]	**admissible axial load P [kN/m] for 3D walls effective length of wall [m]**													
	2.60	2.80	3.00	3.20	3.40	3.60	3.80	4.00	4.20	4.40	4.60	4.80	5.00	5.20
10.5	133	125	116	108	99	90	79	67	56	45	34	23	13	3
14.5	184	172	160	149	137	125	109	93	77	61	47	32	18	4
17.5	222	208	194	179	165	151	131	112	93	74	56	39	21	4

table 4.7.b 50 mm concrete + 50 mm EPS + 50 mm concrete

concrete-grade [N/mm²]	**admissible axial load P [kN/m] for 3D walls effective length of wall [m]**													
	2.60	2.80	3.00	3.20	3.40	3.60	3.80	4.00	4.20	4.40	4.60	4.80	5.00	5.20
10.5	152	147	142	136	131	126	121	116	110	105	100	95	88	74
14.5	210	203	196	188	181	174	167	160	152	145	138	131	121	102
17.5	254	245	236	227	219	210	201	193	184	175	166	158	146	123

table 4.7.c 40 mm concrete + 100 mm EPS + 40 mm concrete

concrete-grade [N/mm²]	**admissible axial load P [kN/m] for 3D walls effective length of wall [m]**													
	2.60	2.80	3.00	3.20	3.40	3.60	3.80	4.00	4.20	4.40	4.60	4.80	5.00	5.20
10.5	200	194	188	182	176	170	163	157	151	145	139	133	127	120
14.5	277	268	260	251	243	234	226	217	209	200	192	183	175	166
17.5	334	324	313	303	293	283	272	262	252	242	231	221	211	201

table 4.7.d 50 mm concrete + 100 mm EPS + 50 mm concrete

5. 3D Beams and Deep Beams

NOTATION

a .... length of span of a bracket
α .... angle of inclination
a_S .... area of reinforcement per meter
A_S ...... area of reinforcement
d......... effective depth
d_P .... height of panel in a beam
fc........ specified compressive strength of concrete
fy .... specified yield strength of steel
lg........ length of span
M_{MAX} max. moment under service load
q......... uniformly distributed load
σ_I .... diagonal tensile stress acc. to ÖNORM
s .... depth of the support of a deep beam
τ......... shear stress under service load
$\tau_{012}/_{03}$ limits of shear stress acc. to DIN
t_1 .... thickness of external concrete shell (tension edge)
t_2 .... thickness of internal concrete shell (compression edge)
V_{MAX} . max. shear force under service load
x......... depth of compression zone of flexural members
z......... lever arm of internal forces

Basics

Especially in case of wall openings 3D wall elements must serve as beams, as well. In this context, a subdivision of beams into 3 categories is possible:

1. The 3D panel can transfer loads without the use of additional reinforcement.
2. The 3D panel requires additional rebars.
3. A conventional reinforced concrete element has to be used instead of the 3D panel.

The following chapters shall deal with the load-bearing capacity of slender beams with or without an additional reinforcement, and the use of deep beams made from 3D elements.

5.1. Beams

Beams made from 3D panels are to be designed just as conventional reinforced concrete beams. However, on account of practical aspects some restrictions apply to the arrangement of shear reinforcement. Owing to the small thickness of the concrete layers the placing of shear reinforcement in vertical and horizontal direction is hardly possible since this reinforcement would not be sufficiently covered by concrete. The same reason restricts also the use of inclined shear reinforcement. Therefore it is recommended to consider shear strength as limiting value for the use of 3D panels in beams.

Another restriction for the use of 3D beams is the necessity of additional flexure reinforcement. Owing to the fact that reinforcement in the 3D panel is distributed over the entire depth, the design differs from the conventional design of flexure.

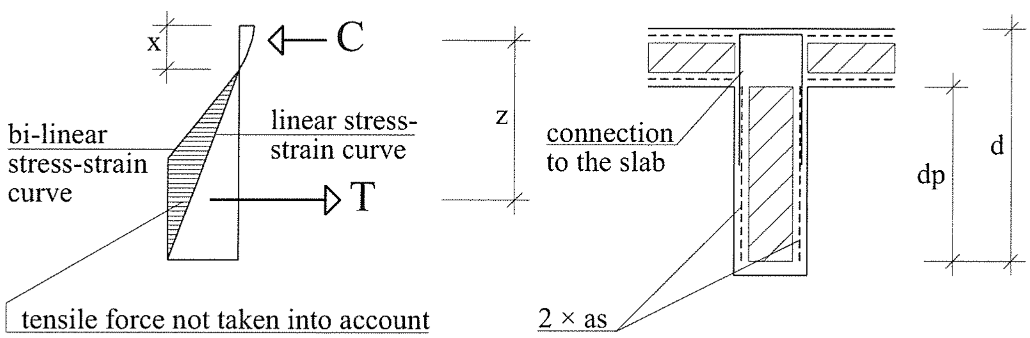


figure 5.1.a Beam model

As the basic reinforcement of the panel is very small, the concrete's compressive strain will always be well below 0.5 ‰. For the sake of simplicity, the stress-strain curve of steel is assumed to be linear. Therefore there are still some safety reserves and a check calculation of the diagonal tensile stress can be omitted.

For common cross sections with a relatively small panel reinforcement, the following approximation can be used:

x .... maximum 0.10 d
z .... approx. 2/3 d

However, these values are only applicable if the panel extends almost over the entire tensile zone ($d_P \approx d - x$). The value of x decreases with an increasing width of the compression chord and with an increasing concrete strength. The value of 0.10 d applies to a width of 20 cm and a concrete strength of 10.5 N/mm² as well as to the common panel reinforcement (2 × 1.41 cm²/m and ST500). As a rule, these values represent the worst case.

The table of the moment coefficients has been made according to the following criteria:

- linear stress-strain curve of steel
- concrete quality = B15 (fc = 10.5 N/mm²)
- width of compression chord = 20 cm
- global safety factor = 1.75

According to this table, the admissible moment results in

$$M = 2 \cdot as \cdot fy \cdot d^2 \cdot me / 1.75 = 80.6 \text{ kN/m} \cdot d^2 \cdot me$$

dp/d	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1
me	0.30	0.30	0.30	0.29	0.27	0.25	0.21	0.16	0.09

table 5.1.a Moment coefficients me

A ratio of dp/d > 0.9 has to be treated as equivalent to dp/d = 0.9. Intermediate values can be interpolated linearly. Table 5.1.b shows the admissible moments without additional reinforcement for lintel depths between 30 and 100 cm. Depth d is applicable as per figure 5.1.a and refers to the effective depth including an assumed floor slab thickness of 20 cm. In case of a smaller floor slab thickness, the results will be on the safe side.

d [cm]	30	40	50	60	70	80	90	100
M [kNm]	1.60	3.48	5.84	8.70	11.84	15.47	19.58	24.17

table 5.1.b Admissible moments for 3D beams without additional reinforcement [kNm]

If the admissible moment is exceeded, additional reinforcement will have to be used. However, this results in a modification of the internal forces in such a way that a smaller load bearing capacity of the panel reinforcement has to be assumed (extension of the compressive zone = restriction of the tensile zone). Our recommendation for compressive strain of a 3D slab (2 ‰) does not apply to beams. Dimensioning can be done according to DIN with the following values:

- Max. compressive strain of concrete 3.5 ‰
- Max. steel strain 5.0 ‰

In the extreme case the tensile zone is reduced to approx. 60 % of the effective depth of the beam. Hence the values for dp/d > 0.6 have to be treated as equivalent to dp/d = 0.6. The moment still to be considered can be assumed by approximation to be about 2/3 of the moment according to tables 5.1.a and 5.1.b. However, the flexural resistance may only be considered if the shear stress is within shear range 1 according to DIN 1045 ($\tau_0 < \tau_{012}$). That means a maximum shear stress of 0.50 N/mm² for B15 and of 0.75 N/mm² for B25. In case of bigger shear stress the panel reinforcement is required to cover shear stress.

Example : Beam with a total depth of 50 cm
Given moment: 7.00 kNm
The moment capacity of the panel is 2/3 · 5.84 = 3.90 kNm
The additional reinforcement has to be designed for 7.00-3.90 = 2.10 kNm

Owing to the small moment capacity, the use of 3D beams without additional reinforcement is restricted to door and window lintels with a convenient ratio between depth and length. However, if this ratio exceeds the values given in table 5.1.c, the lintel can no longer be considered a slender beam. In this case we are dealing with a deep beam as per section 5.2.

The moment capacity of a 3D lintel without additional reinforcement is increased only insignificantly if thee limiting values of table 5.2.a or 5.2.b have been exceeded. In particular, this is due to the fact that the depth of the tensile zone in deep flexural members is disproportionately small. Very large cracks form in deep beams with cracked cross sections. Therefore deep beams are designed on the assumption of uncracked cross sections. Thus it is possible to calculate the load-bearing capacity of the 3D beam without additional reinforcement with a reduced depth according to the tables 5.1.a and 5.1.b:

simple beam	$d_{MAX} = 0.5 \times$ length
continuous beam (edge)	$d_{MAX} = 0.4 \times$ length
continuous beam (inside)	$d_{MAX} = 0.3 \times$ length
cantilever beam	$d_{MAX} = 1.0 \times$ length

table 5.1.c Recommended maximum design depths of slender 3D beams

5.2. Deep Beams

According to DIN 1045, slender beams become deep beams if d/l_0 exceeds the value 0.5. In this case *d* refers to the effective depth of the deep beam and l_0 to the distance between the zero points of moments calculated according to the traditional structural analysis. By analogy this results in the following definitions of deep beams:

simple beam	$d \geq 0.5 \times$ length
continuous beam (edge)	$d \geq 0.4 \times$ length
continuous beam (inside)	$d \geq 0.3 \times$ length
cantilever beam	$d \geq 1.0 \times$ length

table 5.2.a Limits for deep beams according to DIN

The following section shows a simple and safe design of deep beams. In general these data base on the regulations of DIN 1045. The American ACI regulations concerning the limits of slender beams differ significantly from these values. The ACI code gives two definitions for deep flexural members. For flexure the limit of depth/length is 0.4 for continuous spans or 0.8 for simple spans. For shear the limit of the ratio depth/length is 0.2.

kind of span	flexure	shear
simple span	depth / length ≥ 0.8	depth / length ≥ 0.2
continuous span	depth / length ≥ 0.4	depth / length ≥ 0.2

table 5.2.b Limits for deep beams according to ACI

5.2.1. Flexure

The moments are calculated according to the rules of conventional structural analysis. The subsequent calculation of the tensile reinforcement must be made by taking into account lever arms having considerably smaller coefficients than those for slender beams. In addition, a maximum steel stress of 41.2 kN/cm² can be taken into consideration. In particular, this becomes necessary since, in case of a deep beam, the deformations of a cracked cross section would lead to excessively wide cracks and, as a consequence, to a considerably restricted usability. Therefore tensile and compressive strains, and thus the depth of the tensile zone and the lever arm of internal forces, always base on an uncracked cross section. A separate check calculation of compressive stresses can be omitted for deep beams. Table 5.2.1.a shows the lever arms of internal forces for different types of deep beams. These values are valid for positive and negative moments. The lever arms calculated in such a way remain constant from an effective depth of d = 1.0 lg or d = 2.0 lg (for cantilevers). The data of line 2 (continuous beam - edge) apply also to the negative moment above the first internal support.

simple beam	$0.5 < d/lg < 1.0$ $d/lg \geq 1.0$	$z = 0.3\ d\ (3.0 - d/lg)$ $z = 0.60\ lg$
continuous beam (edge)	$0.4 < d/lg < 1.0$ $d/lg \geq 1.0$	$z = 0.5\ d\ (1.9 - d/lg)$ $z = 0.45\ lg$
continuous beam (inside)	$0.3 < d/lg < 1.0$ $d/lg \geq 1.0$	$z = 0.5\ d\ (1.8 - d/lg)$ $z = 0.40\ lg$
cantilever beam	$1.0 < d/lg < 2.0$ $d/lg \geq 2.0$	$z = 0.65\ lg + 0.10\ d$ $z = 0.85\ lg$

table 5.2.1.a Lever arm of internal forces

Thus the required additional reinforcement is

$$As = \frac{1.75 \cdot M_{MAX}}{fy \cdot z}$$

i.e. 1.75.... safety coefficient (DIN)
fy .... specified yield strength of steel $\leq$ 41.2 kN/cm²
z......... lever arm according to table 5.2.1.a

Basically, the use of additional reinforcement for deep beams is always necessary. As a special exception, additional reinforcement can be omitted only in case of secondary components up to a span of 2.00 m and in case of panel reinforcements that meet the requirements of figure 5.2.3.a.

In case of designing according to ACI it is possible to use also the lever arm according to table 5.2.1.a. However, the ultimate moment has to be used and a strength reduction factor of 0.90 has to be taken for fy.

5.2.2. Shear Design

The shear forces are calculated according to the rules of the conventional structural analysis. With regard to this calculation only the reaction force at the first support of a continuous beam needs to be increased by 15 %. Shear resistance at the support is largely determined by steep diagonal compressive stresses. Thus a calculation of shear reinforcement just like in case of beams is not necessary. The diagonal tensile stresses in the support area are covered by the minimum mesh reinforcement. This minimum mesh reinforcement should be on each side at least 0.05 % of the concrete cross section or 1.5 cm²/m and corresponds therefore approximately to the cover mesh reinforcement of a 3D panel. However, part of the literature demands an even higher reinforcement of this basic mesh (up to 0.15 % per side). According to this higher value, the reinforcement area of the basic mesh for a wall with 2 × 50 mm concrete results in 1.5 cm²/m. This value corresponds almost to the cover mesh of the panel.

If, however, the required concrete cross section that needs to resist the compressive forces exceeds 50 mm per side, the 3D wall will have to be replaced by a conventional reinforced concrete wall.

Stress p at the support of a deep beam may be calculated according to the following formula:

$$p = \frac{V_{MAX}}{(t_1 + t_2) \cdot s} \leq \frac{fc}{2.1}$$

i.e. s......... depth of the support; s must be chosen in such a way that it does not exceed 1/5 of the smallest span adjacent to the support.
2.1...... safety factor (concrete failure)

In addition, some standards demand the check calculation of the diagonal tensile stresses. These tensile stresses must not exceed the maximum value of the admissible shear stress of a reinforced cross section (= τ_{03} acc. do DIN). By analogy to the beam, the diagonal stress for a uniformly distributed load is:

$$\sigma_1 = 1.2 \cdot \frac{V_{MAX}}{(t_1 + t_2) \cdot d} \qquad \text{whereby } d \leq lg \quad \text{(ÖNORM)}$$

In a calculation based on the American ACI-318 standard, the shear strength can be determined by the formulae of chapter 11.8. Mind that in a calculation according to ACI the shear reinforcement must not be staggered.

5.2.3. Placing of Reinforcement

In parts the arrangement of reinforcement in deep beams differs considerably from the arrangement in conventional beams. In addition to the flexural reinforcement, a minimum reinforcing mesh as per chapter 5.2.2 is yet required. For the arrangement of the reinforcement see also figures 5.2.3.a and 5.2.3.b.

Minimum reinforcing mesh:

- Overlap all panel splices with splice mesh. The overlapping length in the tension area is at least 4 times the pitch of the mesh wires. Therefore in the bottom part of a deep beam splice mesh with a width of 45 cm has to be used.
- Loads suspended at bottom must be secured by means of a suspension reinforcement. This reinforcement has to extend up to lg/2 from the lower edge and has to be completely anchored. The wall's dead weight up to this height is considered as suspended at bottom, as well.

Bottom reinforcement:

- Distribute the complete bottom reinforcement over a depth of 0.1 lg.
- In the area between 0.1 lg and 0.3 lg, additional 50 % of reinforcement at midspan have to be placed. Thereby the area of the minimum mesh reinforcement may be included.
- Place the reinforcement without staggering over the entire length and anchor them completely at the support. At the end support it is recommended to perform this anchoring by means of horizontal but not vertical U-shaped stirrups. A lap splice above an inside support has to be designed with full development length.

Top reinforcement:

- Arrange the complete top reinforcement in the area between 0.3 lg and 0.7 lg or at the upper edge of the wall.
- In continuous deep members 50 % of the top reinforcement shall be placed over the entire length of span.
- Arrange additional 30 % of the top reinforcement in the area between 0.1 lg and 0.3 lg. Thereby the area of the minimum reinforcing mesh may be included.

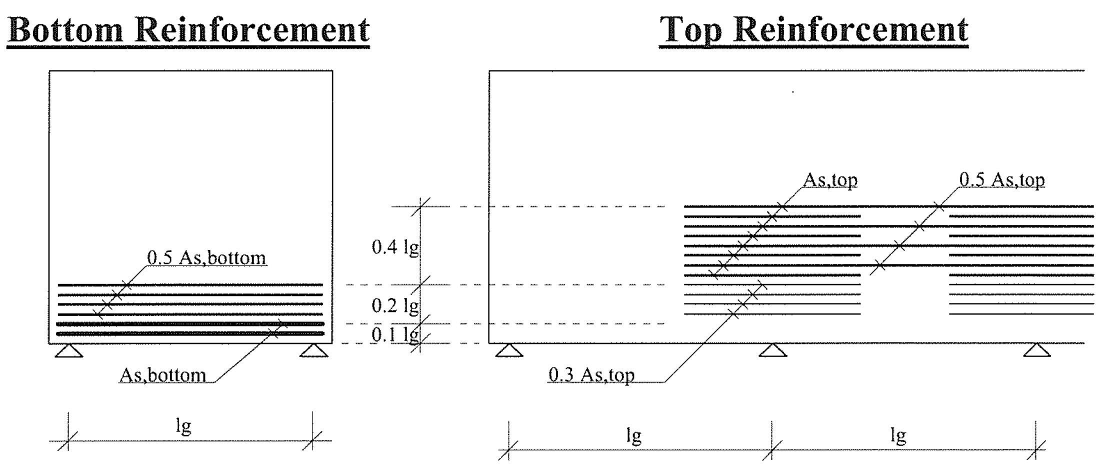


figure 5.2.3.a Arrangement of reinforcement in deep beams

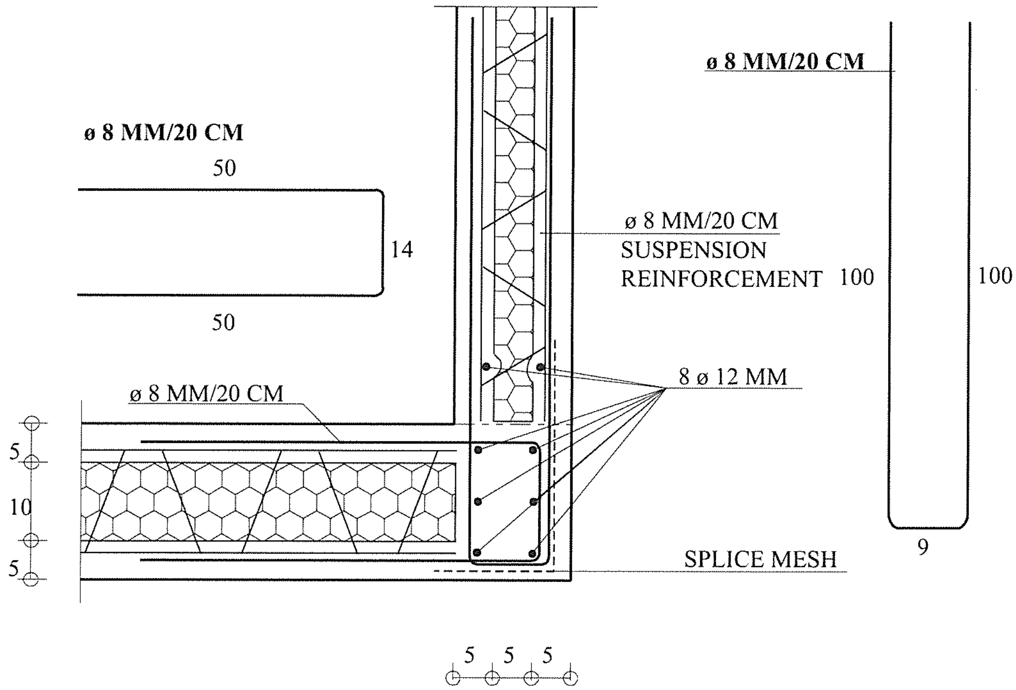
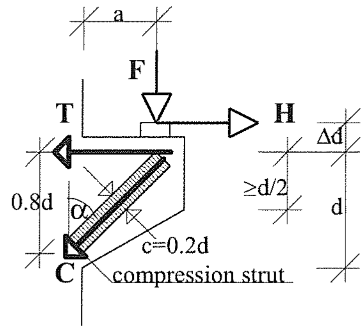


figure 5.2.3.b Typical bottom reinforcement of a deep beam

5.3. Brackets and Corbels

In principle, brackets and corbels are cantilever deep beams. In most cases the occurring load is a concentrated load near the cantilever end. Due to this non-uniformly distributed load, there are significant differences compared with the uniformly loaded cantilever deep beam. The ratio between depth and length is within the range of $2 \geq d/a \geq 1$. Brackets higher than 2 a are to be treated as equivalent to d = 2 a. Brackets are calculated as truss systems with tension ties and compression bars. Therefore a check calculation of shear forces can be omitted as the shear force is transferred directly to the compression bar. In the following, the DIN standard is taken into consideration. The results coincide very well with those of the calculations according to ACI. For more details see chapter 11.9 of ACI-318.



F = vertical force
H = horizontal force
T = tensile force in the reinforcement
C = compression force in concrete

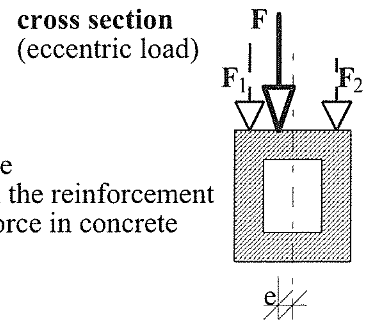


figure 5.3.a Forces within a bracket

The safety of a bracket is ensured if the following prerequisites are fulfilled.

The reinforcement of the tension tie has to be designed according to tensile force T. The safety coefficient is 1.75 (steel failure). At the bracket's end, this reinforcement has to be anchored by horizontal loops. For determination of the effective depth, take into consideration that the tensile reinforcement often consists of several layers. T results from:

$$T = \frac{F \cdot a}{0.8 \cdot d} + H \cdot \left(1 + \frac{\Delta d}{0.8 \cdot d}\right)$$

The ideal strut of the inner truss (compression strut) is assumed with $(t_1+t_2)\cdot c$ whereby (t_1+t_2) is the width of the two concrete shells of the 3D wall. The value of *c* is equivalent to 0.2 d. The maximum stress in this strut is to be assumed analogously to the DIN approximation method with 0.95·fc. The safety factor is 2.1 (concrete failure).

$$C = \frac{F \cdot a + H \cdot \Delta d}{0.8 \cdot d \cdot \sin\alpha} \leq \frac{0.19 \cdot (t_1 + t_2) \cdot d \cdot fc}{2.1}$$

$$\text{with } \alpha = \arctan\left(\frac{a}{0.8 \cdot d}\right)$$

In case a load is applied out of centers (figure 5.3.a, right side), the two forces F_1 and F_2 have to be determined in such a way that force F is their resultant. Dimensioning of the bracket will then be carried out for the concrete shell that is subject to the higher load.

It is not recommended to calculate the theoretic shear stress $\tau = F/((t_1+t_2)\cdot z)$. If the maximum shear stress possible (τ_{03}) is fully utilized, the admissible loads resulting thereof can go far beyond the load-bearing capacity of the strut.

6. Deflection

NOTATION

α .... inclination of diagonals
As ...... area of tension reinforcement
As'...... area of compression reinforcement
b......... width of the cross section (mostly 1 meter)
B_I .... flexural rigidity of uncracked cross section
B_I' ...... flexural rigidity of uncracked cross section for long-term deflection
B_{II} ...... flexural rigidity of cracked cross section
B_{II}' ..... flexural rigidity of cracked cross section for long-term deflection
Δ .... deflection
Δ_{DIAG}. deflection due to the diagonals
d......... effective depth of the 3D slab
d_{PAN}... total thickness of the panel up to the outer edge of the cover mesh
E_C ...... modulus of elasticity of concrete
E_S....... modulus of elasticity of steel
fc........ specified compressive strength of concrete
f_R .... modulus of rupture of concrete
I_G .... moment of inertia of uncracked cross section
I_{CR} ..... moment of inertia of cracked cross section
I_E........ effective moment of inertia
K........ curvature in [m^{-1}]
λ......... factor for additional long-term deflection acc. to ACI
lg........ length of span of slab
M .... bending moment
M_{MAX} maximum moment under service load
M_{CR} ... cracking moment
n_{DIAG} . number of diagonals per m^2
q......... uniformly distributed service load
ρ'........ reinforcement ratio for non-prestressed compression reinforcement where As' shall be the value at midspan for simple and continuous spans, and at support for cantilevers
ρ' = As'/bd
V........ shear force at support
W....... section modulus referring to the tension edge
ξ......... time-dependent factor for additional long-term deflection acc. to ACI. It shall be permitted to assume the factor ξ for sustained loads to be equal to 2.0 (5 years or more).
x......... depth of the compression zone
y_t........ distance of centroidal axis to extreme fiber in tension
z......... lever arm of internal forces

Three methods are described to calculate the deflection of 3D slabs. The first method starts from a description of the cross section's curvature. The deflections calculated with this method correspond quite well to the observed behavior of 3D slabs. However, owing to the large extent of calculation according to the curvature method a computer-assisted solution is advisable. The second method meets the ACI regulations and, on the basis of the internal forces and the cracking moment, it allows the determination of a mean rigidity of the cross section. The third method is a mere approximate solution based on ACI regulations, as well. However, in the relevant area this approximation provides for results that are on the safe side. In contrast to the curvature method the approximation method can be used easily.

6.1. The Curvature Method

Like all other methods for the calculation of deflections it starts from the fact that the curvature of a cross section corresponds approximately to the second derivative of the deflection curve. As a consequence, it is necessary to determine the curvature of the cross section over the total length and to apply this curvature as mathematical "load". The "moments" calculated show directly the slab's deformation. However, for a 3D cross section it is necessary to take into account the influence of the diagonal as well. In most cases this amount is much lower than the share of flexural deformation and, therefore, it can be determined with some simplifications without any considerable effects on the final result.

6.1.1. Curvature

The calculation of curvature varies clearly in two fields:

1. curvature of uncracked cross section
2. curvature of cracked cross section

Owing to the assumption that the entire cross section is either cracked or uncracked, it is possible to determine the minimum and maximum limit of deflection to be expected. This section, however, describes a method that calculates deflection directly by using the effective rigidity.

Flexural rigidity of an uncracked cross section results in:

$$B_I = I_G \cdot E_C$$

Rigidity of a cracked cross section depends mainly on the tension reinforcement and the neutral axis:

$$B_{II} = As \cdot z \cdot (d - x) \cdot E_S$$

The values z and x may be taken from flexural design of maximum moment. The determination of these values for every parting point is of interest in case of a computer-assisted calculation only.

6.1.2. Uncracked Cross Section

The curvature of an uncracked cross section can be determined according to the common structural analysis.

$$K = \frac{M}{B_I}$$

By approximation the influence of the reinforcement on the moment of inertia can be neglected. However, for a more accurate calculation it is necessary to take into account their influence in relation to the moduli of elasticity. Yet, the determination of the constant values of concrete reveals some considerable inaccuracies in this case. Both the modulus of elasticity and the modulus of rupture are very rough values only. In the literature especially the data on modulus of rupture vary by up to 25 %. As a consequence, from the very beginning every calculation of deflection can be considered an estimation only. In addition, due to long-term effects, the cracking moment falls by up to 20 %. Table 6.1.2.a shows the characteristic cocncrete values according to DIN.

Concrete grade (W28)	fc [N/mm²]	Modulus of elasticity [N/mm²]	Modulus of rupture [N/mm²]
B 15	10.5	26,000	2.0
B 25	17.5	30,000	2.7
B 35	23.0	34,000	3.2
B 45	27.0	37,000	3.8
B 55	30.0	39,000	4.3

table 6.1.2.a. Characteristic concrete values according to DIN (after 28 days)

Owing to the moment of inertia, the section modulus and the modulus of rupture the cracking moment results directly in:

$$M_{CR} = W \cdot f_R$$

i.e. f_R .... modulus of rupture according to table 6.1.2.a

The cracking moment defines the transition from uncracked state to cracked state. Such a cracking moment applies to the short-term status. These values are valid if concrete is at least 28 days old at the time when the full load is applied for the first time. If the concrete is only 7 days old when the full load is applied for the first time, the modulus of elasticity and the modulus of rupture according to table 6.1.2.a will have to be reduced by 25 %.

6.1.3. Cracked Cross Section

The deformation behavior of a cracked cross section is far more difficult to describe than the one of an uncracked cross section. If the cracking moment is exceeded only insignificantly, a few cracks only will appear, and concrete between the cracks will help substantially to reduce deformation. However, as soon as the moment has grown to a multiple of the cracking moment, the behavior of the member will correspond to a completely cracked cross section. Therefore, the prerequisite for the application of a mixed calculation of curvature is a careful application and after-treatment of the concrete at the bottom side of the slab to keep the number of cracks low.

Curvature of the cracked cross section results in:

$$K = \frac{M_{CR}}{B_I} + \frac{4}{3} \cdot \frac{M - M_{CR}}{B_{II}} \leq \frac{M}{B_{II}}$$

i.e. M..........existing bending moment in the point observed

Owing to the fact that the curvature line calculated according to the above formula looks rather complicated (see figure 6.1.4.c), it can be substituted by "point loads" (so-called "angle weights"). To do so, it is necessary to divide the slab up into a sufficient number of sections (e.g. 10 or 20) to determine afterwards the curvature in every parting point. So, the area below the curve and thus, the theoretical "point loads" in the parting point can be determined by dividing the area in triangles.

$$F = (K_L + 4\,K_M + K_R) \cdot \frac{1}{6} \cdot a$$

i.e. F........ mathematical "force" in the point observed
K_L...... curvature left of the section
K_M..... curvature in the point observed
K_R...... curvature right of the section
a......... length of a section

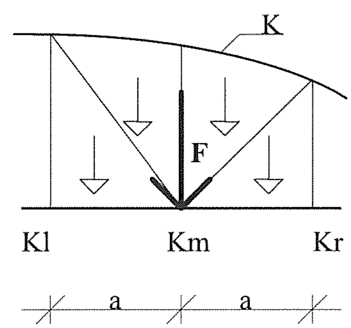


figure 6.1.3.a "Angle weights"

6.1.4. Short-term Deflection

In the next step the "forces" calculated in item 6.1.3 are applied as "load". The "moments" resulting from this load show directly the deflection in any point of the slab.

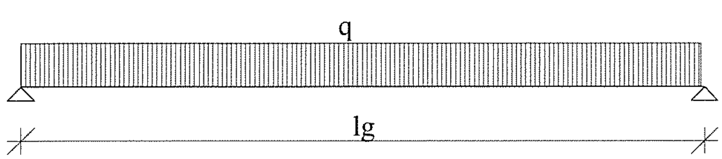


figure 6.1.4.a System drawing for simply supported slab with uniformly distributed load

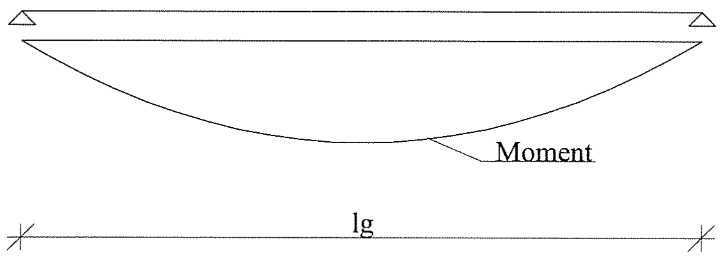


figure 6.1.4.b Moments in a simply supported slab

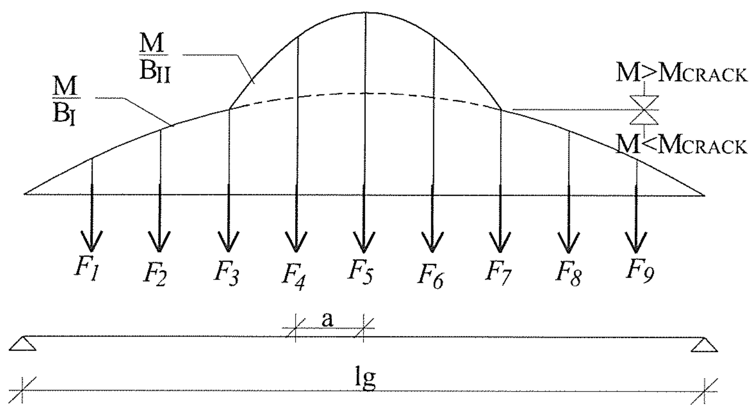


figure 6.1.4.c Curvature and "angle weights"

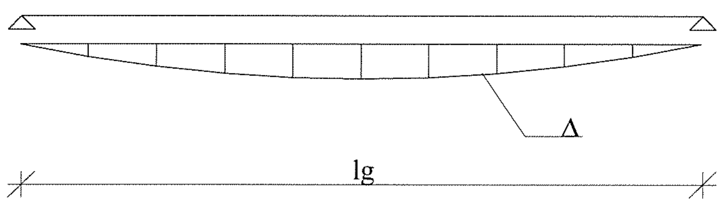


figure 6.1.4.d Short-term deflection calculated with "angle weights"

6.1.5. Long-term Deflection (Creep and Shrinkage)

In general, concrete shows signs of considerable deformation in case of long-term loading. This deformation depends on the relative utilization of concrete, on the time of application of the full load and on climatic conditions (humidity) during hardening. Owing to some elements of uncertainty a very rough estimation only can be made. This results in a reduced flexural rigidity. In this case deflection is calculated separately for sustained load, i.e. the load which is effective over an extended period of time.

It is possible to apply the following factors in case of average conditions on site and thorough after-treatment of concrete (in this simplified method the influence of compression reinforcements on creep deformation is not taken into consideration):

uncracked cross section: $B_I' = 1/3 \cdot B_I$
cracked cross section: $B_{II}' = 2/3 \cdot B_{II}$

Usually these values and the observations are quite the same. Owing to a very cautious assumption of flexural rigidities for long-term observation it is possible to waive the reduction of cracking moment (see 6.1.2). Experience has shown that the live load is not effective over the entire period of time. Therefore, for the calculation of long-term deflection it may be reduced by 40 %. However, in this case the moments are reduced, as well, causing uncracked cross sections to have a disproportionately big influence on deflection. Thus the location where a cracked cross section has to be assumed should be taken from full live load.

6.1.6. Shear Deflection

In contrast to other load-bearing structures shear deformation of the diagonals in a 3D slab is quite important, as well. This deformation depends on the inclination and number of diagonals. Owing to the fact that inclination increases with the number of diagonals shear deformation will be almost identical both at 200 diagonals/m² and at 100 diagonals/m².

For a slab under uniformly distributed load it is possible to calculate deflection as a result of shear deformation according to the following formula:

$$\Delta_{DIAG} = \frac{V \cdot lg}{4 \cdot b \cdot n_{DIAG} \cdot \cos^2\alpha \cdot z^2} \cdot \frac{d_{PAN}}{\sin\alpha \cdot E \cdot A}$$

i.e.
- V............ shear force at the support but not more then twice the admissible shear force for a panel without additional shear reinforcement (see 3.6)
- b............. width of the slab = 1.00 m
- α............ inclination of diagonals
  for 100 or 50 mm EPS the following values apply:
  100 diagonals/m² : α = 65.7° or 54,1°
  200 diagonals/m² : α = 73.3° or 64.3°
- A_{DIAG}..... area of cross section of a diagonal (ø 3.8 mm = 0.113 cm²)

Force in the diagonals and, as a consequence, their deformation was assessed according to horizontal shear force (see chapter 3). Strictly speaking, the above formula applies only to a simply supported slab. Owing to the small shear deformation, however, it may be applied by approximation to all slab systems.

This share of deflection occurs only if we are dealing with a pure 3D slab. Stiffenings made from concrete between the panels (e.g. lattice girders) reduce this share to a minimum so that it can be neglected.

The maximum limit of shear deflection for standard 3D panels (100 mm EPS, 200 diagonals/m²) is approximately 1 mm per 1 m length of span. This means, the shear deflection for a 3D slab with a span of 5.0 m is less or equal to 5 mm.

6.2. Deflection according to ACI 318-89 (Chapter 9.5)

Generally, the calculation according to the American standard ACI can be compared with the method of approximation according to chapter 6.3. This method allows to determine an average value for the effective moment of inertia that can be applied over the entire length of the slab. This effective moment of inertia depends on the moments of inertia in a cracked and non-cracked cross section and on the ratio between the cracking moment and the existing moment. As soon as the short-term calculation is done the influence of creep is assessed.

Example :

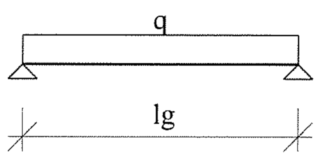


fig 6.2.a Simply supported slab

$$\Delta = \frac{5 \cdot q \cdot lg^4}{384 \cdot E_C \cdot I_E}$$

$$I_E = \left(\frac{M_{CR}}{M_{MAX}}\right)^3 \cdot I_G + \left[1 - \left(\frac{M_{CR}}{M_{MAX}}\right)^3\right] \cdot I_{CR}$$

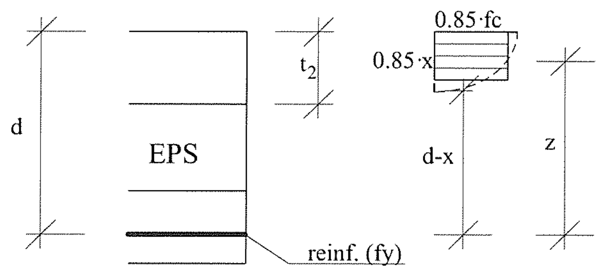


figure 6.2.b Internal forces of a 3D-slab

$$I_{CR} = As \cdot (d-x) \cdot z \cdot (E_S / E_C)$$

$$M_{CR} = \frac{f_R \cdot I_G}{y_t}$$

i.e. f_R .... modulus of rupture of concrete

$f_R = 7.5 \cdot \sqrt{fc}$ fc in [psi]; 1 N/mm² = 145.03 psi

$f_R = 0.623 \cdot \sqrt{fc}$ fc in [N/mm²]

Additional long-term deflection resulting from creep and shrinkage of flexural members shall be determined by multiplying the immediate deflection caused by the sustained load considered, by the factor

$$\lambda = \frac{\xi}{1 + 50\rho'}$$

ξ......... It shall be permitted to assume the factor ξ for sustained loads to be equal to 2.0 (5 years or more).

6.3. Simplified Method

The method described in the following chapter has well proven its worth for the use of standard slabs (EPS thickness = 100 mm, concrete thicknesses 4-7 cm) and has been confirmed by various test results. The general procedure corresponds to the ACI method.

The general issue starts from the assumption of a homogenous cross section. This cross section consists of the two concrete layers and, possibly, of an additional reinforcement. The panel's reinforcement can be neglected for this calculation. However, for the calculation of deflection 1/5 of the calculated moment of inertia only is taken. This reduced moment of inertia is a mean value between a cracked and uncracked concrete section.

$I_E = I_G / 5$

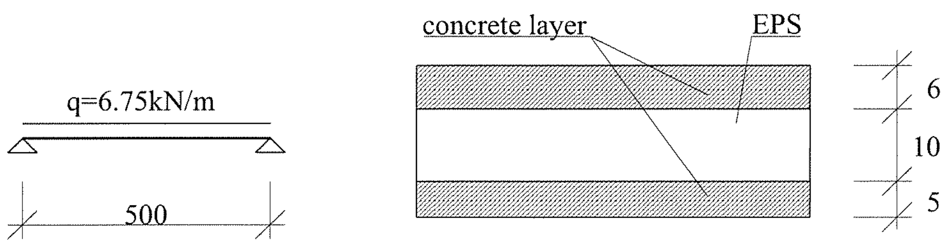


figure 6.3.a System drawing

When neglecting reinforcement the moment of inertia results from table 6.3.a in

$I = 68\ 364\ cm^4/m$
$I_E = I_G / 5 = 13\ 673\ cm^4/m$

According to the general rules of structural analysis deflection can be computed as follows

$$\Delta = \frac{5\ q\ lg^4}{384\ E_C\ I_E}$$

i.e. q......... 6.75 kN/m (= 0.0675 kN/cm)
lg .... 500 cm
E_C...... 3000 kN/cm² (B25)
I_E .... 13 673 cm⁴

Hence deflection is 1.34 cm (= lg/373). Usually the admissible deflections lie between lg/200 and lg/300. In order to reduce deflection in the final state it is possible to design the slab also with a camber.

The tables 6.3.a and 6.3.b show the moments of inertia for slabs with 100 or 50 mm EPS and a top concrete layer of 50 to 90 mm. All values refer to a width of 1.00 m. The panel's reinforcement was neglected.

EPS 100	top concrete layer [mm]				
bottom concrete layer	50	60	70	80	90
40 mm	48297	56333	64546	73067	82001
50 mm	58333	68364	78567	89078	100010
60 mm	68364	80400	92616	105152	118125

table 6.3.a Moments of inertia I_G for EPS-100 [cm^4/m]

EPS 50	top concrete layer [mm]				
bottom concrete layer	50	60	70	80	90
40 mm	21631	26333	31455	37067	43231
50 mm	27083	32910	39192	46001	53402
60 mm	32910	39900	47385	55438	64125

table 6.3.b Moments of inertia I_G for EPS-50 [cm^4/m]

6.4. Applicability of Various Methods

The above sections described three methods:

- Curvature method with or without structural function of concrete in cracked state
- ACI method
- Approximation method

The curvature method including the structural function of concrete after exceeding the cracking moment and the ACI method can be used generally for design and matches quite well the actual deformation observed. The calculation that neglects the influence of concrete after exceeding the cracking moment is recommended especially as a check calculation if, on account of unfavorable influences (i.e. insufficient after-treatment), the number of cracks is considerably higher than expected. In order to estimate the maximum deflection of floor slabs with average spans (3 - 5 m) under unfavorable circumstances it is possible to use the method of approximation since this method requires a mean value between the cracked and uncracked cross section. In case of wider spans or a particularly high number of cracks it is recommended to apply the check calculation.

6.5. Limitation of Deflection

Admissible deflection depends on the corresponding standards and codes and lies between 1/300 and 1/200 of the length of span. If the calculated deflection lies above this admissible value, it is possible to reduce deformation by a couple of countermeasures:

- The slab may be cambered at midspan. This camber should not exceed the admissible deflection and, for the calculation, its effect has to be considered 50 % only.
- The slab should be shored beyond the usual period of time. Basically, the central row of props must not be removed before the concrete has achieved full strength, i.e. at least some 3 weeks after the top concrete layer of the slab has been finished. A prolongation of this period of time, however, results in a well perceptible improvement of the slab's creep behavior. If there are more than 2 rows of supports, it is possible to remove the props at the edge already after 1 - 2 days only if the reinforcement at the support is covered by at least one concrete layer. Otherwise a noticeable deformation may occur already in this stage on account of the lacking structural function of this reinforcement.
- In general, the after-treatment of concrete at the bottom side of the slab has to be done very carefully. Especially when using shotcrete there is a higher risk of shrinkage cracks, exerting thus a considerable negative influence on deflection. It is only the achieved full tensile strength at the bottom side of the slab that keeps deformation low. Therefore, when using shotcrete, the theoretical concrete grade should be assumed one degree lower than the one of the actually used concrete.
- In case of a large span or high loads the shear strength of the 3D panel is not sufficient. It is recommended to use continuous shear girders over the entire length of span instead of shear reinforcements in the area of the supports. This case given, shear deflection can be neglected.

6.6. Deflection Diagrams

For the most frequent cases the following diagrams were made out according to the curvature method. Assumptions:

- Concrete grade is 25 N/mm² (fc = 17.5 N/mm², modulus of elasticity = 30,000 N/mm², f_R = 2.7 N/mm²). When using shotcrete for the bottom side of the slab it is advised to increase concrete grade by one degree (see section 6.5.).
- Steel grade of the cover mesh is the same as the one of the additional reinforcement and amounts to 500 N/mm². The use of additional reinforcement of higher steel grade increases deflection only slightly. The area of reinforcement was assessed exactly, and with this theoretical value it was taken into consideration for the calculation of deflection. The total bottom reinforcement was used over the entire length. The top reinforcement in the area of restrained edges was assumed to be unstaggered up to the quarter point of the slab. Basic reinforcement of the panel on both sides is 1.41 cm²/m.

- The shear deflection was assessed for a panel with 100 diagonals/m² since shear deformation in this case is slightly higher than for a panel with 200 diagonals/m² on account of the lower number of diagonals. Although additional shear reinforcement become necessary in some cases, the shear deflection was always taken into consideration to the full.
- Total load is 4.00 to 8.00 kN/m² with a step of 0.50 kN/m². For long-term deflection the live load of 2.00 kN/m² was reduced by 40 %.

If the concrete grade used is higher, a reduced load can be assumed in the diagram. This theoretical load can be converted according to the moduli of elasticity ratio. The result is on the safe side since the modulus of rupture rises more than the modulus of elasticity. As a consequence the share of uncracked cross section increases.

example:	concrete grade = 35 N/mm²	(modulus of elasticity = 3,400 kN/cm²)
	load = 6.75 kN/m²	(according to figure 6.2.a)

theoretical load = 6.75 · (3000/3400) = 5.96 kN/m²

Therefore, the diagram's line for 6 kN/m² will be on the safe side. According to the diagram the example shown in figure 6.2.a results in a deflection of 1.1 cm (B25) or 0.9 cm (B35).

The diagrams always refer to the long-term deflection. The fixing degrees refer to the ratio of the moment at support (M) and the full fixed-end moment (M_F). This full fixed-end moment depends on the number of restrained edges:

one side is restrained $M_F = -ql^2/8$
two sides are restrained $M_F = -ql^2/12$

fixing degree $= \dfrac{M}{M_F}$

An interpolation and extrapolation of the diagrams' values can be done quite accurately. The indication of the slab thickness used in the diagrams refers always to

$\text{concrete}_{BOTTOM} + \text{EPS} + \text{concrete}_{TOP}$

The diagrams show clearly the kink with the following steep rise of deflection that characterizes the exceeding of the cracking moment.

LONG-TERM DEFLECTION (fixing degrees 0/0)
d = 50 + 50 + 60 mm, B25, ST500

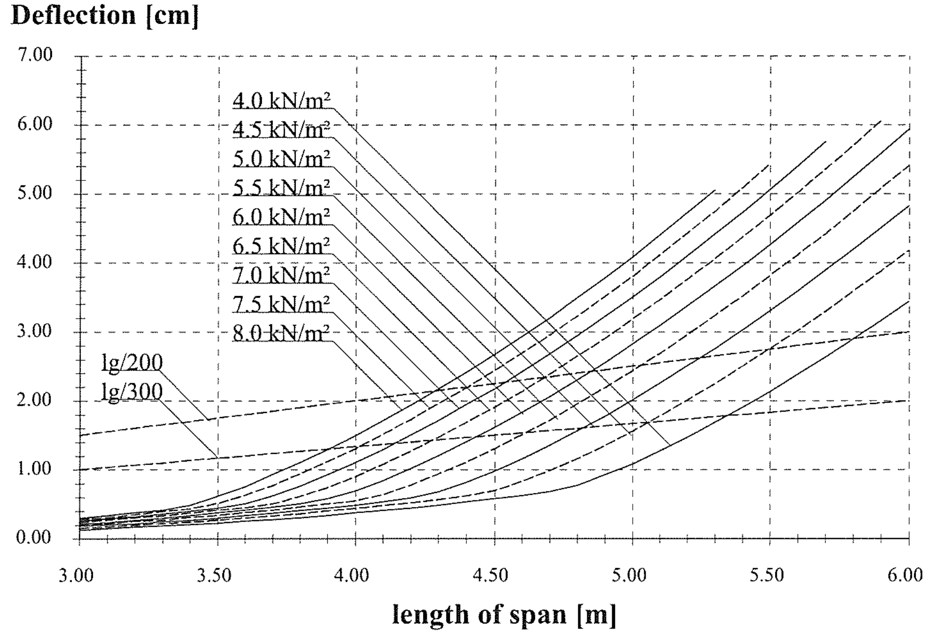


diagram 6.6.a. Slab 50 + 50 + 60 mm, fixing degrees 0/0

LONG-TERM DEFLECTION (fixing degrees 0.5/0)
d = 50 + 50 + 60 mm, B25, ST500

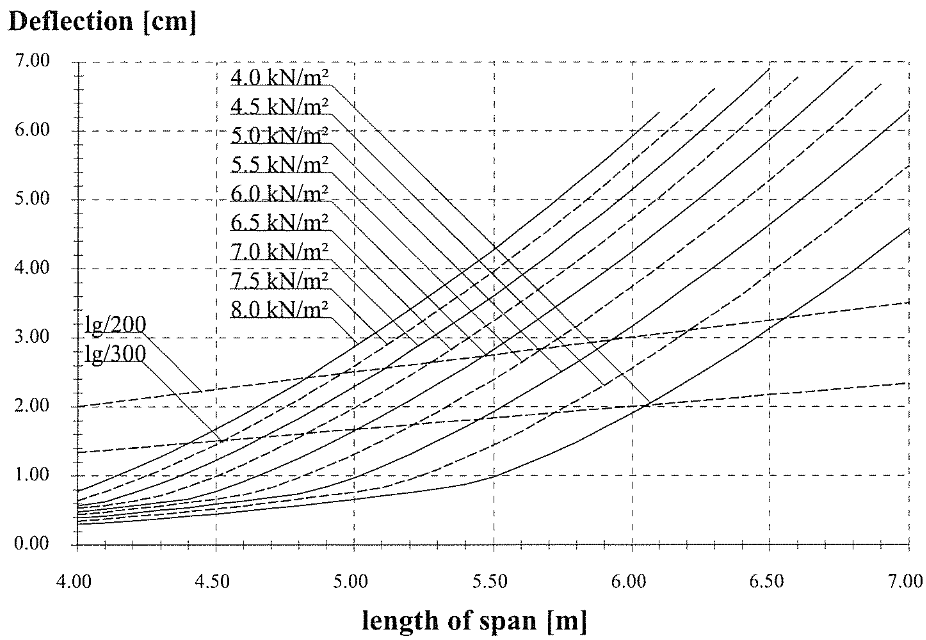


diagram 6.6.b. Slab 50 + 50 + 60 mm, fixing degrees 0.5/0

LONG-TERM DEFLECTION (fixing degrees 0.5/0.5)
d = 50 + 50 + 60 mm, B25, ST500

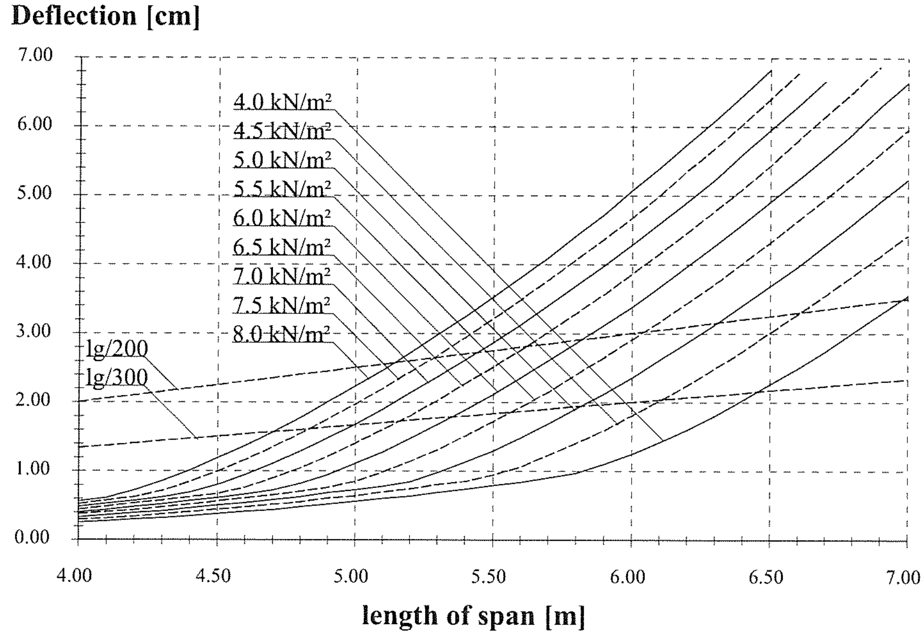


diagram 6.6.c. Slab 50 + 50 + 60 mm, fixing degrees 0.5/0.5

LONG-TERM DEFLECTION (fixing degrees 0/0)
d = 50 + 100 + 60 mm, B25, ST500

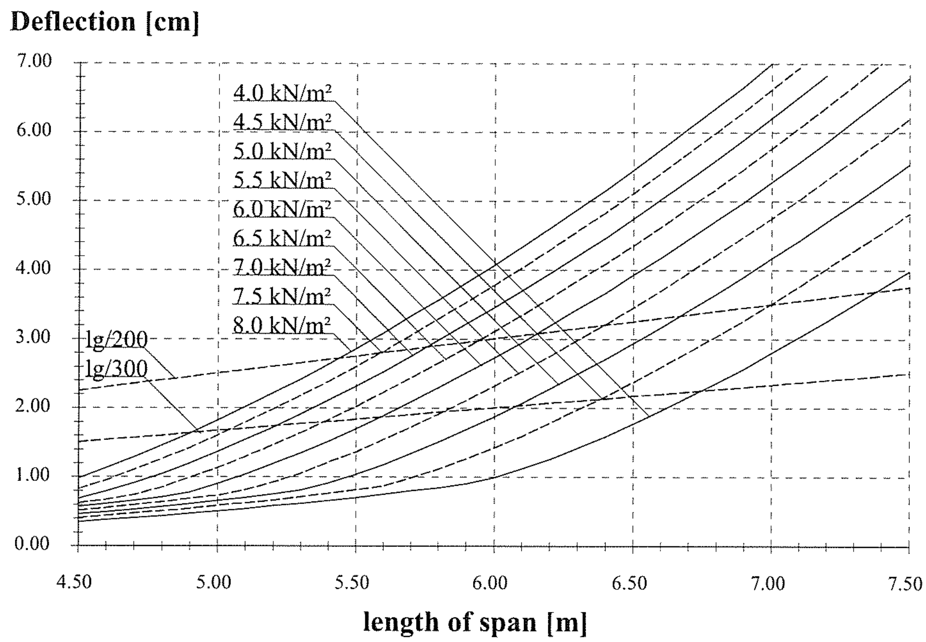


diagram 6.6.d. Slab 50 + 100 + 60 mm, fixing degrees 0/0

LONG-TERM DEFLECTION (fixing degrees 0.5/0)
d = 50 + 100 + 60 mm, B25, ST500

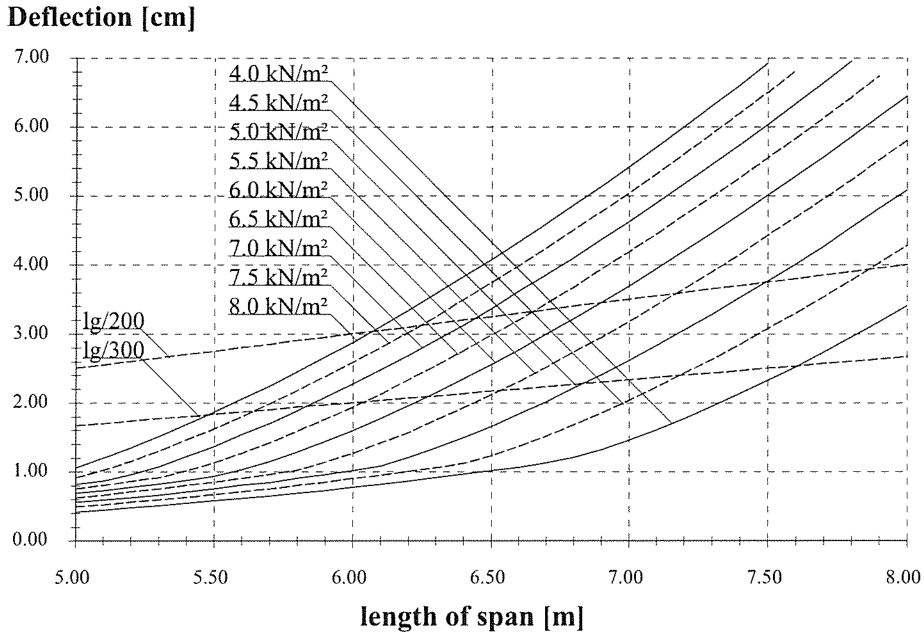


diagram 6.6.e. Slab 50 + 100 + 60 mm, fixing degrees 0.5/0

LONG-TERM DEFLECTION (fixing degrees 0.5/0.5)
d = 50 + 100 + 60 mm, B25, ST500

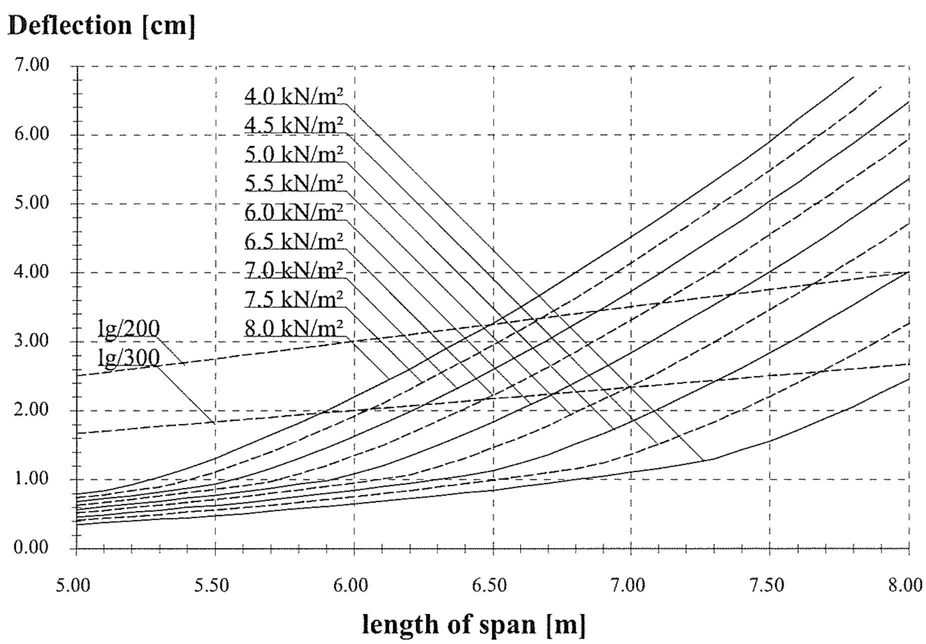


diagram 6.6.f. Slab 50 + 100 + 60 mm, fixing degrees 0.5/0.5

7. Concentrated Loads

NOTATION

b......... effective width of the slab
b_l .... width of the loaded area of a single moment
b_M ...... effective width for the design of the moment
b_V....... effective width for the design of the shear force
d......... effective depth
d_2 .... effective depth of topping concrete layer
fc........ specified compressive strength of concrete
F......... point load under service conditions
F_{ADM}.. maximum admissible point load under service conditions
fy .... specified yield strength of steel
lg........ length of span
M .... moment under service load
M_{ADM} maximum admissible moment under service load
M_t ...... moment in transverse direction
q......... uniformly distributed service load
q_{ADM}.. maximum admissible sevice load (uniformly distributed)
t_2 .... thickness of topping concrete layer
t_X........ extent of concentrated loads in load-bearing direction
t_Y........ extent of concentrated loads in cross direction
V........ shear force at support under service load
V_t .... shear force in transverse direction

All data of the following section were calculated on the basis of a slab with an EPS thickness of 100 mm, a bottom concrete layer of 50 mm and a top concrete layer of 60 mm. The effective widths are on the safe side for 3D slabs with a thicker top concrete layer in proportion to overall depth. In case of a smaller top concrete layer all values have to be reduced in proportion to the thickness of the top concrete layer.

7.1. Point Loads and Line Loads

If a concentrated load is applied to a 3D slab, it is possible to design the slab in the main direction with an effective width according to table 7.1.1.a. To do so, it is necessary to calculate the internal forces from the concentrated loads according to the rules of the traditional structural analysis and to distribute them over the effective width of the slab. The values of the table are designed for the most unfavorable cases and, as a consequence, some of them are clearly on the safe side.

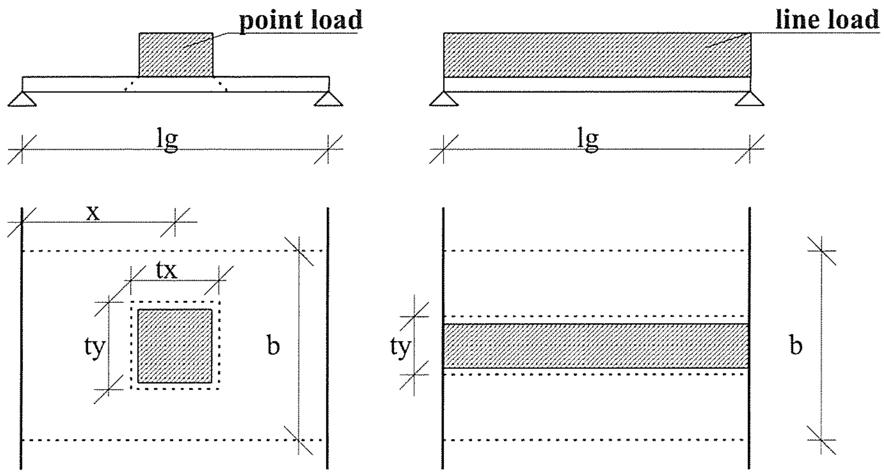


figure 7.1.a 3D slab with point load and line load

The extent of the concentrated loads can be increased in load-bearing and cross direction by the amount of the slab depth. Slab depth, however, refers only to the thickness of the top concrete layer and not to the entire slab thickness. If several concentrated loads have to be taken into consideration within the calculated effective width, it will be necessary to add the internal forces in this area consequently.

The validity of table 7.1.1.a is restricted to the following loaded areas:

Point loads :	extent in cross direction	$t_Y \leq 0.4 \cdot lg$
	extent in longitudinal direction	$t_X \leq 0.2 \cdot lg$
Line loads :	extent in cross direction	$t_Y \leq 0.2 \cdot lg$
	extent in longitudinal direction	$t_X = 1.0 \cdot lg$

If the extent of the loaded area is exceeded, the load has to be split up in several concentrated loads or calculation has to be done according to a more accurate method (e.g. with finite elements).

7.1.1. Effective Widths

Length *x* in table 7.1.1.a indicates the position of the point load. In addition to the internal forces in the actual load-bearing direction of the slab it is necessary to prove also the forces in transverse direction. In this case moments and shear forces occur as well, but they act on the top concrete layer only. The truss behaviour of the 3D cross section in transverse direction cannot be taken into consideration on account of the vertical arrangement of the diagonals (see figure 3.1.c). The data for internal forces in transverse direction apply for simply supported slabs. In case of slabs restrained on one or both sides, these values are slightly on the safe side.

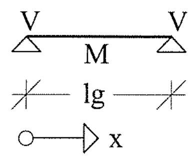
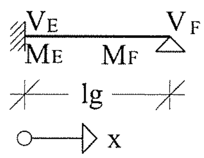
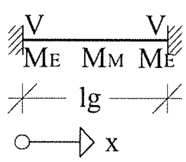
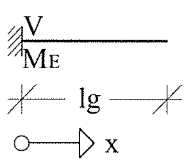
System	Internal Force	Point Load	Line Load
	V M	$b = t_Y + 0.5\ x$ $b = t_Y + 1.5\ x\ (1 - x/lg)$	$b = t_Y + 0.20\ lg$ $b = t_Y + 0.60\ lg$
	V_E V_F M_E M_M	$b = t_Y + 0.3\ x$ $b = t_Y + 0.4\ (lg - x)$ $b = t_Y + 0.5\ x\ (2 - x/lg)$ $b = t_Y + x\ (1 - x/lg)$	$b = t_Y + 0.20\ lg$ $b = t_Y + 0.20\ lg$ $b = t_Y + 0.35\ lg$ $b = t_Y + 0.50\ lg$
	V M_E M_M	$b = t_Y + 0.3\ x$ $b = t_Y + 0.5\ x\ (2 - x/lg)$ $b = t_Y + x\ (1 - x/lg)$	$b = t_Y + 0.20\ lg$ $b = t_Y + 0.30\ lg$ $b = t_Y + 0.40\ lg$
	V M_E	$b = t_Y + 0.3\ x$ $b = t_Y + x$	$b = t_Y + 0.40\ lg$ $b = t_Y + 0.75\ lg$

table 7.1.1.a Effective widths for concentrated loads

7.1.2. Point Loads

For simply supported slabs the moment in transverse direction can be calculated as follows

$$M_t = \frac{F}{10}$$

If the point load is applied to the free edge of a cantilever slab or is away from it less than lg/6, this moment will have to be doubled. Such a transverse moment is effective over a width of lg/3. The required additional reinforcement must have a length of 2/3 of the effective span of the slab plus anchoring length. 4/3 of the cantilever length have to be assumed for cantilever slabs. Besides, in the main direction 1/3 of the additional reinforcement has to reach into the free end and needs to be anchored by means of U-shaped stirrups. The slightly negative transverse moments are covered by the basic reinforcement of the panel and can be neglected for further considerations.

Transverse moments

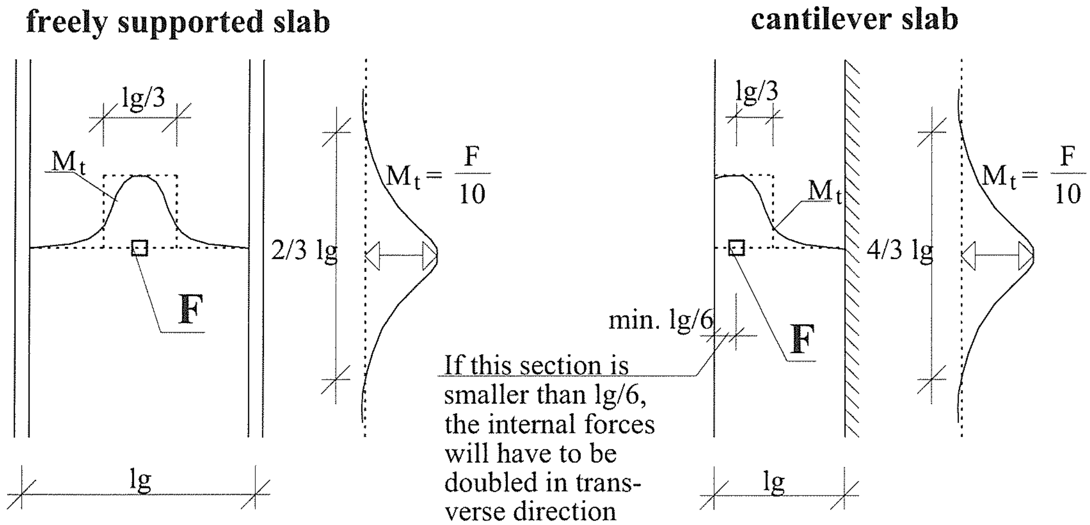


figure 7.1.2.a Moments in transverse direction

In a slab with 6 cm concrete topping, the effective depth can be assumed to be 4.5 cm. The panel's reinforcement amounts to 1.41 cm²/m (ST500). Hence the max. admissible point load results from

$$M_{ADM} = 1.41 \cdot 50 \cdot 0.95 \cdot 0.045 / 1.75 = 1.72 \text{ kNm/m}$$
$$F_{ADM} = 10 \cdot M_{ADM} = 17.2 \text{ kN}$$

Shear stress of a point load can be proved by a punching calculation. To do so, it is necessary to prove safety against punching of the top concrete layer. A connection to the bottom concrete layer cannot be taken into consideration on account of the unfavorable deformation behavior of the diagonals. Shear stress at the edge of the concentrated load has to be measured in order to prove safety against punching.

Shear Perimeter

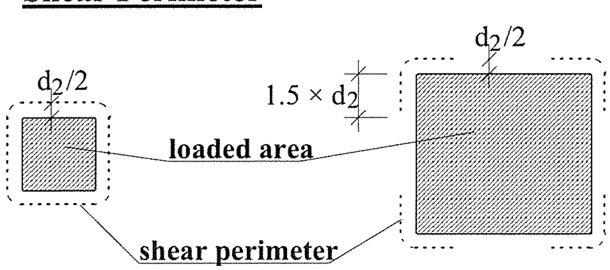


figure 7.1.2.b Shear perimeter

The shear perimeter is led in parallel around the loaded area. Sectors with a radius of $r = d_2/2$ are arranged in the corners. D_2 refers to the corresponding effective depth of the top concrete layer. In case of major loaded areas a distance of 1.5 x d_2 only can be taken to cover shear stress when starting from the corners.

In a slab with 6 cm concrete topping, the effective depth d_2 can be assumed with 4.5 cm. Owing to the small dimensions of the concrete layer, a shear reinforcement can not be added. For B25, a shear stress of 0.05 kN/cm² must not be exceeded. The restrictions in figure 7.1.2.b delimit the loaded area to be taken into account at the maximum to 13.5×13.5 cm and the shear perimeter to 68.1 cm. Hence the max. admissible concentrated load results from

$$F_{ADM} = 68.1 \cdot 4.5 \cdot 0.05 = 15 \text{ kN}$$

In most of the cases, this value should be sufficient. If a panel with 50 mm EPS instead of 100 mm EPS is used in the area of the concentrated load, the admissible concentrated load will even be increased to more than 68 kN. Hence the risk of punching can mostly be compensated by using a thinner panel and, as a consequence, by a thicker top concrete layer.

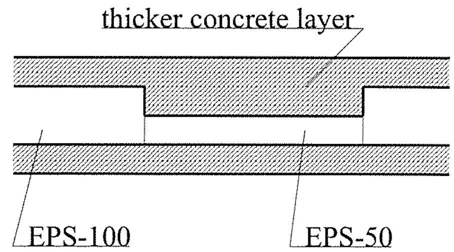


figure 7.1.2.c Slab with thinner panel

7.1.3. Line Loads

Analogously to the point loads, the transverse moments for line loads result in

$$M_t = \frac{q \cdot lg}{25}$$

If the line load ends at the free edge of a cantilever slab or less than lg/6 away from it, this moment will have to be doubled. Such a transverse moment is effective always over the entire span of the slab. The increased value at the free edge of a cantilever slab is limited to 1/6 of the cantilever length. Like in case of point loads, the required additional reinforcement must have a length of 2/3 of the effective span of the slab plus anchoring length. 4/3 of the cantilever length have to be assumed for cantilever slabs. The data of figure 7.1.3.a applies.

Transverse moments

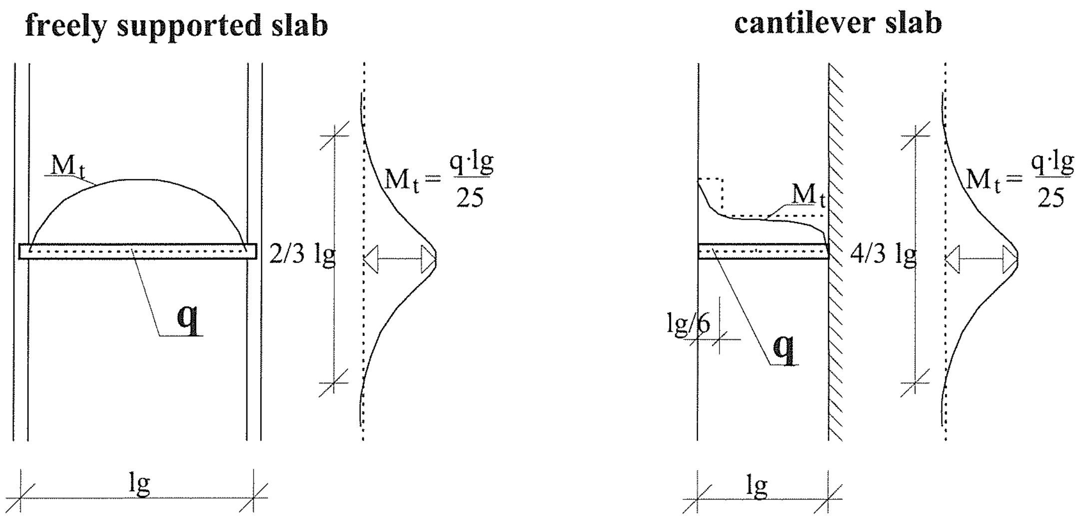


figure 7.1.3.a Moments in transverse direction

In a slab with 6 cm concrete topping, the effective depth can be assumed to be 4.5 cm. The panel's reinforcement amounts to 1.41 cm²/m (ST500). Hence the max. admissible line load results from

$$M_{ADM} = 1.41 \cdot 50 \cdot 0.95 \cdot 0.045 / 1.75 = 1.72 \text{ kNm/m}$$
$$q_{ADM} \cdot lg = 25 \cdot M_{ADM} = 43.1 \text{ kN}$$

Besides, it is necessary to calculate also shear force in cross direction. The following formula applies for a line load:

$$V_t = \frac{q}{2}$$

i.e. q represents the intensity of the line load. After a length of lg/3 this shear force is reduced more or less linearly to less than 10 %. As a cosequence, extent of transverse shear force is the same as for transverse moments. At the free edge of a cantilever slab twice the shear force has to be taken into consideration over a length of lg/6. Therefore it is advised to provide allways for edge beams in case of cantilever slabs with concentrated loads.

In a slab with 6 cm concrete topping, the effective depth can be assumed to be 4.5 cm. For B25, a shear stress of 0.05 kN/cm² must not be exceeded. Hence the maximum admissible line load results from

$$q_{ADM} = 2 \cdot 100 \cdot 4.5 \cdot 0.95 \cdot 0.03 = 25.7 \text{ kN/m}$$

In most of the cases, this value should be sufficient. If a panel with 50 mm EPS instead of 100 mm EPS is used in the area of the line load, the load will even be increased to more than 54 kN/m. Hence this problem can mostly be compensated by using a thinner panel and by a thicker top concrete layer (see figure 7.1.2.c).

7.1.4. Line Loads in Cross Direction

Line loads in cross direction up to a length of 0.4 · lg have to be designed like point loads by taking into consideration the effective width of the slab. The load has to be split up in case of wider areas. Owing to the fact that the load concentrates at the edges in case of a line load with a rigid construction (e.g. a 3D wall), the concentrated loads at the edges of the line load have to be taken into account with the value q·0.4·lg or half the total line load in this case. The remaining load has to be considered as continuous line load without additional load distribution. Internal forces in cross direction have to be calculated with these point loads then.

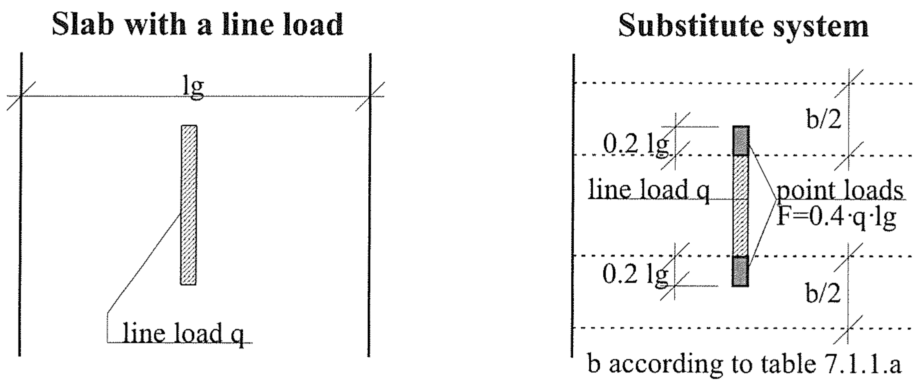


figure 7.1.4.a Line load perpendicular to direction of slab panels

7.2. Single Moments

Initiating moments in a 3D component is a very critical problem. The effective width of the slab or wall depends not only on the position of the point of application but – in case of several moments – on the distances between them as well.

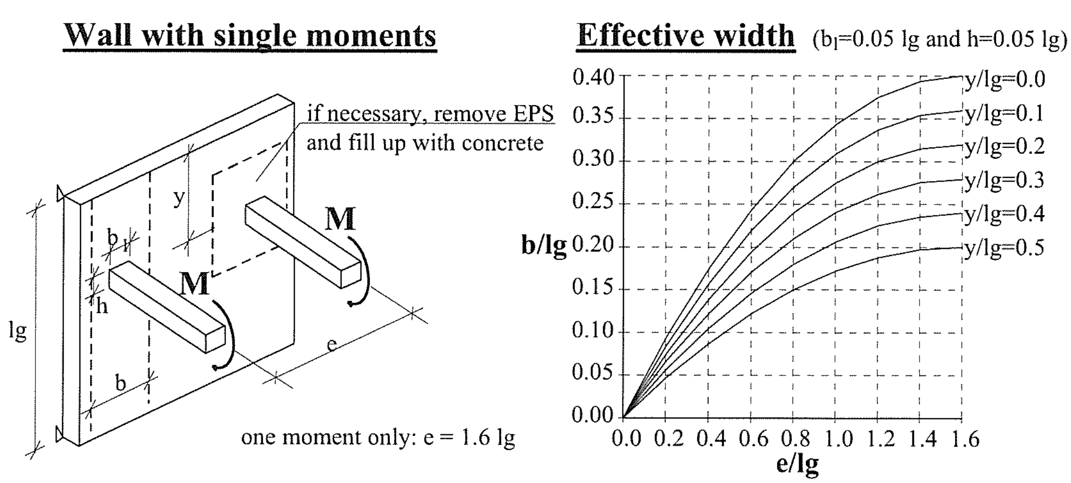


figure 7.2.a Wall with single moments

The effective width of slab for a single moment results in

$$b = \frac{e}{1.6 \cdot lg} \cdot \left(2 - \frac{e}{1.6 \cdot lg}\right) \cdot \left(b_1 + h + \left(0.3 - \frac{y}{2.5 \cdot lg}\right)\right) \qquad (1)$$

for the values see figure 7.2.a
if $e > 1.6 \cdot lg$ the values for b will remain constant

For y take always the smaller distance to the support. As a consequence, the effective width of the slab is the smallest if the moment is located at midspan. If a square section of the EPS is removed at the moment's point of application analogously to figure 7.2.a, at most twice the value according to formula (1) is allowed as effective width. This case given, the square section without EPS must correspond to the total effective width.

7.3. Flush Beam Strips

In general, 3D slabs are acting as one way slabs and require continuous supports. In the area of interrupted supports this line support can be replaced by an ideal girder. In order to avoid excess deformations of the slab in direction of this girder, the length of such girders has to be limited to the 15-fold size of the 3D slab. In case of wider effective spans it is recommended to use higher beams to be able to avoid cracks in the light walls on the floor above. Another field of application of flush beam strips in 3D slabs is the transfer of major concentrated loads. Especially lattice girders are suitable as reinforcement elements. This case given, the recommended height of the girder is, of course, not limited to 1/15 of the effective span.

Flush beam strips are to be designed like I-beams. In this case the slab thicknesses correspond to the two concrete layers. By approximation it is possible to take the effective widths of the slab from sketches 7.3.a and 7.3.b. These values are considerably lower than the usual assumptions for the design of flush beam strips in cast-in-situ slabs. Besides, for a 3D slab the width in the area of the positive and negative moment is the same. Width results in

$$b_M = b_{Wall} + 2 \times 0.1 \cdot lg$$
$$b_V = b_{Wall}$$

i.e. b_{Wall}... width of the wall

Of course, for an edge beam the width of the wall + $0.1 \cdot lg$ must be assumed only. The width of the wall only must be taken for the design of the shear force.

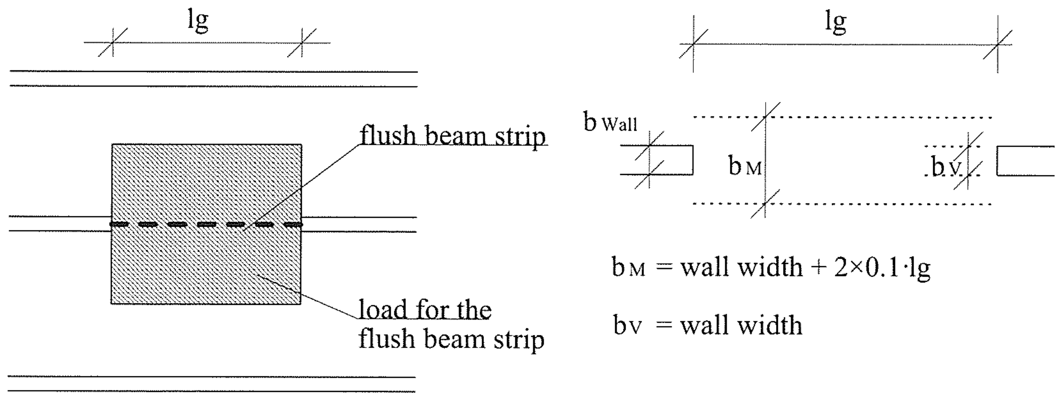


figure 7.3.a Flush beam strip

The effective width of the slab for a flush beam strip in load-bearing direction is determined by analogy. However, the girder width has to be put in instead of the wall width. In case of a very wide girder the values of table 7.1.1.a should not be exceeded without a prior more accurate check calculation. In both cases the shear-resistant connection with the concrete layers must be ensured by the use of additional reinforcement elements (e.g. splice meshes).

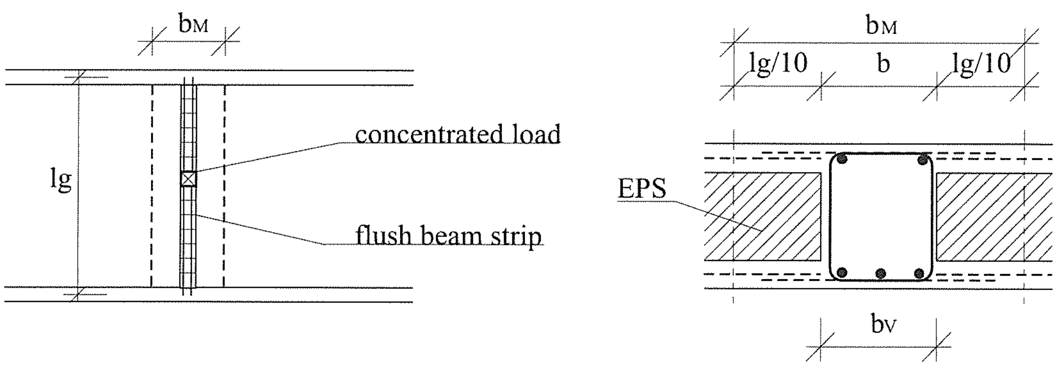


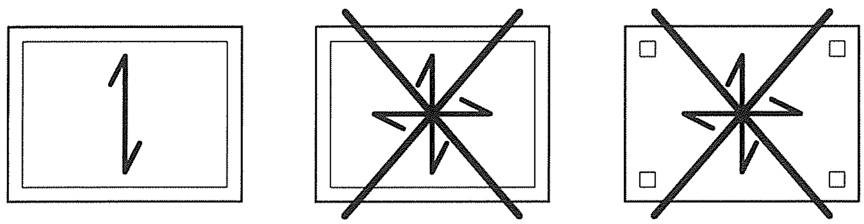
figure 7.3.b Strengthened slab

8. Slabs

For the notation see chapters 2 to 7.

8.1. System

3D slabs are always designed as simply supported or continuous slabs. A slab acting in two ways cannot be recommended with 3D panels.



one-way slab (preferred with 3D) | **two-way slab** | **flat slab** (not recommended with 3D)

figure 8.1.a Slab systems

Therefore, slabs made from 3D elements require always continuous supports. In an area where continuous supports are interrupted flush beam strips have to form a support, e.g. above doors having the same height as the room (see 7.3).

Usually slabs are designed as continuous supported systems. This continuous effect must be taken into consideration only if the panels of the individual floor slabs run in the same direction. A restraint at the cross edge of another slab bay must not be assumed.

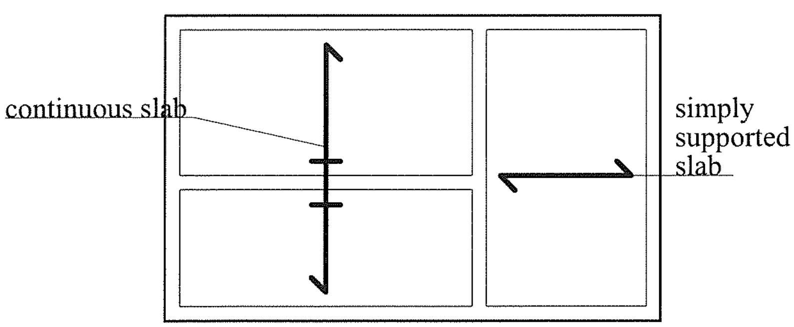


figure 8.1.b Slab system with continuous floor slab and a simply-supported slab

In some special cases only it is possible to connect a cantilever slab at the cross edge of another floor slab (see 8.4.1). This case given, however, the allowed cantilever moment is restricted to a very low value. If this moment is exceeded, it will be necessary to take special measures. The application of concentrated loads is possible to a limited extent, either (see section 7.1).

8.2. Minimum Reinforcement

The thickness of 3D slabs is limited by the thickness of the panels' EPS core (40 to 100 mm). In order to avoid an excessive dead weight, the thickness of the bottom concrete layer should not exceed 40 to 50 mm. In general, the top concrete layer has a thickness of 50 or 60 mm. The minimum thickness of the bottom concrete layer in case of a continuous slab with negative moments is 50 mm. If it is only 40 mm thick the slab has to be calculated as a simply supported slab. Besides, it will be difficult to place additional reinforcement in case of a 40 mm thick concrete layer.

Concrete grade B25 (fc = 17.5 N/mm²) will do in most cases. For this concrete grade the panel's reinforcement is sufficient as minimum reinforcement of the slab. DIN 1045 requires minimum reinforcement according to the following formula:

$$\mu_{min} = \frac{As}{Ac_T} = \frac{k_0 \cdot f_r}{f_S}$$

i.e.
- As ...... area of reinforcement
- Ac_T .... area of concrete in tensile zone
- k_0 .... 0.4 for flexural cross sections
  1.0 for tensile cross sections
- f_r .... modulus of rupture of concrete
  $f_r = 0.25 \cdot W_{28}^{2/3}$ [N/mm²]
  where W_{28} is the cube strength of concrete after 28 days
- f_S........ effective steel stress according to DIN 1045, table 14

The value of f_S depends on the diameter and the position of the rebar. For the panel's reinforcement (ø 3.0 mm, ST500) it amounts always to 400 N/mm². When using a steel grade below ST500 f_S must not be assumed to be higher than 80 % of the yield strength. Stress f_S that depends on the rebar's diameter can be seen from table 14 of DIN 1045.

For the calculation of the concrete's modulus of rupture a minimum nominal strength of 35 N/mm² has to be taken into consideration. However, for 3D cross sections there arises another risk since flexural cross sections are characterized in general by a triangular stress block in the tensile zone while stresses in the tensile zone of a 3D slab run trapezoidally. Therefore, the theoretical concrete grade has to be assumed one degree above the actual concrete grade to keep the safety standards (this fact is already included in table 8.2.a). Basing on this correlation the usual dimensions of the bottom concrete layer result in the following minimum reinforcement:

concrete layer [mm]	40	50	60
≤ B25	1.07	1.34	1.61
B35	1.27	1.58	1.90

table 8.2.a Minimum reinforcement in [cm²/m]

Table 8.2.a shows that the panel's reinforcement is sufficient as minimum reinforcement only for a concrete layer thickness of 50 or 40 mm. For thicker concrete layers it is inevitable to provide always for additional reinforcement. These minimum reinforcement corresponds quite well to minimum reinforcement according to Austrian standard ÖNORM B4200.

The values of table 8.2.a apply only if the center of gravity of the cross section lies within the EPS. If, in case of a very thick topping concrete, the centroidal axis lies within the top concrete layer, it will be necessary to take into consideration the entire tensile zone of the 3D cross section for the determination of minimum reinforcement. The panel's top reinforcement should not be taken into account since it is located close to the neutral axis. Depending on the thickness of the bottom concrete layer the maximum thickness of the top concrete layer ranges from 80 to 90 mm for a 50 mm EPS core, and from 100 to 110 mm for a 100 mm EPS core when applying table 8.2.a.

The required minimum reinforcement, however, increases for thicker rebar diameters owing to the decreasing effective steel stress f_S. For bars with a diameter of up to 8 mm it is possible to assume only 350 N/mm² for outside components, such as cantilever slabs. A value of 1.84 cm²/m (B25) or 2.17 cm²/m (B35) has to be put in for a top concrete layer of 60 mm.

This increased minimum reinforcement is decisive also for the reinforcement at the support. Usually, stirrups with a ø of 8 mm are used at the outer support. For the inner support slightly thicker bars (ø 10 mm) are recommended. During construction these straight bars are also used temporarily to transfer compression forces and are subject to buckling stress.

In some cases it is required to strengthen the top concrete layer. If the slab is subject to a concentrated load, possibly the top concrete layer will not be sufficient to transfer the transverse moment and shear force. A way to solve this problem without increasing the overall slab thickness is the use of a panel with a thinner EPS core (50 instead of 100 mm) and, as a consequence, a thicker concrete layer. Thus the panel's cross reinforcement has to satisfy the requirements for minimum reinforcement. Owing to the fact that this slab is considered a regular concrete slab, the panel reinforcement would be sufficient for concrete grades up to B35, provided that the thickness of the top concrete layer remains below approximately 110 mm.

8.3. Dimensioning the Reinforcement at Support

The practical dimensioning of the U-shaped stirrups at the support is done according to German standard DIN. According to DIN, at least half of the slab's maximum reinforcement has to run to the support where it has to be anchored safely. This implies that the stirrup's cross section must be at least half of the reinforcement at midspan.

Figure 8.3.a shows the forces at the support. By assuming that the compression diagonal with the force C runs under 45°, the tensile force T is identical with shear force V. When taking into consideration the influence of the reinforcement's development approximately the same applies to the 3D-slab. This means, the reinforcement has to be designed at least according to shear force V.

reinforced concrete beam **3D-slab**

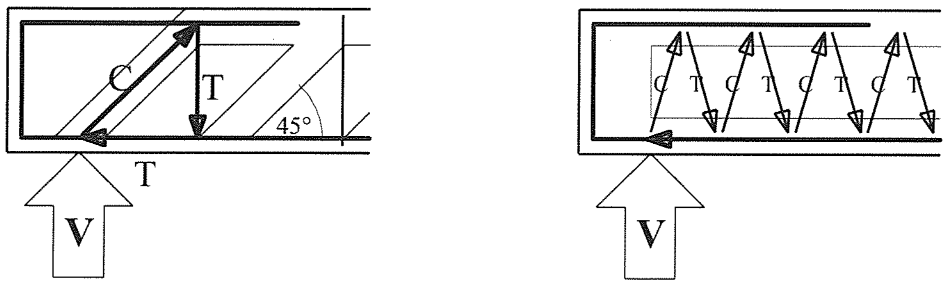


figure 8.3.a Transfer of shear force

In slabs that are subject to minor loads, the reinforcement at the support must not be smaller than minimum reinforcement, viz. ø 8 mm with a pitch of 25 cm in general.

8.4. Handling of Transverse Moments

Transverse moments occur mostly in 3 cases:

- cantilever slabs at the cross edge of a floor slab
- roof overhangs in panel's cross direction
- in the area of concentrated loads (see chapter 7)

8.4.1. Cantilever Slabs at the Cross Edge

Cantilever moments that act at the cross edge of a floor slab can be borne by the slab to a very restricted extent only. They cause not only a problem to structural strength but to deformation of the inside slab as well. While structural strength can be achieved by additional reinforcement, deformation depends mainly on the slab's cross section and can be influenced only slightly by enlarging the reinforcement.

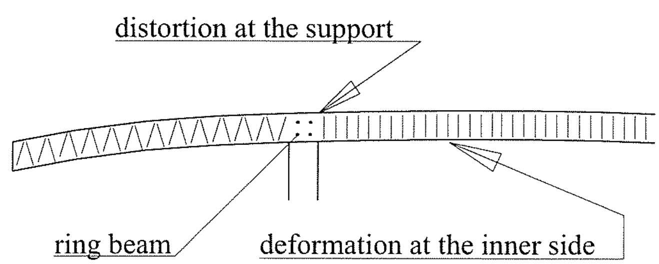


figure 8.4.a Deformation of a 3D slab by a cantilever moment

The tensile force of the cantilever slab is transferred to the interior slab. This slab works like a deep beam with a load applied at the bottom edge. Therefore, a ring beam is necessary. The maximum admissible moment cannot be determined exactly. Due to experience the cantilever moment shall not be higher than the moment given by the basic reinforcement of the 3D panel.

By applying a safety factor of 1.75 the following admissible cantilever moments under service load must not be exceeded in case of a 50 mm concrete layer at the slab's bottom side (compression zone):

EPS-100 $M_{ADM} = 6.5$ kNm/m
EPS-50 $M_{ADM} = 4.5$ kNm/m

At the inside the cantilever reinforcement has to be viewed analogously to the calculation of deep beams with a load applying at the bottom edge. Therefore, the reinforcement must reach at least up to the half length of the internal span. This implies that the panel joints at the slab's top side have to be overlapped completely by splice mesh. Due to the fact that this area is subject to flexure, an overlap of 4 mesh loops has to be kept according to table 1.4.a. Thus, the width of the splice mesh in this area is 45 cm. Besides, the length of the cantilever reinforcement at the inside must be always at least 1.5 times the cantilever length.

8.4.2. Roof Overhang

Pitched roofs are made predominantly with overhangs at all sides. Usually the panels run normally to the eaves. Thus a cantilever forms at the gable side in the panel's cross direction. This moment has to be assigned to the bottom concrete layer only. The effective depth of this layer is only t_l – 15 mm, i.e. t_l refers to the thickness of the bottom layer that ranges from an average 40 to 50 mm. Therefore, under usual circumstances the following example shall demonstrate the most unfavorable case.

load	$q = 4.0$ kN/m²	(weight of slab, roof and a minor live load)
concrete B25	$h = 40$ mm	(effective depth = 25 mm)
reinforcement	1.41 cm²/m	(cover mesh ø 3 mm, 50 × 50 mm)

$$M_{ADM} = 1.41 \cdot 50 \cdot 0.025 \cdot 0.9 / 1.75 = 0.91 \text{ kNm/m}$$

For a cantilever slab the moment is $q \cdot lg^2/2$; thus the admissible length results in

$$lg_{ADM} = \sqrt{\frac{2 \cdot M}{q}} = 0.67 \text{ m}$$

As a consequence, a roof overhang of half a panel width can be considered to be safe. With a concrete layer of 50 mm and a load of only 3.0 kN/m² the admissible overhang will increase to 0.92 m. This implies that even under the most favorable circumstances an overhang of one entire panel width goes far beyond the admissible extent.

8.4.3. Cantilever Slabs supported by Beams (Flush Beam Strips)

If the cantilever moment exceeds the values depicted in item 8.4.1, the moment has to be transferred by beams or flush beam strips.

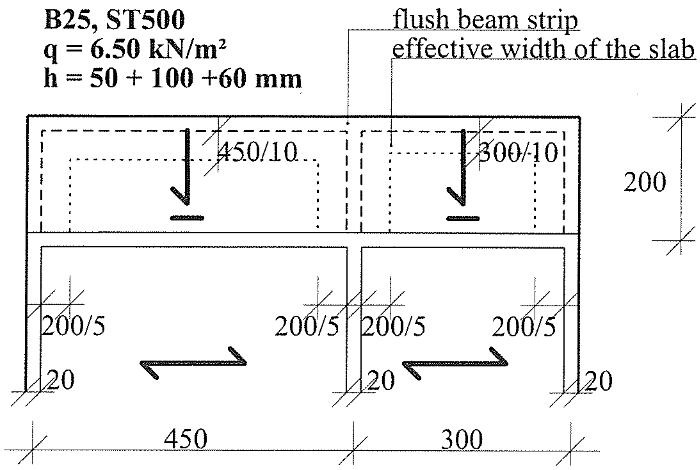


figure 8.4.3.a Cantilever slab with flush beam strips

Seen from the structural point of view, the cantilever beams are designed as an elongation of the internal wall and have also the same width. However, the edge beam running crossways is designed only according to the structural requirements and should be as narrow as possible. Possibly the ordinary edge of the cantilevered slab with a U-shaped splice mesh and additional longitudinal reinforcement at the top and bottom side may be sufficient.

A cantilever moment of $q \cdot lg^2/2 = 13.0$ kNm/m has to be transferred in the example according to figure 8.4.3.a. However, the admissible cantilever moment is 6.5 kNm/m only. This share may be applied at the cross edge of the inner floor slab. The remaining cantilever moment has to be transferred by the flush beam strips.

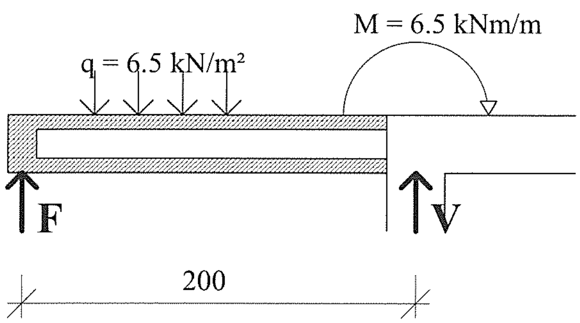


figure 8.4.3.b System drawing of the cantilever slab; force F has to be transferred by the beams.

The force F, depicting the load of the outer girder grid results in

$$F = ql/2 - M/lg = 6.5 \cdot 2.00/2 - 6.5/2.00 = 3.25 \text{ kN/m}$$

Shear force V in the 3D slab results in

$$V = q \cdot l - F = 2.00 \cdot 6.50 - 3.25 = 9.75 \text{ kN/m} < 14.3 \text{ kN/m (table 3.6.a)}$$

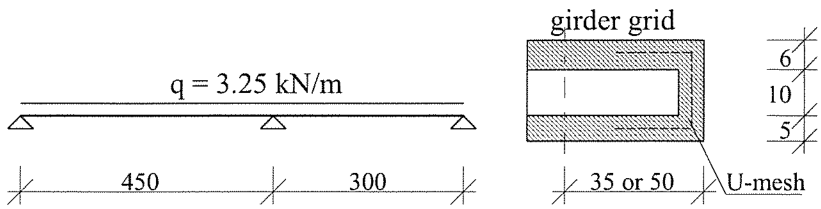


figure 8.4.3.c System drawing of the edge beam

The support reactions of this continuous beam constitute the load for the cantilever beams (= F_R in figure 8.4.3.d). The dead weight needs not to be taken into consideration in any cases since it is already included in the floor load. It is the beam in the middle of the three cantilevering beams that is designed. For the edge beams it is possible to lay approximately half of the reinforcement required for the middle beam.

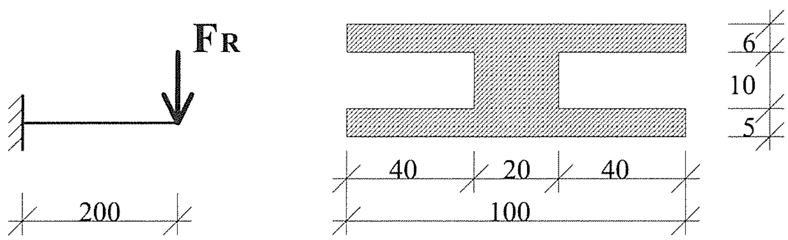


figure 8.4.3.d System drawing of the middle cantilever beam

```
CONTINUOUS BEAM OUTSIDE
concrete grade B25   steel grade ST500
h = 21.0 cm     d,t = 17.5 cm    d,b = 18.0 cm

MOMENTS
M,min     [kNm/m]                       -6.40
As,top    [cm²/m]                        1.28
                 |-------part 1--------|-------part 2--------|
M,max     [kNm/m]          5.34                   1.16
As,bottom [cm²/m]          1.07                   0.23

ZERO POINT OF MOMENTS
                 |-------part 1--------|-------part 2--------|
pitch     [cm]   |      362       |  88 | 131 |      169      |

SHEAR FORCES (basic value = 0.0 kN/m)
V,max     [kN/m]    5.89                  7.01
as,shear  [cm²/m]   1.19                  1.46
                 |-------part 1--------|-------part 2--------|
V,min     [kN/m]                -8.73                   -2.74
as,shear  [cm²/m]                1.82                    0.55
```

For longitudinal reinforcement 2 ø 10 mm each on the top and on the bottom are sufficient. In the area of the middle support it is necessary to use additional shear reinforcement at the U-mesh (e.g. 5 ø 8 mm, e = 25 cm). The edge load F_R of the cantilever beam is the sum of the two shear forces at the middle support and results in

$$F_R = 8.73 + 7.01 = 15.74 \text{ kN}$$

```
CANTILEVER BEAM MIDDLE
concrete grade B25   steel grade ST500
h = 21.0 cm     d,t = 17.5 cm    d,b = 18.0 cm


MOMENTS
M,min     [kNm/m]                   -31.48
As,top    [cm²/m]                     6.64
                   --------part 1--------|


SHEAR FORCES (basic value = 0.0 kN/m)
                   --------part 1--------|
V,min     [kN/m]                    -15.74
as,shear  [cm²/m]                     3.32
```

6 ø 12 mm (= 6.79 cm^2) are installed as flexural reinforcement. The stirrup cage is designed with ø 8 mm, e = 25 cm (= 2 × 2.01 cm^2/m). In both edge beams 3 ø 12 mm each are required only.

For edge beams it is advised to check whether a stirrup cage made from rebars is required. On the assumption that it is sufficient to use U-mesh for edge strengthening, the width of the edge beam results in

$$b = 0.05 + 2.00/5 = 0.45 \text{ m}$$

The entire shear force in the area of the left cantilever beam is

$$V = 5.89 + 0.45 \cdot 9.75 = 10.28 \text{ kN}$$

The admissible shear force is composed of the splice mesh and the panel's share (200 diagonals per m^2) :

$$V_{ADM} = 6.5 + 0.4 \cdot 14.3 = 12.2 \text{ kN} > 10.28 \text{ kN}$$

Thus it is not necessary to use at the edge a stirrup cage made from rebars.

8.5. Design Example

The following section shows an example of the structural design of a 3D slab. A conventional computer program for the calculation of continuous beams is used to calculate the floor slabs.

8.5.1. System Basics

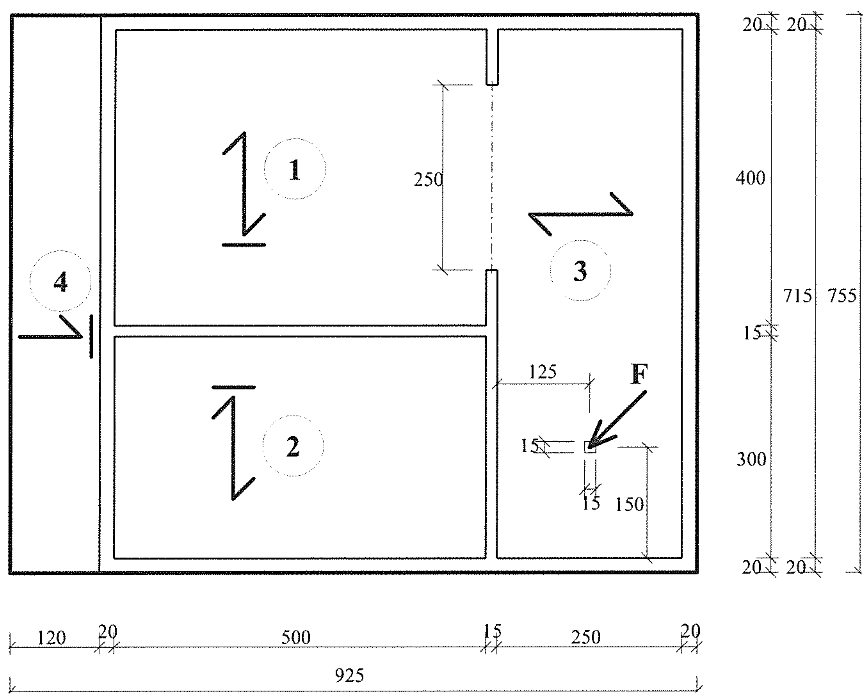


figure 8.5.1.a
Example of a ground plan

The loads are determined for a slab with 50 + 100 + 60 mm. Usually the live load for residential buildings is 2.00 kN/m².

dead load				
concrete (50 + 60 mm)			2.75	kN/m²
floor construction			1.75	kN/m²
	D.L.	=	4.50	kN/m²
live load	L.L.	=	2.00	kN/m²
TOTAL LOAD	**q**	**=**	**6.50**	**kN/m²**
POINT LOAD	**F**	**=**	**20.00**	**kN**

The floor slabs 1 and 2 are calculated as continuous spans. Floor slab 3 must be considered as simple span. The cantilever slab 4 is connected to the cross edge of slabs 1 and 2.

The slab panels have 200 diagonals per m² and a 100 mm EPS core. Basic reinforcement is 1.41 cm²/m. Without additional shear reinforcement the panel can resist a shear force of 14.3 kN/m (table 3.6.a). A safety factor of 1.75 according to DIN was chosen.

8.5.2. Design of Reinforcement

In the following calculations the flexural reinforcement (=A_S) refers to the overall reinforcement while shear reinforcement (=a_S) indicates only the additional reinforcement for the panel. This shear reinforcement is applied as U-shaped stirrup made of splice mesh. The reinforcement data refer to a slab strip of 1.00 m and have to be multiplied by 1.20 for a standard panel.

The minimum moment at midspan that can be found in a slab restrained on one or both sides is assumed always as the moment in a slab with fully restrained edges. This takes into consideration the obstructions of torsion at the point of support. This moment at midspan depends on the number of restrained edges.

one side is restrained	$M = 9ql^2/128$
two sides are restrained	$M = ql^2/24$

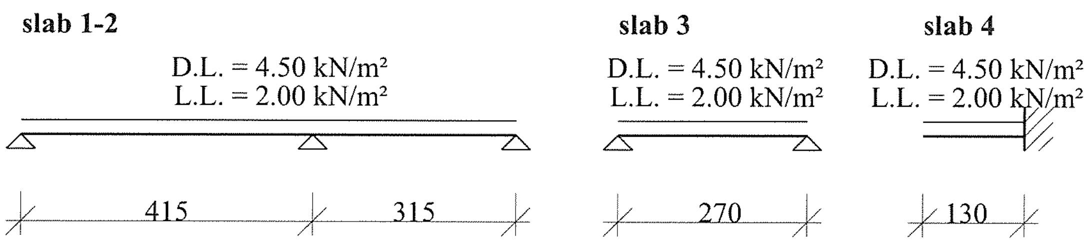


figure 8.5.2.a System drawing

8.5.2.1. Continuous Beams

```
CONTINUOUS BEAM 1-2
concrete grade B25   steel grade ST500
h = 21.0 cm      d,t = 17.5 cm     d,b = 18.0 cm


MOMENTS
M,min     [kNm/m]                      -11.43
As,top    [cm²/m]                        2.39
                  |-------part 1--------|-------part 2--------|
M,max     [kNm/m]          9.29                   4.19
As,bottom [cm²/m]          1.88                   0.84


ZERO POINTS OF MOMENTS
pitch     [cm]    |     319       |  96  | 146  |      169      |
                  |-------part 1--------|-------part 2--------|
pitch     [cm]    |      370       |  45  |  88 |      227      |


SHEAR FORCES (BASIC VALUE = 14.3 kN/m)
V,max     [kN/m]    10.99                  13.87
as,shear  [cm²/m]    0.00                   0.00
                  |-------part 1--------|-------part 2--------|
V,min     [kN/m]                 -16.24                  -7.38
as,shear  [cm²/m]                  0.41                   0.00
```

```
CONTINUOUS BEAM 3
concrete grade B25   steel grade ST500
h = 21.0 cm     d,t = 17.5 cm    d,b = 18.0 cm


MOMENTS
                     |-------part 1--------|
M,max      [kNm/m]            5.92
As,bottom  [cm²/m]            1.19


SHEAR FORCES (basic value = 14.3 kN/m)
V,max      [kN/m]     8.78
as,shear   [cm²/m]    0.00
                      |-------part 1--------|
V,min      [kN/m]                      -8.78
as,shear   [cm²/m]                      0.00


CONTINUOUS BEAM 4
concrete grade B25   steel grade ST500
h = 21.0 cm     d,t = 17.5 cm    d,b = 18.0 cm


MOMENTS
M,min      [kNm/m]                    -5.49
As,top     [cm²/m]                     1.10
                      --------part 1--------|


SHEAR FORCES (basic value = 14.3 kN/m)
                     --------part 1--------|
V,min      [kN/m]                     -8.45
as,shear   [cm²/m]                     0.00
```

These calculations show that additional tensile and shear reinforcement is to be placed only in the first floor slab. The additional bottom reinforcement runs over the entire panel length. 4 ø 8 mm per panel are selected as reinforcement. This results in a total reinforcement area of 3.09 cm²/m. Owing to the position of the zero points of moments the top reinforcement with a length of 2.50 m is laid between floor slabs 1 and 2. As reinforcement ø 8 mm are arranged in a spacing of 20 cm.

The cantilever reinforcement has to reach inside up to the half of the inside span or must be at least 1.5 times the cantilever length. This brings about a total length of approx. 3.5 m. Therefore, the first panel joint must be designed with the full overlapping length. To do so, splice mesh (45cm) is used on account of the rebar's bigger development length. The cantilever reinforcement (ø 8 mm, e = 25 cm, as = 2.01 cm²/m) with a length of 2.50 m is laid centrally over the outside wall. In order to avoid cracks, a top reinforcement not less than the minimum reinforcement must be provided for floor slab 3, i.e. ø 8 mm with a pitch of 25 cm.

For a 1.0 m wide slab strip the indicated additional shear reinforcement is 0.41 cm²/m. Therefore, $0.41 \cdot 1.20 = 0.49$ cm²/m are required per panel. A stirrup cage made of splice mesh is sufficient. Its length can be calculated on the basis of the exceeded admissible shear force, i.e. $16.24 - 14.3 = 1.94$ kN/m. Thus the length of the additional reinforcement is 1.94 kN/m / 6.50 kN/m² = 0.30 m.

8.5.2.2. Point Load

The point load in floor slab 3 has to be designed according to chapter 7. The first calculation refers to the safety against punching of the top concrete layer. The effective depth d_2 results in d = 60 - 15 = 45 mm. The effective side length according to paragraph 7.1.2 results in

$$b = 3 \cdot d = 3 \cdot 45 = 135 \text{ mm}$$

Thus the shear perimeter is

$$u = 4 \cdot b + 2 \cdot \pi \cdot d = 823 \text{ mm}$$

Therefore, shear stress is

$$\tau = \frac{F}{u \cdot d} = \frac{20000}{823 \cdot 45} = 0.54 \text{ N/mm}^2 > 0.50 \text{ N/mm}^2$$

The top concrete layer is not sufficient. There are two ways to increase the load-bearing capacity without changing total thickness:

- application of a flush beam strip
- application of a panel with a 50 mm EPS core (instead of 100 mm) below the load

In this case, load is not that high to use a flush beam strip. In practice it is recommended to use a panel with a 50 mm EPS core. For the sake of simplicity this will be a wall panel with 100 diagonals per m² only. Admissible shear force is 10.9 kN/m (table 3.6.b). For the design of shear forces it is necessary to take into consideration the additional dead weight of 1.25 kN/m². Therefore the internal forces for the uniformly distributed load result in

$$\begin{aligned} q &= 6.5 + 1.25 &&= 7.75 \text{ kN/m}^2 \\ M &= ql^2/8 &&= 7.06 \text{ kNm/m} \\ V &= ql/2 &&= 10.46 \text{ kN/m} \end{aligned}$$

A distribution width may be assumed for the design of the moment and the shear force according to table 7.1.1.a.

$$\begin{aligned} b_M &= 0.15 + 0.06 + 1.5 \cdot 1.25 \cdot (1 - 1.25/2.50) &&= 1.15 \text{ m} \approx 1.0 \text{ m} \\ b_V &= 0.15 + 0.06 + 0.5 \cdot 1.25 &&= 0.84 \text{ m} \approx 0.8 \text{ m} \end{aligned}$$

For the simply supported beam with a point load at midspan the moment is

$$\Delta M = F \cdot lg/4 = 12.5 \text{ kNm}$$

Thus the total moment lies clearly within the admissible range. As a consequence, the required additional reinforcement results in

$$\Delta As = \frac{1.75 \cdot (12.5 + 7.06)}{0.9 \cdot 0.18 \cdot 50} - 1.0 \cdot 1.41 = 2.82 \text{ cm}^2 \quad (6 \text{ ø } 8 \text{ mm} = 3.02 \text{ cm}^2)$$

Additional shear force is F/2 = 10.0 kN. Owing to the fact that there is a minor restraint at the inner support, the theoretical shear force is increased by 10 %. Thus the total shear force results in

$$V = 1.10 \cdot (10.46 + ½ \cdot 20.0/0.80) = 25.26 \text{ kN/m} > 10.9 \text{ kN/m}$$

The required shear reinforcement is designed for a shear force of

$$\Delta V = (25.26 - 10.9) \cdot 0.80 = 11.40 \text{ kN}$$

According to paragraph 3.7.3 a U-shaped stirrups made of splice mesh can resist a shear force of 6.5 kN/m. Thus 2 stirrup cages are required. Shear force under the concentrated load is still

$$lg = 25.26 - (1.25 \cdot 7.75) = 15.57 \text{ kNm/m} > 10.9 \text{ kNm/m}$$

Therefore, the stirrups made of splice mesh must run over the entire span. In this case it is advisable to fix the U-mesh to both panel joints in the area where the point load is applied. In order to make sure that the stirrup cages lie within the distribution width, the thinner panel's width has to be roughly 60 cm. In this case the calculation is on the safe side.

The transverse moment under a single load is

$$M_t = \frac{F}{10} = 2.00 \text{ kNm/m}$$

$$As = \frac{1.75 \cdot M_t}{0.9 \cdot d_2 \cdot fy} = \frac{1.75 \cdot 2.00}{0.9 \cdot 0.095 \cdot 50} = 0.82 \text{ cm}^2\text{/m} < 1.41 \text{ cm}^2\text{/m}$$

In cross direction the panel’s reinforcement is sufficient. The transverse moment is decreased immediately at the point of the top concrete layer's transition from 110 mm to 60 mm. In this panel joint it is sufficient that the splice mesh on top reaches into the area of the thicker concrete layer. As a matter of course, the splice mesh must be 45 cm wide right below the load.

8.5.2.3. Flush Beam Strip

Between floor slab 1 and 2 there is a 2.5 m long distance without support. In order to provide for a continuous support a flush beam strip is necessary in this area. The ratio

$$lg/h = 2.70/0.21 = 13 < 15$$

lies within the admissible range (section 7.3). The beam’s load results from the dead load and the support reactions of floor slab 1 and 3. The reactive force of floor slab 3 is known from the calculation of the slab. However, for floor slab 1, too it is necessary to take into consideration a support reaction at its cross edge.

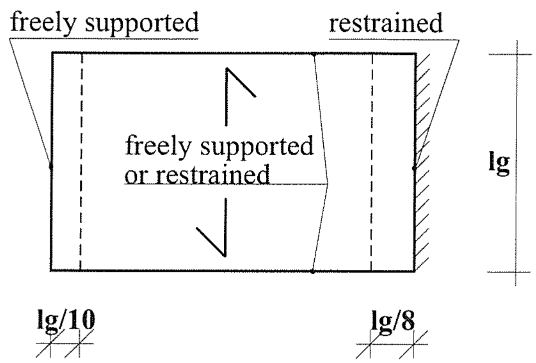


figure 8.5.2.3.a Support reactions at the cross edge of a 3D slab. The kind of support at the longitudinal edge (restrained or simply supported) has no important influence on the support at the cross edge.

load	
dead load = 0.21 · 0.15 · 25	= 0.79 kN/m
floor slab 1 = 4.15 · 6.50 / 8	= 3.37 kN/m
floor slab 3	= 8.78 kN/m
TOTAL LOAD	≈ 13.00 kN/m

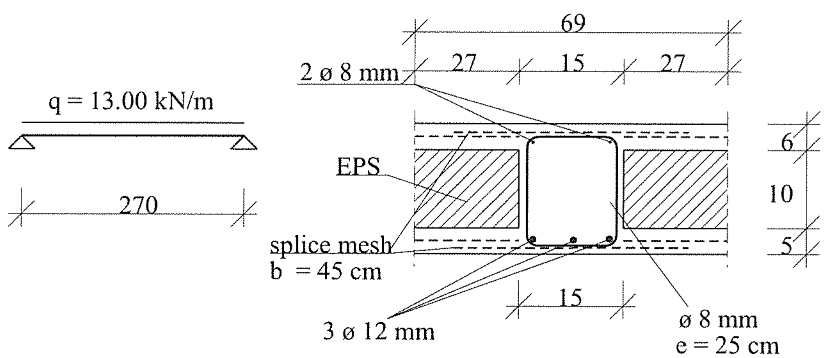


figure 8.5.2.3.b System drawing and reinforcement of the flush beam strip

The internal forces due to the uniformly distributed load can be calculated as follows:

$M = ql^2/8 \quad = 11.85$ kNm
$V = ql/2 \quad = 17.55$ kN

The moment is still well within the admissible range. The calculation results in

$$As = \frac{1.75 \cdot 11.85}{0.9 \cdot 0.17 \cdot 50} = 2.71 \text{ cm}^2 \qquad (3\ ø\ 12,\ As = 3.39 \text{ cm}^2)$$

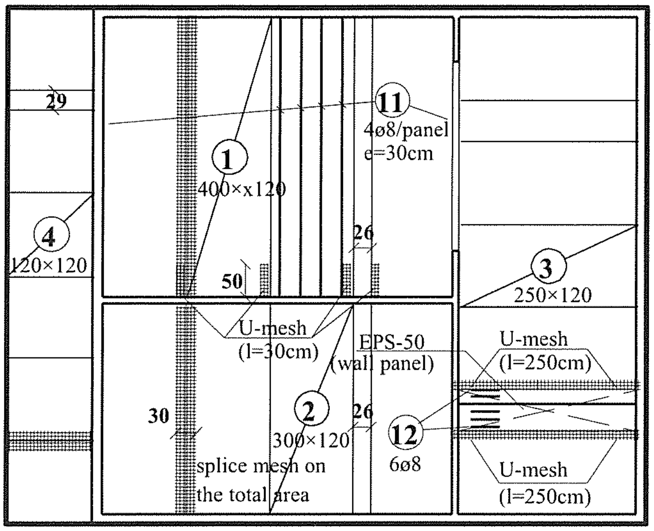
Thus shear stress results in

$$\tau = \frac{17.55}{15 \cdot 0.95 \cdot 17} = 0.072 \text{ kN/cm}^2 \rightarrow \text{ min. shear reinforcement acc. to DIN required}$$

$$as = \frac{1.75 \cdot 0.4 \cdot 7.2 \cdot 25}{50} = 2 \times 1.26 \text{ cm}^2/\text{m} \qquad (ø\ 8,\ e = 25 \text{ cm},\ as = 2 \times 2.01 \text{ cm}^2/\text{m})$$

8.5.3. Panel Layout

The cutting list includes only 3D panels with 100 mm EPS. Panels with 50 mm EPS are not mentioned in the list since we are dealing with one wall panel with 100 diagonals per m² only.



CUTTING LIST (EPS100, 200 DIAG/M²)

item	pcs	length	width	area
1	4	4.00	1.20	19.20
1a	1	4.00	0.26	1.04
2	4	3.00	1.20	14.40
2a	1	3.00	0.26	0.78
3	5	2.50	1.20	15.00
3a	1	2.50	0.60	1.50
4	6	1.20	1.20	8.64
4a	1	1.20	0.29	0.35
TOTAL AREA [M²]				60.91

ORDERING LIST

pcs	lengthxwidth	EPS	diag	area
5	4.00x1.20	100	200	24.00
4	3.00x1.20	100	200	14.40
9	2.50x1.20	100	200	27.00
TOTAL AREA [M²]				65.40

figure 8.5.3.a Layout example (panels and additional bottom reinforcement)

The reinforcement elements at the support – stirrups and straight bars – are not shown in the panel layout in figure 8.5.3.a.

8.5.4. Top Reinforcement

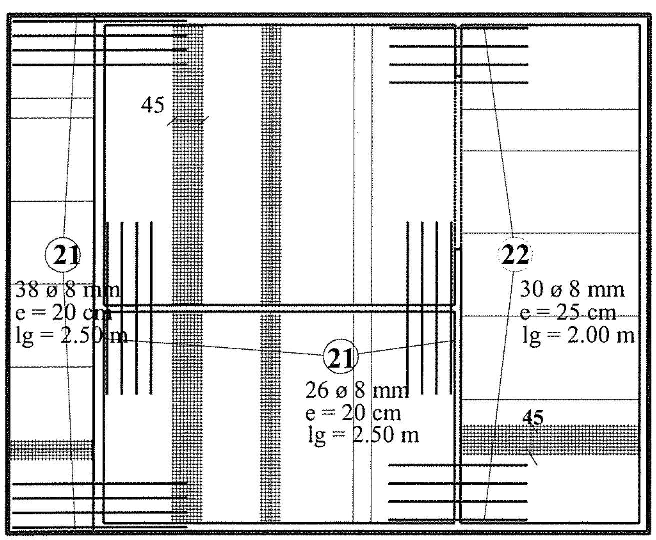


figure 8.5.4.a Negative reinforcement

Joints in the top reinforcement have to be covered with a 45 cm wide splice mesh only if there are negative moments or concentrated loads. Generally a 30 cm wide splice mesh is sufficient. In general, the rebars of this example were laid centrally above the support.

8.5.5. Framing of Openings

When designing the reinforcement of the framing it is necessary to take into account both the moment and the shear force. In general it is sufficient to place the reinforcement missing in the area of the opening on the left and on the right edge.

Shear force in the corner of an opening is considered analogously to chapter 7 (concentrated loads). The effective width of the slab can be assumed to be 1/10 of the slab's span on both sides of the opening. For the calculation of the additional reinforcement, both the transverse force and the moment are determined in this strip with a width of $b = 2 \cdot 0.1 \cdot$ length. For most cases shear reinforcement will become necessary in the vicinity of the support. Often it may be possible to do without them in the center of the floor slab.

For larger openings it is necessary to take into account also the reinforcement in cross direction. Owing to the fact that the loads in cross direction are carried by the top concrete layer only, mostly a flush beam strip will be necessary. For small openings it is possible to omit them.

The simplest shear reinforcement is a U-shaped splice mesh that forms the edge of the opening. If a strong reinforcement is required it is possible to use also a conventional stirrup cage.

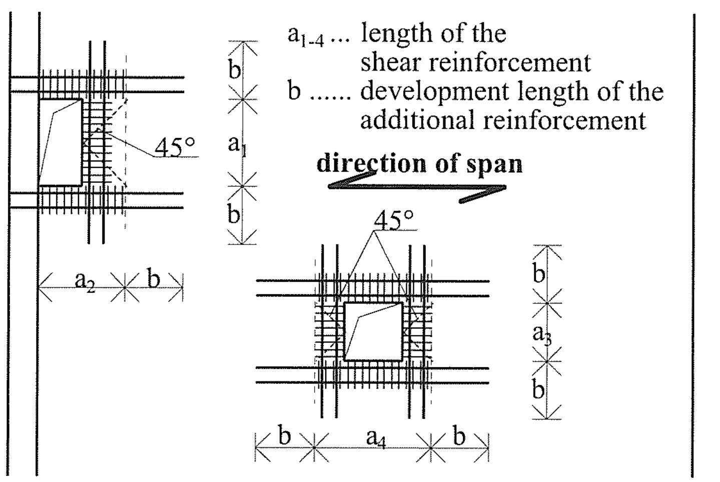


figure 8.5.5.a Framing of slab openings

Figure 8.5.5.a shows 2 slab openings. The first one is located right next to a wall, the second one lies in the center of the floor slab. The additional shear reinforcement must run in the direction of span and should go beyond the end of the opening. When assuming a direction of distribution of the stresses in the concrete to be 45°, this additional length corresponds to the opening's half width. From this point on the additional longitudinal reinforcement needs to be anchored, as well. This development length depends on the kind of reinforcement and on concrete grade. It is laid down in the individual local standards. However, it must always be taken into consideration that the quality of bond between the reinforcement and a thin shotcrete layer is less than for a conventional concrete slab. For ribbed rebars, 80 – 100 times the diameter is recommended. When using smooth rebars anchoring without hooks is not allowed.

8.6. Roof Slab Mechanism

For the example of figure 8.5.1.a, a roof slab of 3D panels is to be made. The overhang is 60 cm on all sides. The roof inclination is 25°. Due to the minor load, panels with 50 mm EPS are used. A concrete layer of 50 mm is applied at both sides.

The two roof halves often are constructed in such a way that they support each other like in a three-hinged arch. The occurring horizontal forces are transferred via the cross walls (y-direction). The roof acts also as a diaphragm and possibly has to be provided with additional reinforcement in transverse direction. As a recommendation the panel splices in the lower third of the slab and in the area of the inner supports have to be overlapped on both sides with 45 cm wide splice mesh.

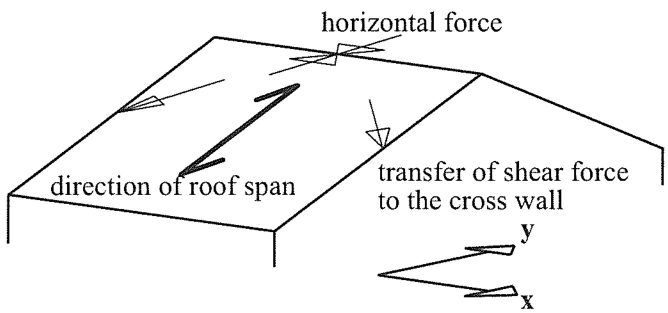


figure 8.6.a Load transfer mechanism in a roof slab

8.6.1. System

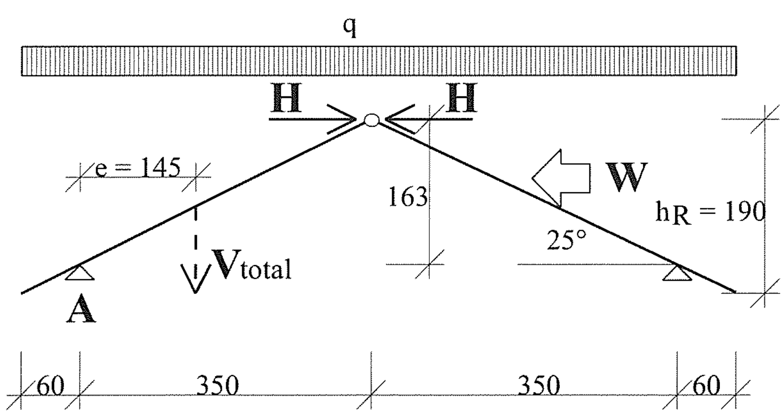


figure 8.6.1.a System drawing of a roof slab (y-direction)

load			
concrete = 50 + 50 mm		=	2.50 kN/m²
roof tiles		=	0.50 kN/m²
	D.L.	=	3.00 kN/m²
live load	L.L.	=	1.00 kN/m²
total load			4.00 kN/m²

8.6.1.1. Horizontal Wind Load

Wind pressure q_W is assumed with

$$q_W = 1.0 + 0.5 \text{ kN/m}^2 = 1.5 \text{ kN/m}^2 \text{ (pressure + suction)}$$

The horizontal wind load acting upon the roof results in

$$W = q_W \cdot h_R = 1.5 \cdot 1.9 = 2.9 \text{ kN/m}$$

8.6.1.2. Horizontal Force

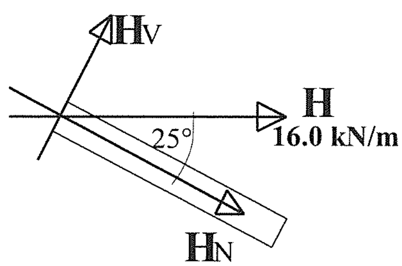
Horizontal force H is acting in the roof ridge. It is calculated by adding up the moments around point A. V_{total} is the total load of one side of the roof. H_l is the horizontal force without considering wind load.

$$\Sigma M_A = H_l \times h - V_{total} \cdot e = 0$$

$$H_l = 4.00 \times 4.10 \times 1.45 / 1.63 = 14.6 \text{ kN/m}$$

The horizontal force H in the ridge is composed by force H_l and half of the wind load W. Hence H results from

$$H = 2.9/2 + 14.6 = 16.0 \text{ kN/m}$$



$$H_V = H \cdot \sin(25°) = 6.8 \text{ kN/m}$$

$$H_N = H \cdot \cos(25°) = 14.5 \text{ kN/m}$$

figure 8.6.1.2.a The horizontal force in a roof slab

8.6.2. Diaphragm Action of a Roof Slab

The roof slab has to transfer the horizontal forces from the ridge to the cross walls (internal walls and gable walls). To do so it has to work like a beam supported by these cross walls. This beam is subject to moments and shear forces in plane of the slab. The load corresponds to force H_N in figure 8.6.1.2.a. The influence of the roof overhang (60 cm) is negligible.

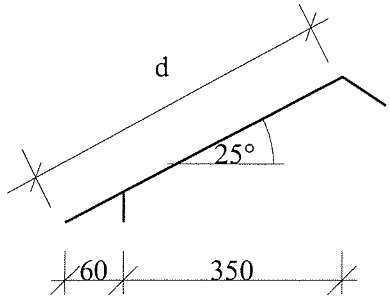


fig 8.6.2.a Effective depth of the roof slab

$d = (3.5 + 0.6) / \cos(25°) = 4.50$ m

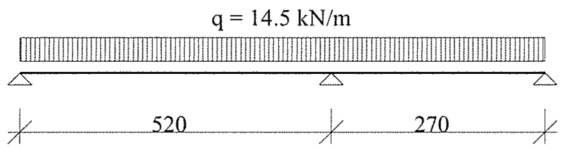


figure 8.6.2.b System drawing of the roof slab (x-direction)

ratio depth / length = 450/520 = 0.87 $\rightarrow$ i.e. calculation as deep beam.

```
CONTINUOUS BEAM, ROOF SLAB
concrete grade B25   steel grade ST500
h = 455.0 cm     d,t = 450.0 cm    d,b = 450.0 cm


MOMENTS
M,min    [kNm/m]                          -36.78
                  |-------part 1--------|-------part 2--------|
M,max    [kNm/m]         32.35                  1.22


SHEAR FORCES (basic value = 0.0 kN/m)
V,max    [kN/m]    30.63                     33.20
                  |-------part 1--------|-------part 2--------|
V,min    [kN/m]                    -44.77                  -5.95
```

8.6.2.1. Flexural Design

The calculation of flexure is necessary for the first part with a span of 5.20 m only. The lever arms of internal forces are calculated according to table 5.2.b.

$z_B = 0.5 \cdot 4.5 \cdot (1.9 - 4.5/5.2) = 2.33$ m (lever arm for bottom reinforcement)
$z_T = 0.45 \cdot 2.7 = 2.03$ m (lever arm for top reinforcement)

$As_{BOTTOM} = 1.75 \cdot 35.47 / (2.33 \cdot 41.2) = 0.65$ cm²
$As_{TOP} = 1.75 \cdot 40.33 / (2.03 \cdot 41.2) = 0.84$ cm²

In the area of the main tensile reinforcement at midspan (= 0.10 lg, see figure 5.2.3.a) there are:

$$As_{Panel} = 2 \times 0.1 \cdot 5.20 \cdot 1.41 = 1.47\ cm^2 > 0.65\ cm^2 \rightarrow$$ i. e. no additional reinforcement is required.

In the area of the main tensile reinforcement above the inner support (= 0.40 lg) there are:

$$As_{Panel} = 2 \times 0.4 \cdot 2.70 \cdot 1.41 = 3.05\ cm^2 > 0.84\ cm^2 \rightarrow$$ i. e. no additional reinforcement is required.

The roof slabs do not require any additional horizontal reinforcement due to diaphragm action. As a general recommendation the necessity of a considerable amount of additional reinforcement is a sign of insufficient construction. In this case additional internal supports are required to shorten the length of span. Mostly it is sufficient if the roof slab is wide enough to be considered as a deep beam.

8.6.2.2. Reaction Force at Support

The reaction force in the support of the roof slab generated by the diaphragm action is transferred to the walls in y-direction (gable walls and internal walls) via the panel joint. It is allowed to neglect the calculation of in plane shear strength of the roof slab. However, the critical section is the connection between the roof slab and the cross walls. Thereby the force in plane of the slab has to be transferred completely by the L-shaped splice mesh. The shear strength of the reinforcement is assumed to be 60 % of the specified yield strength ($fs = 0.6 \cdot 50 = 30\ kN/cm^2$).

$$Vmax = 44.77\ kN$$
$$As = 1.75 \cdot 44.77 / 30 = 2.61\ cm^2$$

Thus, on each side of the wall, the minimum length of the connection reinforcement is 2.61/1.41 = 1.63 m. The length of the wall is 3.80 m. Therefore, the provided connection reinforcement is more than required.

9. Walls

For the notation see chapters 2 to 7.

In general, walls made from 3D elements can be considered as load-bearing walls. The design of a wall with regard to axial forces can be carried out according to DIN 1045 or ACI 318 (see chapter 4). The basic requirements for a load-bearing 3D wall are:

- Slenderness λ should not exceed 70.
- The concrete's cement content should be about 300 kg/m³.
- The concrete shells (concrete layers) must have a minimum thickness of 40 mm (inside) or 50 mm (outside).

Except for a few cases it is necessary to consider the wall to be simply supported. The connection reinforced to the slab is not designed for flexure. A restraining moment would cause a frame action of the construction. Generally 3D walls are not designed to take such high moments. The necessary additional reinforcement would neutralize the advantages of a 3D panel and, all in all, it would entail higher costs. The minor moments that can be transferred via the usual reinforcement to the slab's support can be considered as an additional eccentricity of the axial load (see chapter 4).

9.1. Determination of Buckling Length and Eccentricity

9.1.1. Requirements

The crux of wall design is the determination of the wall's buckling length (effective length). For the most simple example one of the 4 basic cases according to Euler applies. The cases 2 and 3 hardly occur in practice. Mostly a mean value between restrained and hinged supports will be applicable.

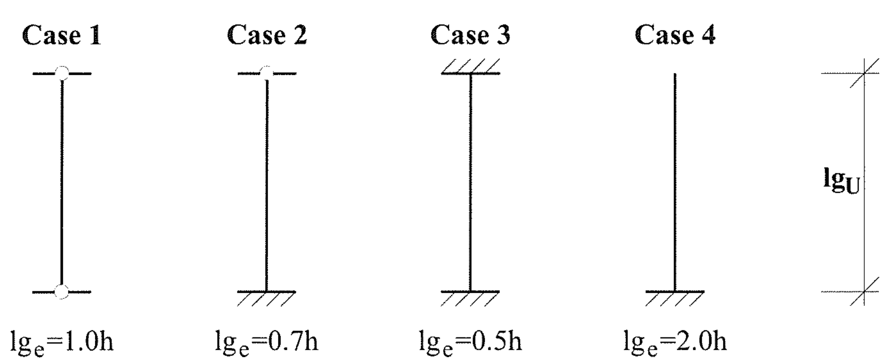


figure 9.1.1.a Support conditions for walls

Primarily the cases 1 and 4 are important for 3D walls. Buckling length lg_e in these cases results in

case 1 $lg_e = 1.0 \cdot lg_U$
case 4 $lg_e = 2.0 \cdot lg_U$

For this buckling length, slenderness

$$\lambda = lg_e/r$$

has to be determined. The value *r* refers to the wall's radius of gyration. The diagrams of section 4.6 include also the radii of gyration for the most commonly used 3D cross sections. The approximation method according to DIN 1045 may be applied only for cases with a wall slenderness of $\lambda \leq 70$. If actual slenderness is bigger than 70, there are several ways of design to solve this problem without increasing the cross section. For minor loads see section 4.2.2.

Another parameter to use the diagrams is eccentricity *e*. The eccentricity refers to the distance between the point of load application and the center of gravity of the cross section. This eccentricity implies

- planned bending moments
- unintended restraints

For 3D walls it is possible to reckon with the following minimum eccentricities on account of unintended restraints:

external walls: h/6
internal walls: h/8

In this formula *h* refers to the total thickness of the wall. These data apply for average conditions. However, the values have to be adapted for very favorable circumstances (e.g. external wall under a big cantilever slab) or for very unfavorable circumstances (e.g. internal wall with considerably varying spans of the floor slabs supported). For walls higher than 3.0 m the eccentricities should be increased proportionally. Therefore, an eccentricity of 30 mm can be considered to be sufficient in most cases. For walls according to figure 9.1.1.b the acting load has to be assumed to be always in the center of gravity of the loaded concrete shell (mostly inside). In this special case the load-bearing capacity of a wall can be increased noticeably in comparison with a symmetrical wall of the same thickness when using different concrete shell thicknesses.

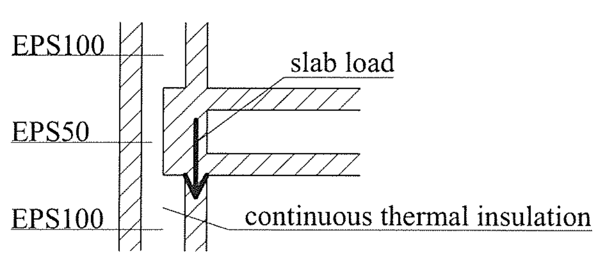


figure 9.1.1.b External wall with continuous thermal insulation. The slab load is transferred to the inside concrete shell only.

Basing on moment M and regular force F the planned eccentricity e_0 results from the structural calculation:

$$e_0 = M/F$$

The method of approximation can be applied only if the entire eccentricity reaches to the center of the two concrete shells at most. Maximum eccentricity is achieved in case of a wall with a continuous thermal insulation and, therefore, with a load acting on one side only. That means, no planned bending moments are allowed.

9.1.2. Flexurally Rigid Connections

Owing to the flexurally rigid connections between the wall and the neighboring components (slab or foundation, figure 9.1.2.a) it is possible to reduce buckling length to 60 - 80 % of the wall's height. The actual buckling length depends on the stiffness of the components that are connected. In this case, however, the admissible maximum eccentricity will be exceeded. The wall would have to be designed as a frame construction. A positive influence on the load-bearing behavior of the wall can be reckoned with only if the floor slabs supported by the wall have very small spans. Applying such a procedure without an exact calculation of internal forces seems to be safe only in case of a restraint in a foundation slab having a much higher stiffness than the connected 3D wall. For an approximate calculation of the buckling length it is possible to assume the following buckling length for a 3D wall:

Case: restrained in the foundation slab buckling length $lg_e = 0.9 \times$ unsupported length

For a wall with a topping 3D slab or concrete slab the clear height of the component can be assumed as unsupported length.

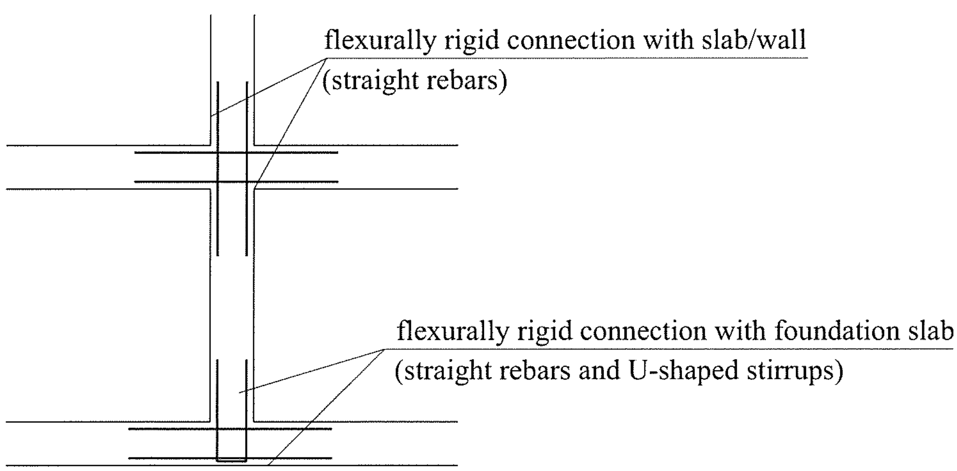
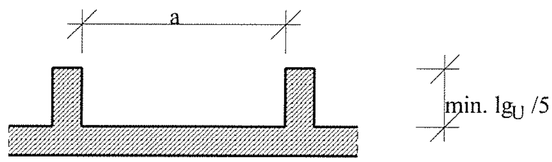


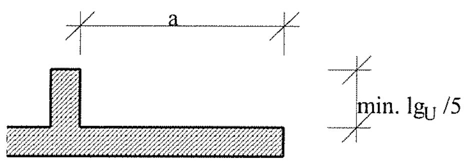
figure 9.1.2.a Flexurally rigid connection of a wall

9.1.3. Crosswalls

Buckling length lg_e of a 3D wall can be shortened also by stiffening walls in cross direction. In the following section the regulations of the Austrian standard are described. Standards of other countries, however, (e.g. German DIN) differ only insignificantly. The stiffening walls have to be constructed as 3D walls or as concrete walls, and the maximum spacing in between must be 8 m (or 12 m if there is a 3D slab or a reinforced concrete slab on top). The length of the stiffening wall must be at least 1/5 of the height.



$$lg_e = lg_U \cdot \frac{a}{\sqrt{lg_U^2 + a^2}}$$



$$lg_e = lg_U \cdot \frac{3a}{\sqrt{lg_U^2 + (3a)^2}}$$

figure 9.1.3.a Cross stiffening of walls

In both cases lg_U refers to the unsupported length of the wall without cross stiffening. If there are openings that exceed 1/3 of the wall's height (figure 9.1.3.b) this wall must be considered discontinued. Therefore, we are dealing with a free wall end according to figure 9.1.3.a (bottom).

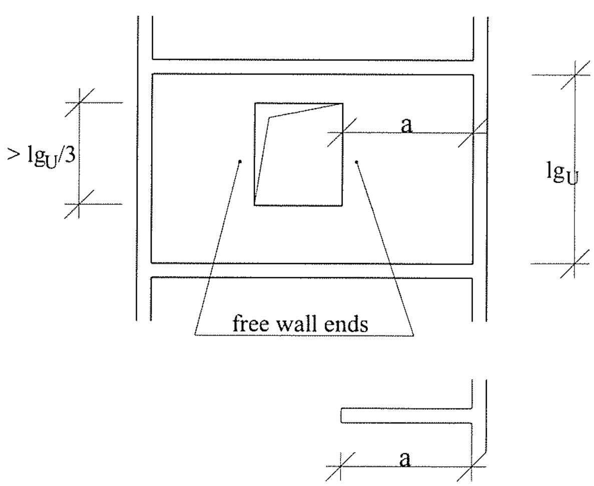


figure 9.1.3.b Free wall ends next to a window opening

9.2. Design of walls

The first step of designing is the determination of a wall's critical section. In most cases this is a free-standing wall pillar between two doors or a wall section right next to an opening. In these areas concentrated loads from the floor above are applied. For this heavily loaded area of the wall a distribution width for the transfer of the load is assumed. In a wall without openings the admissible load is hardly exceeded.

9.2.1. Loads

The wall's load includes both reactive forces acting from the storeys above and the dead load of the wall.

9.2.1.1. Floor loads

In general, the loads resulting from floor slabs can be seen from the structural calculation. For varying spans of the neighboring floor bays it is important that the reactive forces do not fall below a minimum value.

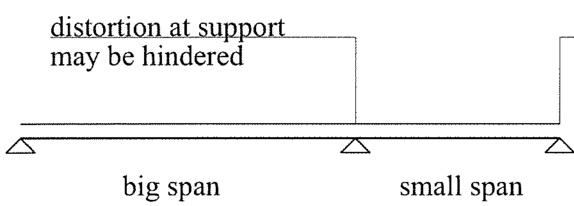
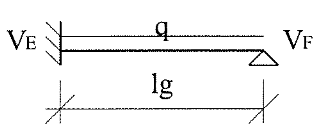


figure 9.2.1.1.a Sketch of a continuous slab

Owing to the influence of the bigger neighboring span the outside support of the smaller floor slab has a very small reactive force in case of a continuous slab according to figure 9.2.1.1.a. This calculation, however, requires that the slab at the inside support can be distorted unhindered. If distortion is obstructed, the reactive forces of figure 9.2.1.1.b apply. By analogy, this restriction was taken into consideration also for slab design as shown in section 8.5.

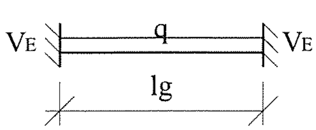
The minimum limit of reactive force results from figure 9.2.1.1.b.



$$V_E \geq \frac{5}{8} \cdot q \cdot lg$$

$$V_F \geq \frac{3}{8} \cdot q \cdot lg$$

figure 9.2.1.1.b Minimum reactive forces



$$V_E \geq \frac{1}{2} \cdot q \cdot lg$$

Most computer programs for the calculation of continuous beams don't take this restriction into account.

9.2.1.2. Wall Loads

For the determination of the wall's dead load the areas of big wall openings should be deducted only. Smaller and mid-sized openings are to be included in the wall's weight to compensate some inaccuracies.

- The window reveals may have a considerable concrete thickness to compensate inaccuracies occurring during panel erection.
- Owing to the fact that it is impossible to align the wall panels in an exact plane, the actual concrete layer will always be thicker by a couple of millimeters than the concrete layer assumed for structural analysis.

9.2.2. Distribution Width

For the calculation of the wall it is assumed that the concentrated vertical loads from the storeys above are transferred in an angle of 45°. Owing to the fact that the continuous reinforced 3D wall has a very good distribution effect it is possible to assume the value $lg_U/2$ to be the maximum distribution width for a concentrated load, i.e. the theoretical distribution of load is ensured up to the wall's midspan (see figure 9.2.2.a). In this formula lg_U refers to the clear height of the wall.

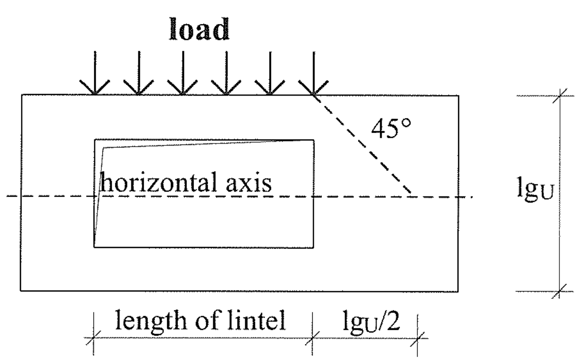


figure 9.2.2.a Elevation of a wall

$lg_U/2$ = distribution width of the load above the opening

9.2.3. Strengthenings

An increase of load-bearing capacity can be achieved by additional reinforcement. However, in case of a 3D wall this is not a practical solution. In order to prevent these additional reinforcement from buckling it would be necessary to enclose them by horizontal rebars. Usually the concrete layers of a 3D wall are rather thin, and, as a consequence, it is hardly possible to install these two reinforcement layers. The additional vertical reinforcement of figure 9.2.4. is necessary for construction purposes only.

If a wall does not meet the structural requirements, the strengthening of the concrete will be the only practical solution, i.e.

- thicker concrete layer
- higher concrete grade

In addition, it is possible to install a traditional reinforced concrete column at very critical sections. Owing to the fact that a higher concrete grade must be used for the entire storey and that a reinforced concrete column would lead to problems with thermal insulation, it is advisable to use a panel with a thinner EPS core (e.g. 50 instead of 100 mm) in this area without changing the wall's overall thickness. Besides, it will be difficult to achieve a higher concrete grade than B25 (fc = 17.5 N/mm²). For this case the concrete shells may have either the same thickness (e.g. 2 × 75 mm) or different thicknesses (e.g. 50 + 100 mm). In case of an external wall the thicker layer is required for inside, of course. If the wall is calculated with minimum eccentricity according to section 9.1.1 an asymmetrical wall design, however, will bring about a slight increase of the load-bearing capacity since the influence of the bigger compression zone is partly compensated by the decreasing maximum eccentricity and the slightly increasing slenderness.

9.2.4. Arrangement of Reinforcement

The additional horizontal reinforcements above the window openings result from the calculation of the lintel. The splice meshes in the window corners are required to avoid 45° cracks in this area. The vertical rebars are only necessary to replace the cover mesh, for instance, that was removed for plumbing, fitting and wiring (e.g. switches). The horizontal rebars are to be installed according to the structural analysis (see chapter 5).

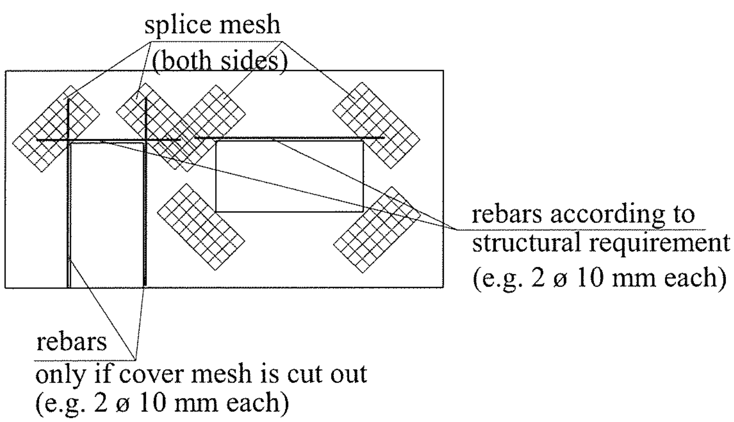


figure 9.2.4.a. Typical wall reinforcement with splice meshes in the corners of openings.

9.3. External Wall

The following example shows the calculation of the external wall of a 3-storey residential building. The load of the slab is assumed to be 6.75 kN/m². The wall consists of 3D panels with a 100 mm EPS core and 2 × 50 mm concrete layers with concrete grade B25 (fc = 17.5 N/mm²).

9.3.1. System

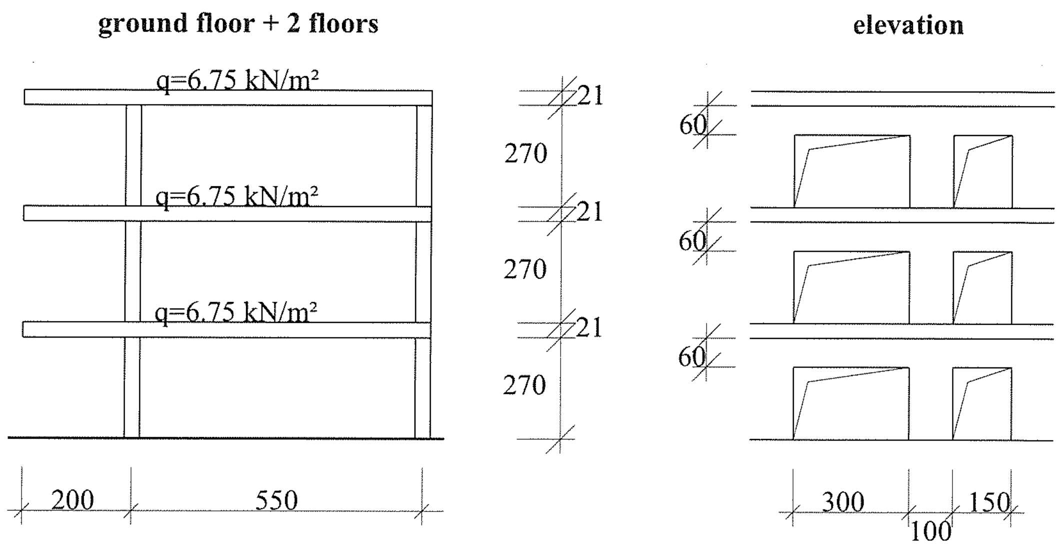


figure 9.3.1.a Cross section of the building and elevation

9.3.2. Loads

In practice, the load of the wall can be determined according to the reactive forces of the slab's calculation. However, for this example an approximate determination would be more useful.

The load per slab results in
$F_{slab} = 6.75 \times (2.00 + 0.55 \cdot 5.50) \quad = \quad 33.9$ kN/m

The load per wall results in
$F_{wall} = 25\ kN/m^3 \cdot (0.05 + 0.05\ m) \cdot 2.91\ m \quad = \quad \underline{7.3\ kN/m}$
$F_{storey} \quad = \quad 41.2$ kN/m

$\mathbf{F_{total} = 3 \times 41.2 \quad = 123.6\ kN/m}$

Thus, the total load acting on the pillar in the ground floor results in

$F_{req} = 123.6 \times (3.00/2 + 1.00 + 1.50/2) \quad = \quad \mathbf{402\ kN}$

9.3.3. Design

The following input values are necessary for the diagrams of section 4.6:

$h = 50 + 100 + 50$ mm
$\lambda = 2700/76.4 = 35.3$
$e = 30$ mm

According to diagram 4.6.6.a the admissible wall load is

$$F_{adm} = 195 \times 17.5 / 10.5 = 325 \text{ kN} < F_{req} \qquad \text{not OK}$$

This shows that the wall does not meet the requirements. A solution would be a wall pillar with an EPS thickness of only 50 mm but the same total thickness.

$h = 50 + 50 + 100$ mm
$\lambda = 2700/64.0 = 42.2$
$e = 30$ mm

According to diagram 4.6.4.a the admissible wall load is

$$F_{adm} = 250 \times 17.5 / 10.5 = 417 \text{ kN} > F_{req} \qquad \text{OK}$$

9.3.4. Door Lintels

In the external wall there are two door lintels with a total height of

$$h = 60 + 21 = 81 \text{ cm}$$

The effective depth can be assumed to be 76 cm.

Load		
slab load (see 9.3.2)	=	33.9 kN/m
dead load = 0.6·0.1·25	=	1.5 kN/m
total load	=	35.4 kN/m

lintel 1 **lintel 2**

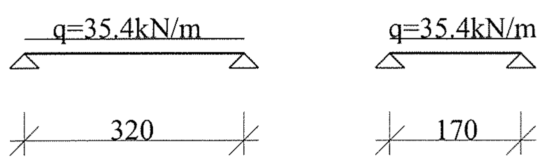


figure 9.3.4.a System drawing of the door lintels

If they are assumed to be freely supported, both lintels can be designed as beams according to section 5.1. The ratio between panel height and effective depth is

$$d_p/d = 55/76 = 0.72$$

According to table 5.1.a the admissible moment results in

$$M = 80.6 \cdot 0.76^2 \cdot 0.30 = 13.97 \text{ kNm}$$

The moments at midspan result in

$$M_1 = 35.4 \cdot 3.20^2/8 = 45.31 \text{ kNm}$$
$$M_2 = 35.4 \cdot 1.70^2/8 = 12.79 \text{ kNm}$$

Lintel 1 only needs additional reinforcement. Maximum shear force in lintel 1 is

$$V_l = 35.4 \cdot 3.2/2 \qquad = 56.64 \text{ kN}$$

Shear stress results in

$$\tau = \frac{56.64}{2 \cdot 5 \cdot 0.9 \cdot 76} = 0.083 \text{kN/cm}^2 = 0.83 \text{ N/mm}^2 \text{ (shear range 2 acc. to DIN)}$$

When it comes to flexure the panel's reinforcement can be taken into account only in shear range 1 (see section 5.1). Therefore, the flexural reinforcement has to be designed for the entire moment. Thus tension reinforcement results in

$$As = \frac{1.75 \cdot 45.31}{0.9 \cdot 0.76 \cdot 50} = 2.32 \text{ cm}^2 \qquad (3 \text{ ø } 12 \text{ mm} = 3.39 \text{ cm}^2)$$

According to DIN 1045 shear reinforcement has to be designed for a shear stress of

$$\tau = \frac{\tau_0^2}{\tau_{02}} = \frac{0.83^2}{1.80} = 0.38 \text{ N/mm}^2 \text{ } (= 0.038 \text{ kN/cm}^2)$$

As a consequence, the necessary shear reinforcement results in

$$as = 100 \cdot \frac{1.75 \cdot 0.038 \cdot 2 \cdot 5}{50} = 2 \times 0.67 \text{ cm}^2\text{/m} \qquad \text{cover mesh is sufficient}$$

9.4. Internal Wall

The following example shows the calculation of a wall of a 3-storey residential building. The data were chosen analogously to example 9.3. The load of the slab was assumed to be 6.75 kN/m². The wall consists of 3D panels with a 100 mm EPS core and 2 × 50 mm concrete layers with concrete grade B25 (fc = 17.5 N/mm²). The door lintel is not calculated in this example.

9.4.1. System

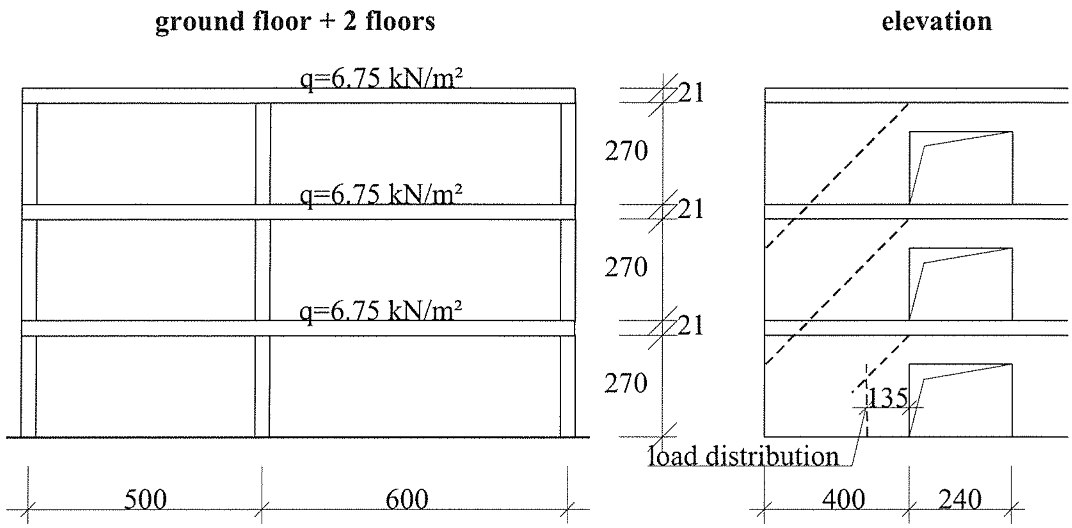


figure 9.4.1.a Cross section of the building and elevation of the internal wall

9.4.2. Loads

The load acting from the slab was determined by means of a computer program for continuous beams. The restriction mentioned in chapter 9.2.1.1 was taken into consideration.

F_{slab}	=	47.4 kN/m
$F_{wall} = 0.25\ kN/m^3 \cdot (5 + 5\ cm) \cdot 2.91\ m$	=	7.3 kN/m
F_{storey}	=	54.7 kN/m

The wall loads of the above storeys can be distributed over a width of 4.0 m up to the edge of the wall. For the reactive force of the lintel in the ground floor only there is a distribution width of $lg_U/2 = 1.35$ m. Therefore, the total load for the pillar in the ground floor is

$F_3 = 1.35 \cdot 54.7 \cdot (2.40/2 + 4.00)/4.00$	=	96 kN
$F_2 = 1.35 \cdot 54.7 \cdot (2.40/2 + 4.00)/4.00$	=	96 kN
$F_1 = 54.7 \times (2.40/2 + 1.35)$	=	139 kN
$\mathbf{F_{req}} =$	**=**	**331 kN**

9.4.3. Design

The following input values are required in the diagrams of section 4.6:

$h = 50 + 100 + 50$ mm

$$lg_e = lg_U \cdot \frac{3a}{\sqrt{lg_U^2 + (3a)^2}} = 270 \cdot \frac{3 \cdot 400}{\sqrt{270^2 + (3 \cdot 400)^2}} = 263 \text{ cm}$$ (see figur 9.1.3.a)

$\lambda = 2630/76.4 = 34.4$

$e = 30$ mm In order to compensate uncertainties in multi-storey buildings the same eccentricity as for an external wall is chosen

According to diagram 4.6.2.a the admissible wall load is

$$F_{adm} = 1.35 \cdot 195 \cdot 17.5 / 10.5 = 439 \text{ kN} > F_{req} \quad \text{OK}$$

This shows that the wall meets the requirements. For other cases it would be possible to install at this point also a traditional reinforced concrete column. In contrast to the 3D wall the safety factor of this column can be assumed to be 2.1. The influence of slenderness is shown rather clearly. For a wall with an EPS core of 50 mm and 2×50 mm concrete layers, load-bearing capacity will be about 295 kN only.

9.5. Earthquake Forces

In areas with a high risk of earthquakes 3D walls have to transfer not only vertical loads but also lateral loads to the foundation anchors. The lateral load depends on the building's weight and on a standardized value for horizontal acceleration. For the most favorable case of an area with low seismic risk 1/100 of the building's weight can be assumed as lateral load.

9.5.1. Earthquake Design according to ACI

In quite general terms, the 3D-Construction System may be considered as being a lateral load-resisting system, hence a system capable of sustaining seismic loads and wind loads.

In conformity with chapter 21 of ACI 318, 3D walls may be regarded as "structural walls" (shear walls), i.e. walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. 3D slabs act as "Structural Diaphragms - Structural Members", such as floor and roof slabs, which transmit inertial forces to lateral- force resisting members.

3D constructions are no frame-constructions in conformity with chapter 21.3; the fact is that earthquake-induced forces are sustained by structural walls, diaphragms, and trusses by applying the provisions of chapter 21.6.

Basically it must be said that the steel grade of the cover mesh of 3D elements does not meet the requirements of paragraph 21.2.5 (cover mesh has a steel grade of 500 N/mm² or 70 ksi and the tensile strength/yield strength ratio is 1.10). Nevertheless we believe that chapter 21.6 is applicable when it comes to the proof of the earthquake-resistant design of 3D constructions.

In general, all 3D walls of a building are capable of sustaining lateral loads. In practice, however, only a few walls will be designed as "shear walls", i.e. they will be provided with "boundary elements". As per the wording of chapter 21.1, boundary elements are portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. The reinforcement of the boundary elements which can be seen as reinforced concrete tension or compression chords combined with 3D elements for transferring lateral loads, is implemented with the required ductile steel grades according to paragraph 21.2.5. The ductility of the entire construction and thus the capability of sustaining seismic loads can be attributed, in the first place, to the boundary elements. However, the boundary elements are designed in strict conformity with chapter 21.6.

3D elements - both in structural walls and in diaphragms - fulfill a secondary function only, i.e. the function to transfer the lateral loads towards the boundary elements. As far as the minimum reinforcement (0.25 %) and the minimum concrete layer thickness (50 mm) are concerned, all 3D walls meet the requirements of chapter 21. Besides, it should be noticed that ACI allows variations explicitly if a sufficient loadbearing capacity is prooven by tests (paragraph 21.2.1.5).

Figure 9.5.4.b shows two designs of boundary elements. Instead of using a wider element (shown on the right) it is possible to lengthen the boundary element. The length of the boundary element, however, should not exceed 10 % of the total wall length.

9.5.2. Loads

For a multi-storey building it has to be taken into consideration that the lateral force increases in the upper storeys. Thus it will be necessary to distribute the entire horizontal force unevenly. The following method to distribute the lateral forces is widely used in a lot of codes and standards.

$$H_i = \frac{G \cdot h}{\sum_{i=1}^{n} Gi \cdot hi} \cdot \alpha \cdot \sum_{i=1}^{n} Gi \cdot (1-\delta_n)$$

i.e.
- Hi....... lateral force on storey i
- G........ weight of the storey
- h......... height of the floor level above ground
- α........ horizontal acceleration
- n......... number of storeys
- δ_n....... 0 (in the worst case)

Drawing up a table is the best way to determine these values. First, it is necessary to determine the total weight of a storey. According to the example of figure 9.5.2.a horizontal acceleration is assumed to be 1 m/sec² = 0.10 g.

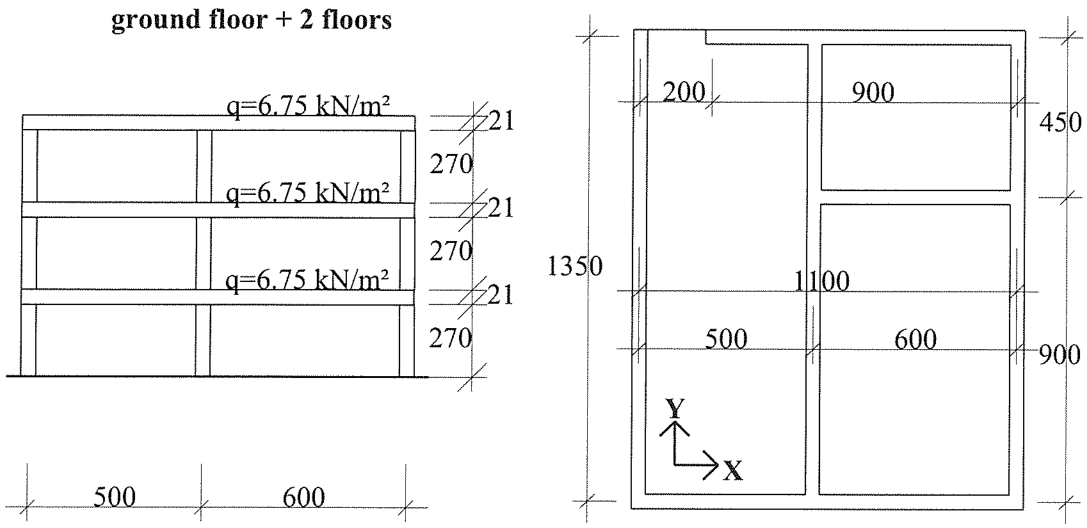


figure 9.5.2.a Section and ground plan

This example refers only to the transfer of forces in direction X. The weight of the storey is determined by means of the dimensions of the walls' axes without taking into account wall or slab openings. The following calculation shows the design according to ACI. Therefore the loads are multiplied with the corresponding safety factors (dead load = 1.4; live load = 1.7).

$$
\begin{array}{lcr}
G_{slab} = 13.50 \cdot 11.00 \cdot (1.4 \cdot 4.75 + 1.7 \cdot 2.00) & = & 1492 \text{ kN} \\
G_{wall} = 1.4 \cdot 2.50 \cdot 2.91 \cdot (3 \cdot 13.50 + 2 \cdot 11.00 + 6.00) & = & \underline{698 \text{ kN}} \\
\mathbf{G_{storey}} & \approx & \mathbf{2200 \text{ kN}}
\end{array}
$$

Thus the total vertical and horizontal loads result in

$$
\begin{array}{lcr}
G_0 = \Sigma G = n \cdot G = 3 \cdot 2200 & = & 6600 \text{ kN} \\
H_0 = \Sigma H = \alpha \cdot \Sigma G = 0.10 \cdot 6600 & = & 660 \text{ kN}
\end{array}
$$

All other calculations are made in a table:

storey	G [kN]	h [m]	G × h	Go [kN]	Hi [kN]
3	2200	8.73	19206	660	330
2	2200	5.82	12804	660	220
1	2200	2.91	6402	660	110
total	6600	-	38412	-	660

table 9.5.2.a List of loads

9.5.3. Internal Forces

By means of the horizontal loads H_i (see 9.5.2) it is possible to give an overview of the shear force and the moments. Figure 9.5.3.a shows the moments and the shear forces.

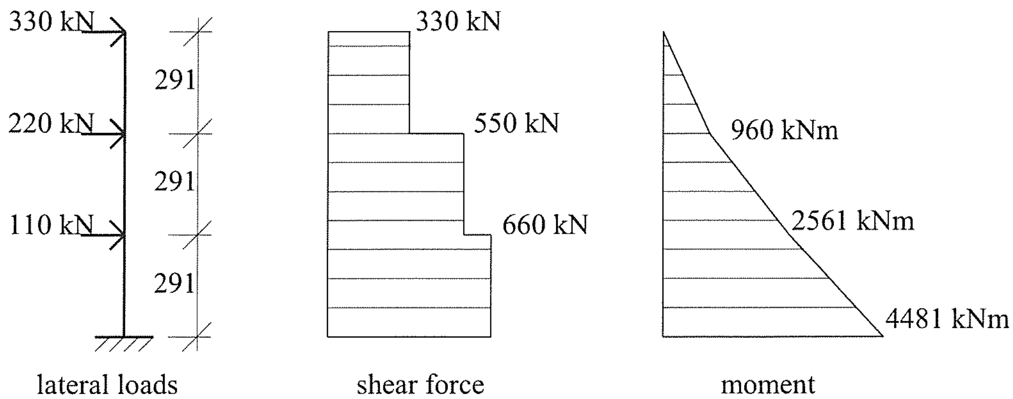


figure 9.5.3.a Internal forces due to lateral loads

The internal forces shown in figure 9.5.3.a have to be transferred to the individual crosswalls. For tall buildings the share of the individual walls corresponds roughly to the ratio of the moments of inertia. For this calculation it is necessary to split up a wall interrupted by doors into the corresponding number of smaller walls. For the sake of simplicity a door opening is assumed only for one of the 3 walls (see figure 9.5.2.a).

In case of 3D walls having the same thickness it is possible to reduce the moment of inertia I' to

$$\begin{array}{ccccccc} \Sigma I' = 11.0^3 & + & 6.0^3 & + & 9.0^3 & = & 2276 \text{ m}^3 \\ \downarrow & & \downarrow & & \downarrow & & \downarrow \\ 58.5\% & & 9.5\% & & 32.0\% & & 100.0\% \end{array}$$

Therefore, in relation to the area it is the longest wall that is loaded most. For the first wall (length = 11.0 m) the internal forces are

$$V = 0.585 \cdot 660 = 386 \text{ kN}$$
$$M = 0.585 \cdot 4481 = 2621 \text{ kNm}$$

9.5.3.1. Walls without Boundary Elements

The determination of shear strength can be made analogously to the example of section 4.4 according to the ACI regulations. In case of earthquakes the effect of the concrete on total shear strength has to be neglected. Therefore, shear strength results in

$$Vs = As \cdot fy \cdot d = 2 \cdot 1.41 \cdot 50 \cdot 0.8 = 112.8 \text{ kN/m}$$
$$Vwu \leq \phi \cdot Vs \cdot lg = 0.85 \cdot 112.8 \cdot 11.0 = 1054.7 \text{ kN} > 386 \text{ kN} \qquad \text{OK}$$

The minimum moment capacity of a wall with uniformly distributed reinforcement can be assumed according to table 5.1.a.

$$M = 80.6 \text{ kN/m} \cdot 0.30 \cdot 11.0^2 = 2926 \text{ kNm} > 2621 \text{ kNm} \qquad \text{OK}$$

In fact, however, we are dealing with a moment plus an axial force in this case. The axial force has a positive influence on the result. In addition, it is possible to include also the influence of the walls in y-direction, if necessary. They act in a composite cross section as compression chord and tension chord. Therefore, the total result lies clearly within the admissible range.

9.5.3.2. Walls with Boundary Elements

Owing to the restrictions in some standards (see section 9.5.1) the load-bearing capacity of the uniformly distributed reinforcement of the panel must be included only in areas with a low or medium risk. For areas with a high risk of earthquakes it will be necessary to find another solution. Strengthened boundary elements would provide an option.

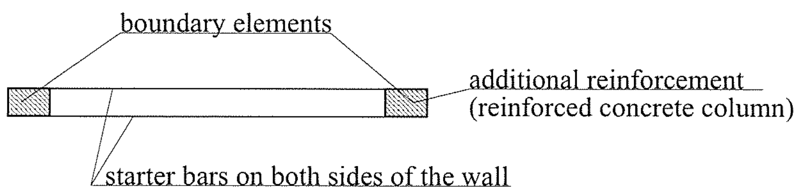


figure 9.5.3.2.a Wall design in areas with a high earthquake risk

It is recommended to use generally boundary elements at the edge of a shear wall (figure 9.5.4.b) in areas with a high earthquake risk. Their influence on the flexural strength can be determined according to traditional methods. In the following example a boundary element with 8 ø 12 mm is used. Reinforcement is distributed over a depth of 40 cm.

$As = 9.05\ cm^2$ (ST500); $d = 11.0 - 0.40/2 = 10.8$ m
$M_U = 0.90 \cdot 9.05 \cdot 50 \cdot 0.9 \cdot 10.8 = 3959$ kNm

As a matter of course it is necessary on both sides of the wall to provide for a tension-proof development for all connecting bars in the foundation. This case given the starter bars must not be considered as reinforcement elements for assembly only.

9.5.4. Practical Design

In practice, problems with earthquake forces occur mainly in multi-storey buildings. Such buildings usually have a regular and, in most cases, also a symmetrical ground plan. External walls of the building and possibly a few internal walls are to be designed preferably as shear walls. Partition walls between apartments serve these purposes in particular.

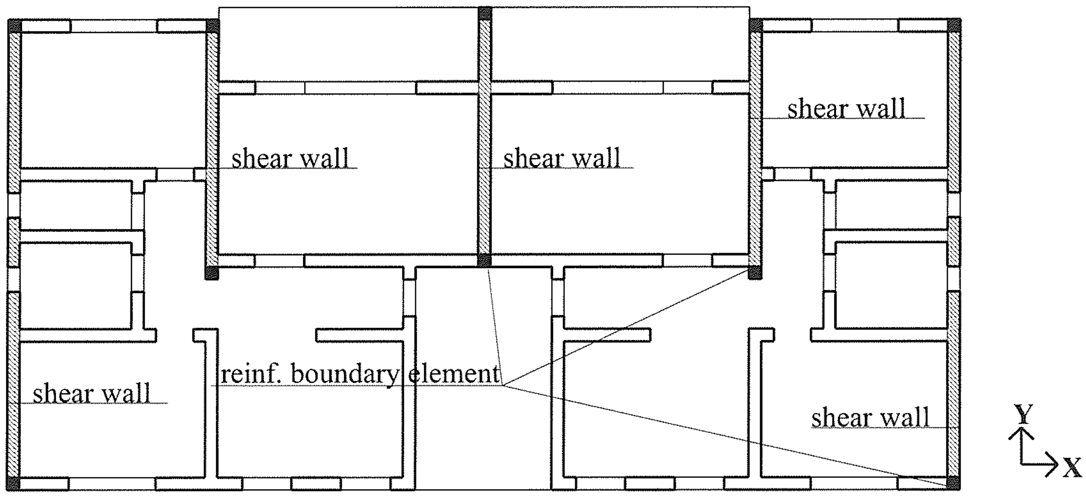


figure 9.5.4.a Schematic layout of shear walls with boundary elements (y-direction)

Figure 9.5.4.a shows a schematic ground plan of a residential building with 2 apartments per storey. The marked shear walls comprise approximately half the area of all walls in direction Y. However, at the same time their share in the building's stiffness amounts to more than 90 %. Therefore, all other walls can be neglected for design without a problem.

All shear walls are to be designed with reinforced boundary elements as shown in figure 9.5.4.b. While the marked internal walls do not have any openings, there are two windows each in the external walls. These areas are to be reinforced by rebars placed at an angle of 45° (figure 9.5.4.c).

Simple design | Strengthened design

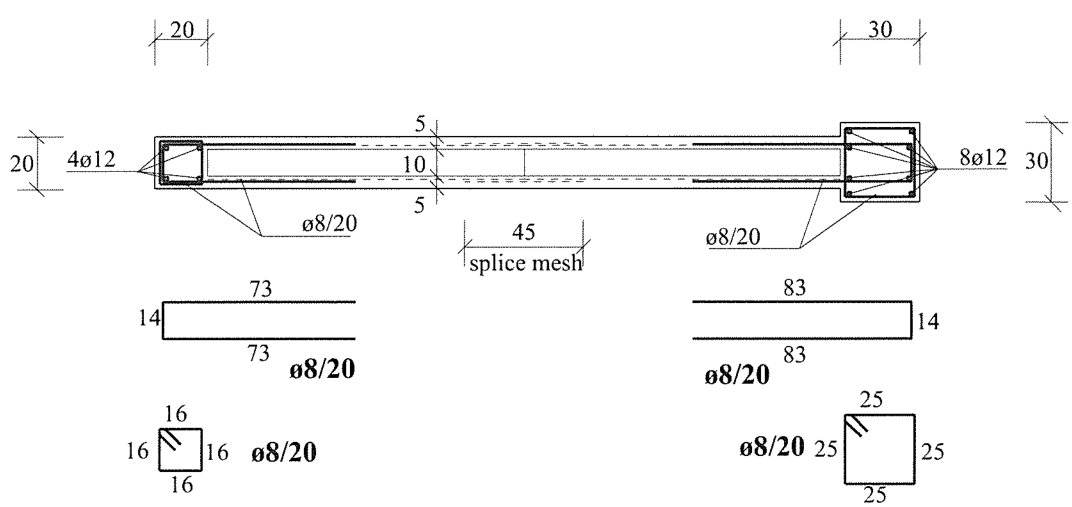


figure 9.5.4.b Boundary elements

Figure 9.5.4.b shows two design options for boundary elements. Instead of using a wider element (shown on the right) it is possible to lengthen the boundary element and to aligne it with the wall. However, the length of the boundary element should not exceed 10 % of the total wall length.

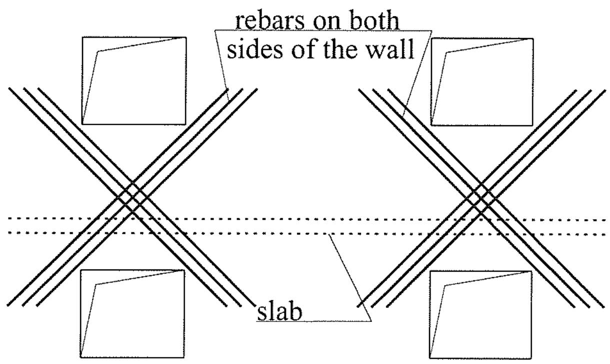


figure 9.5.4.c Additional reinforcement in the area of openings

Figure 9.5.4.c shows the reinforcement pattern between openings. In order to cover this reinforcement sufficiently with concrete, it is necessary to lay rebars between the cover mesh and the EPS. Of course, this cross reinforcement is required on both sides of the wall.

In addition, not only the vertical boundary elements in the walls but also the horizontal ring beam in the slab have to be well reinforced.

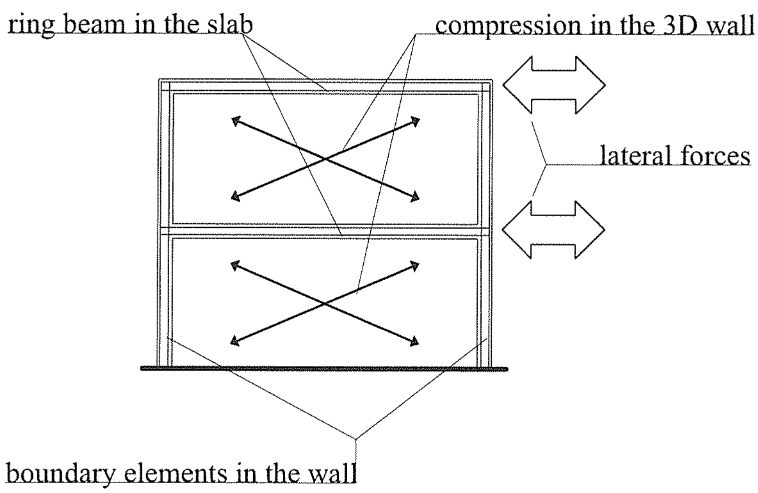


figure 9.5.4.d Scheme of a wall reinforcement in a multi-storey building

10. Prefabricated 3D Constructions

10.1. General

In general, prefabricated components made of 3D panels for slabs and walls have very different design and concrete requirements. The use of prefabricated walls makes sense only if a full-sized concrete layer is applied to both sides. In addition, a thicker concrete layer at the inside of external walls is necessary on account of the requirements in a system using prefabricated slabs. Length of support for slabs should not fall below 10 - 12 cm, if possible. Preferably the entire length of walls should be produced in the precast concrete factory. Then the walls should be transported to the site as a monolithic element. For easier transportation the possible length of such an element is to be restricted to some 12 - 14 m. However, in this case a wall made of conventional (normal weight) concrete would be very heavy. Therefore, the best practicable way seems to be the use of lightweight concrete. With the corresponding concrete thickness it will be possible to compensate also the lower strength of lightweight concrete.

In contrast to the above the requirements for the production of prefabricated slabs are completely different. While it is generally possible to transport wall elements upright due to their shape and height, the shape and size of floor slabs do not allow to transport them as one piece. Therefore, every floor slab has to be divided up into several slab elements with a maximum width of 2 - 2.5 m. Only the bottom side of these elements is concreted in the precast concrete factory (semi-precast element, half slab). Concrete quality of these prefabricated elements can be checked far better in the factory than shotcrete applied on site. Concrete for the slab's top should always be made on site. Thus it is possible to achieve a considerable distribution effect for concentrated loads.

10.2. Prefabricated Walls

10.2.1. Construction Material

In general, lightweight concrete has a lower concrete quality than concrete with normal weight. Especially very light concrete (gas concrete, foam concrete) often has a concrete grade of 5 to 10 N/mm². In this case the wall allways has to be calculated as a plain concrete wall. For concrete with a low quality (up to 10 N/mm²) the safety factor according to DIN 1045 is 0.5 higher than the saftey factor for concrete grade B15 and higher. Therefore, we recommend safety factor 3.5 for this case. Restrictions for the use of lightweight concrete defined in local codes or standards have to be considered separately.

The load-bearing capacity of double-shell walls made of B5.0 shall be examined below. The structure corresponds to figure 10.2.2.b. For working reasons and to ensure protection against corrosion the outside shell should have a minimum thickness of 6 cm.

10.2.2. Connection Details

The load-bearing walls have to be connected with bolts or any other connectors (figure 10.2.2.a) to achieve a stiffening effect. At the same time a recess has to be provided for at the wall element. The recess needs to be poured with concrete after the erection of the walls. This should ensure the wall joints' tightness. For light internal walls it is sufficient to provide for a groove in the wall element. An additional connection by bolts is necessary only if the stiffening wall has to fit special requirements (see section 9.1.3).

A reinforced ring beam is inevitable in the slab to achieve a diaphragm effect. In case of a two-way slab cast in situ it is sufficient to have 2 ø 8 mm as reinforcement of the ring beam. If a prefabricated slab is used, the minimum reinforcement of the ring beam shall be 4 ø 8 mm.

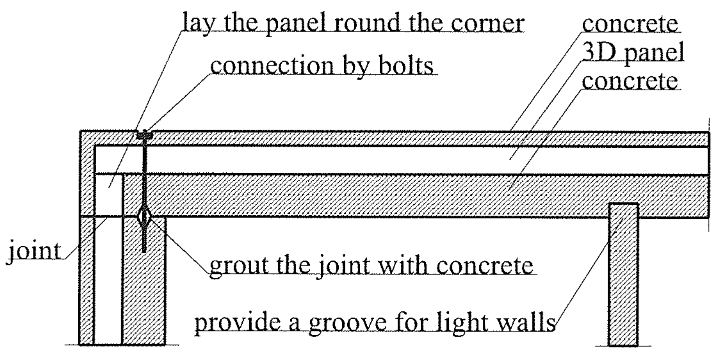


figure 10.2.2.a Connection possibilities for prefabricated walls (ground plan of a wall)

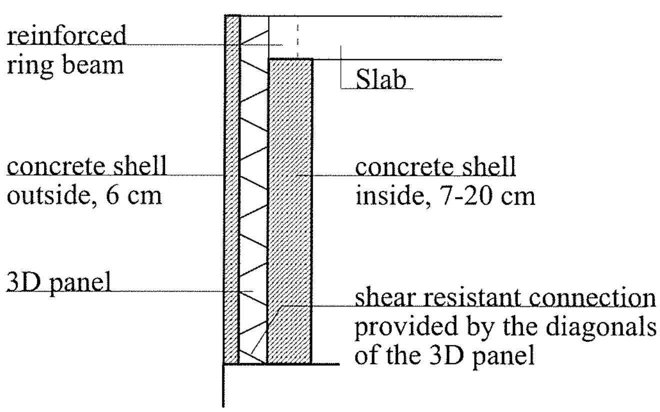


figure 10.2.2.b Structure of a prefabricated 3D wall made of lightweight concrete (section)

10.2.3. Calculation Basis

The calculation bases on the DIN method (see chapter 4). Owing to the fact that the planned eccentricity lies generally in the axis of the inside shell, the outer shell can be considered to be cracked in all cases. Therefore, all further studies of stress distribution refer to the inside shell only. In order to take into consideration the load applied almost in the center an unintended eccentricity of 1/10 of the thickness of the inside shell is taken into account.

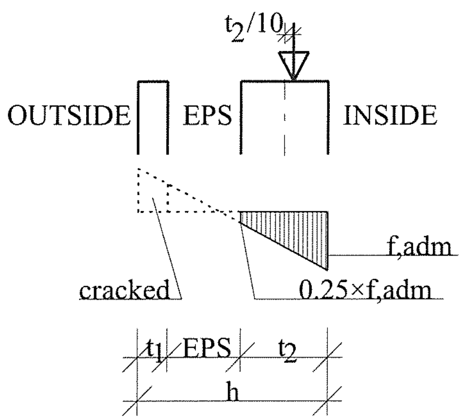


figure 10.2.3.a Stress distribution in a 3D wall with different shell thicknesses loaded on one side only. Unintended ccentricity is $t_2/10$.

In contrast to the observations of section 4 it must not be forgotten that in this case the safety factor is 3.5 (see paragraph 10.2.1).

The table 10.2.3.a shows the admissible vertical loads for walls according to the method of approximation of DIN 1045, chapter 17.9. The table is drawn up according to the following scheme:

eccentricity = loaded on one side only plus $t_2/10$ (= unintended eccentricity)
outside shell = 6 cm concrete
insulation = 10 cm EPS
concrete grade = B5.0 according to DIN

The following values are necessary to determine the admissible vertical load:

t_2 = thickness of internal shell in [cm]
lg_U = clear height of wall

Concrete layer inside [cm]	7	10	12	15	18	20
Clear wall height lg_U [m]	F [kN/m]	F [kN/m]	F [kN/m]	F [kN/m]	F [kN/m]	F [kN/m]
2.00	33.1	49.0	59.7	76.0	92.5	103.7
2.25	31.8	47.3	57.8	73.7	90.0	101.0
2.50	30.5	45.7	55.9	71.5	87.5	98.3
2.75	29.2	44.0	54.0	69.3	85.0	95.7
3.00	27.8	42.3	52.1	67.1	82.5	93.0
3.25	26.5	40.6	50.2	64.9	80.0	90.3
3.50	25.2	38.9	48.3	62.6	77.5	87.7
3.75	23.9	37.2	46.3	60.4	75.0	85.0
4.00	22.5	35.5	44.4	58.2	72.5	82.3
4.25	21.2	33.9	42.5	56.0	70.0	79.7
4.50	19.9	32.2	40.6	53.7	67.5	77.0
4.75	18.6	30.5	38.7	51.5	65.0	74.3
5.00	17.2	28.8	36.8	49.3	62.5	71.7
5.25	15.9	27.1	34.9	47.1	60.0	69.0
5.50	14.6	25.4	33.0	44.9	57.5	66.3
5.75	13.3	23.8	31.1	42.6	55.0	63.7
6.00	11.8	22.1	29.2	40.4	52.5	61.0

table 10.2.3.a Design table for walls loaded on one side only

Intermediate values may be interpolated linearly.

10.3. Prefabricated Slabs

10.3.1. Slabs without Lattice Girder

Prefabricated slabs are produced completely or partially in the precast concrete factory, and they are completed with concrete on site. In general, the top side of the slab is concreted on site to ensure a sufficient load distribution effect in cross direction. Concrete on the bottom is prefabricated completely or partially. Either a complete layer of concrete is applied at the bottom side (figure 10.3.1.a, left side) or the final surface is made of shotcrete subsequently (figure 10.3.1.a, right side). The second variant can be executed easily on the building site and generally does not even require a crane. First the panels are put onto the shoring for the slab upside down. Then the first concrete layer of approx. 3 cm is applied. After one day, the panels can be turned manually.

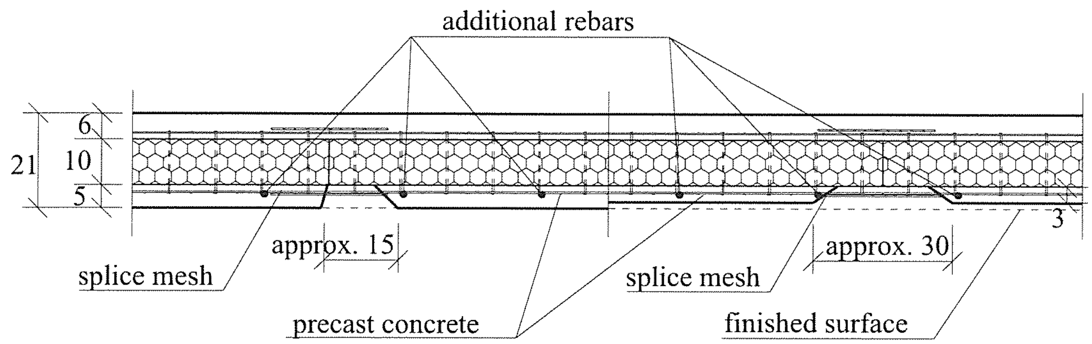


figure 10.3.1.a Slab panels with partly precast concrete at the bottom side

The solutions of figure 10.3.1.a use a continuous bottom reinforcement in cross direction. However, from a structural point of view it is not really necessary to interconnect the panels at the bottom. Cross distribution is achieved generally by the top concrete layer only. To avoid visual problems it is sufficient to seal the joints later.

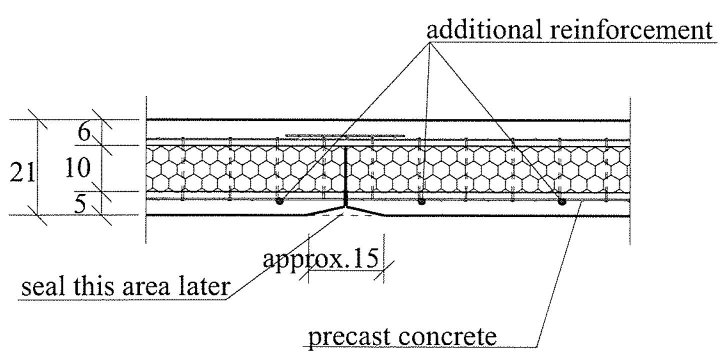


figure 10.3.1.b Slab panels with totally precast concrete at the bottom side

10.3.2. Slab with Lattice Girders

A slab made from 3D panels is considerably restricted by shear strength. Depending on the standard panel types the admissible maximum shear force is 11 to 14 kN/m. Frequently this value is exceeded in slabs and, as a consequence, it makes additional steps necessary. A simple solution for a higher shear strength is a shear reinforcement near the supports. That increases the admissible shear force only slightly. However, if the occurring forces exceed the admissible extent by far, other solutions will become necessary. Such a solution implies always the use of a girder along the entire slab length.

This girder has to contain not only a considerable shear reinforcement but a tensile reinforcement at the bottom as well. For economical reasons the use of a prefabricated lattice girder is recommended.

These girder types are mostly used as part of prefabricated concrete elements. These can be either a small concrete soffit (10 – 12 cm wide) that covers the lattice girder only, or a wide concrete slab (1 – 2.5 m wide) into which both the girder and the 3D panels are placed.

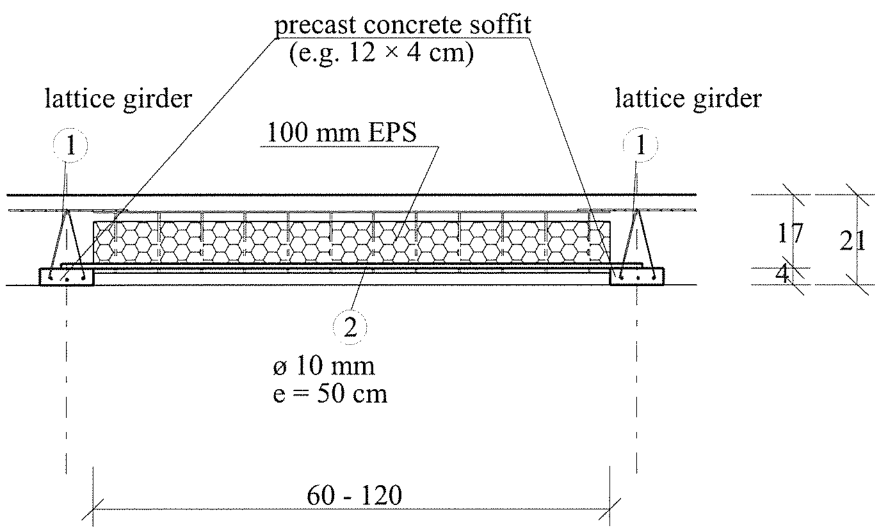


figure 10.3.2.a Lattice girders with precast concrete soffit

This example shows a lattice girder with an additional longitudinal reinforcement within the girder for heavy floor loads. These girders are to be designed according to the rules of traditional reinforced construction. In case of a prefabricated concrete soffit it is recommended to put the 3D panels on the concrete soffits by inserting rebars (ø at least 10 mm) in spacings of 50 cm (item 2 in figure 10.3.2.a). Then the concrete at the bottom can be applied conventionally by means of a shotcrete gun. If the distance between the lattice girders (width of the panels) is bigger than 60 cm the top concrete layer has to be checked as well.

In case of slabs with considerable load and large effective spans several lattice girders can be used. In return, the panels' width can be halved. However, it is not advisable to use very narrow pieces of panels (< 60 cm). This case given, it is more useful to install double lattice girders. In general, it is recommended to provide for a continuous top reinforcement (welded mesh). However, this can be ommited in case of a small shear force only.

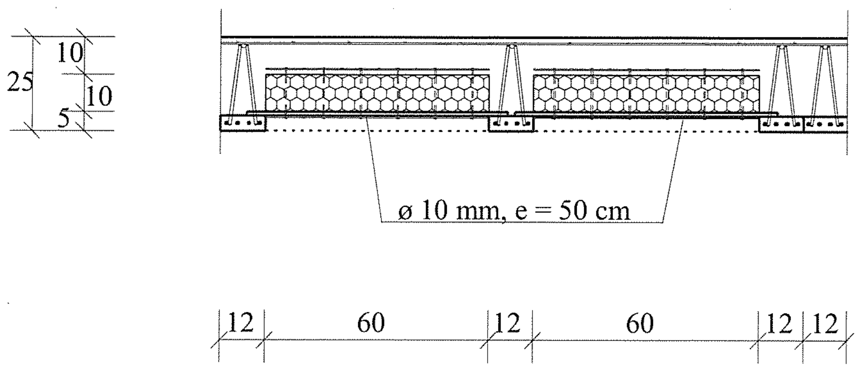


figure 10.3.2.b Thick slab with single or double lattice girders and 3D panels

Figure 10.3.2.b shows an example tested already in practice. Thereby lattice girders and half panels (b = 60 cm) are laid alternately. To be able to place the panels on the lattice girders, bars of ø 10 mm at a pitch of 50 cm are inserted between EPS and cover mesh. To avoid displacement of the panels, at first one concrete layer should be applied from the bottom side.

10.3.3. Prefabricated Slab Elements

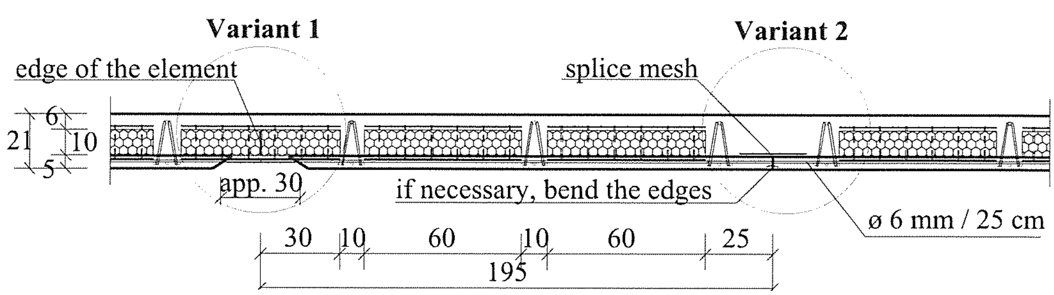


figure 10.3.3.a Slab with lattice girders and 3D panels

Prefabricated slab elements are heavier than narrow concrete soffits and require by all means an appropriate crane. However this allows to save working time considerably. Especially plastering of the slab's bottom side has turned out to be very time-consuming.

The major technical problem is the design of the joint between the slab elements. In variant 1 the concrete element does not extend to the edge of the 3D panel. The joint between the elements is reinforced with splice mesh and closed with shotcrete. In variant 2 the edge of the slab element is made without a panel. Later splice mesh will be laid onto the joint and the entire area will be poured with concrete. In addition, it is possible to seal the joint at the bottom side (see figure 10.3.1.b). For that end, the edges of the element have to be beveled slightly. However, owing to the fact that this can be considered a one-way slab, a structurally efficient transverse reinforcement can be omitted.

10.3.4. Prefabricated Joists

If the span becomes too big so that an overall thickness of 20 – 23 cm is no longer sufficient, the use of prefabricated joists makes sense. The example shown in figure 10.3.4.a can be used for spans of up to 8 - 10 m. When designing the prefabricated component it must be made sure that this beam is able to bear all loads applied during erection and that a small number of supports is necessary only.

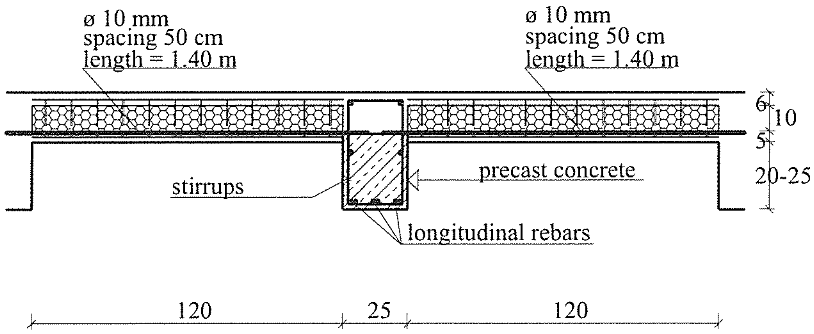


figure 10.3.4.a
Prefabricated joist

10.4. Structural Calculation

Compared with a simple 3D slab the main difference lies in the lattice girder's shear strength. Be aware that the decisive shear force for the panel occurs right next to the support whilst the design of a lattice girder can start out from a lower shear force in a distance of d/2 (figure 3.7.a, left side).

When using lattice girders with 2 diagonal wires (for more details see also paragraph 3.7.2), the admissible shear force results in the following on the assumption of a strut in the concrete inclined by 45°:

$$V = 2 \cdot \frac{a_s \cdot f_y \cdot z}{1.75} \cdot (\sin\alpha + \cos\alpha)$$

i.e. a_s........ cross section of diagonals per meter
z......... lever arm of the internal forces; approx. 95% of effective depth
1.75.... global safety factor according to DIN

In any case the concrete strut, too, has to be checked according to the shear stress τ. Check in level 1:

$$\tau = \frac{\Delta V}{b_0 \cdot z} \leq \tau_{03} \text{ (according to DIN)}$$

i.e. ΔV ..... shear force without the panel's share
b_0 .... width of the concrete's cross section. For a lattice girder this corresponds to the width between the panels (about 10-12 cm).

According to DIN, shear stress must not exceed the value τ_{03}.

Another check for low lattice girders is required at level 2. If the girder's upper chord does not reach into the concrete topping (figure 10.4.b, right side), the shear stress at the top end of the lattice girder has to be proven. In the area of level 2 the entire shear force has to be absorbed by the concrete. Therefore, shear stress must not exceed the value of τ_{01}. Otherwise near the support it is necessary to lay an additional lattice girder over the existing girder. In addition, if there is no continuous top reinforcement, a check in level 3 will be necessary.

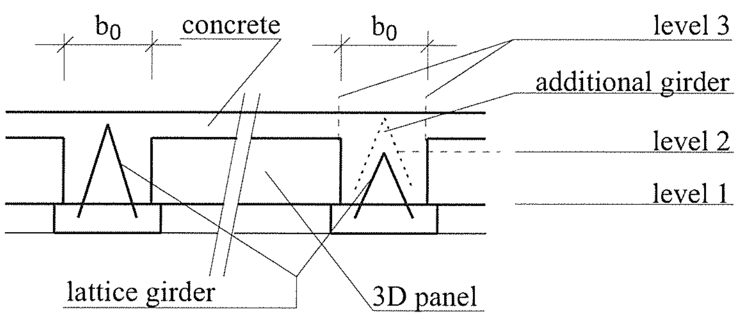


figure 10.4.b Prefabricated slab elements or rib slab

10.5. Details

For the use of lattice girders some more details have to be elaborated to be able to construct the slab safely. This refers in particular to the design of the supports, the necessity of cross ribs for big effective spans and the location of additional reinforcement.

Support details

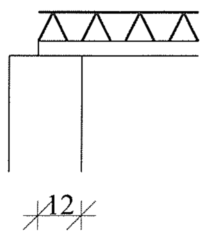


figure 10.5.a Support ≥ 12cm

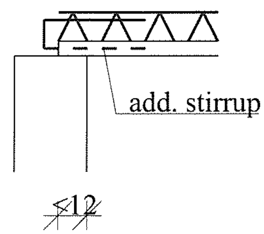


figure 10.5.b Support < 12cm

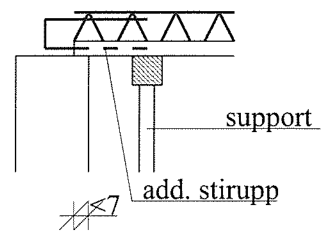


figure 10.5.c Support < 7cm

If the lattice girder's support is not at least 12 cm long, it will be necessary to lengthen the bottom reinforcement. The most favorable solution is to place a U-shaped stirrup in the concrete soffit. However, it is also possible to place a stirrup on top of the soffit on site. If the support is shorter than 7 cm, the lattice girder needs to be supported during concreting in the vicinity of the support.

Cross rib

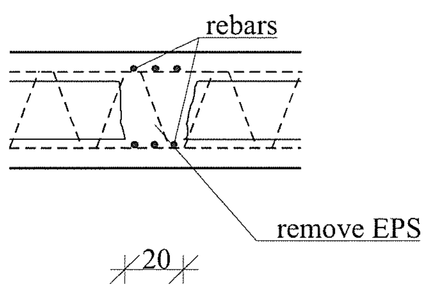


figure 10.5.d Cross rib for effective spans beyond 6.0 m

In slabs with an effective span beyond 6.0 m it is necessary to insert a cross rib at midspan of the slab area. One additional cross rib is recommended for every 1.50 m added. Reinforcement of this cross rib (top and bottom) should correspond roughly to the reinforcement of the lattice girder including additional reinforcement and has to stretch along the entire slab width. The EPS in the area of the cross rib has to be removed (e.g. by burning it out).

The distribution of the additional bottom reinforcement should correspond approximately to the ratio between the shear strength of the panel and the lattice girder. In case of a slab with a required shear resistance of 28 kN/m and a shear strength of the panel of some 14 kN/m, the 3D panel and the lattice girder must contain roughly half the entire tensile reinforcement. If there is not enough space available in the concrete soffit, it will be possible to place a part of the additional reinforcement on top of the soffit, as well. However, the lower effective depth has to be taken into consideration for the calculation. The remaining reinforcement has to be fixed evenly to the panel. It is not necessary to keep strictly to the above distribution of the reinforcement. Instead of a ratio of 50/50 the ratio may be also 40/60.

11. Details

Overview

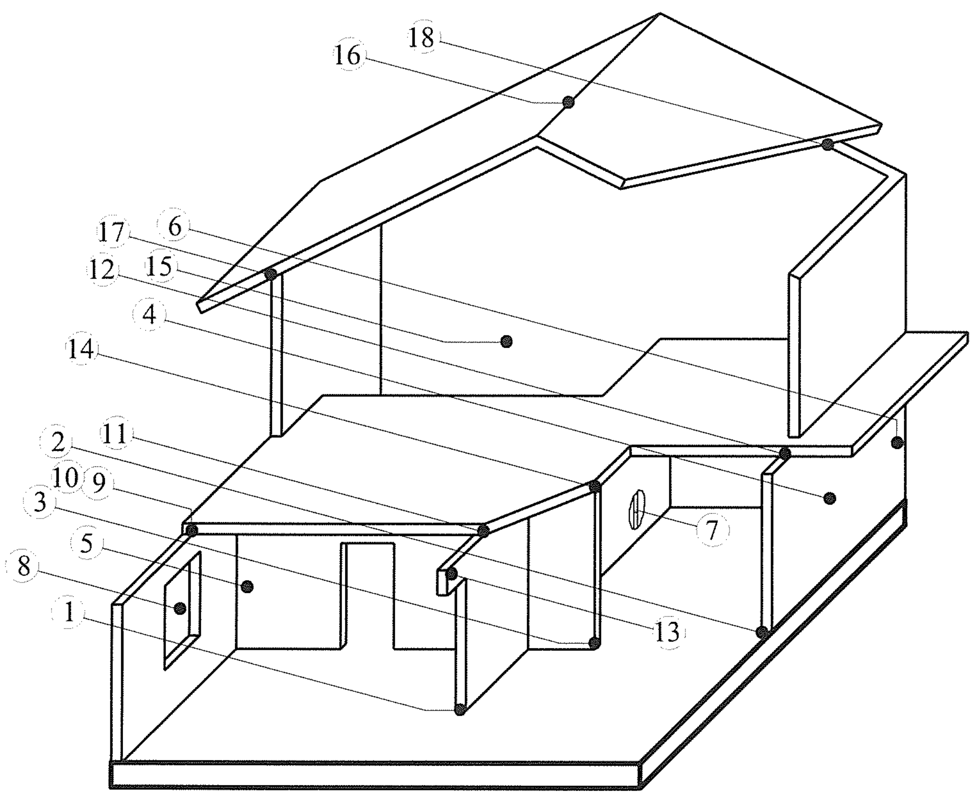


figure 11.a Overview of the structural details

11.1. Connection to the Foundation

Connection Foundation Slab - Inside Wall

FOUNDATION SLAB - INSIDE WALL

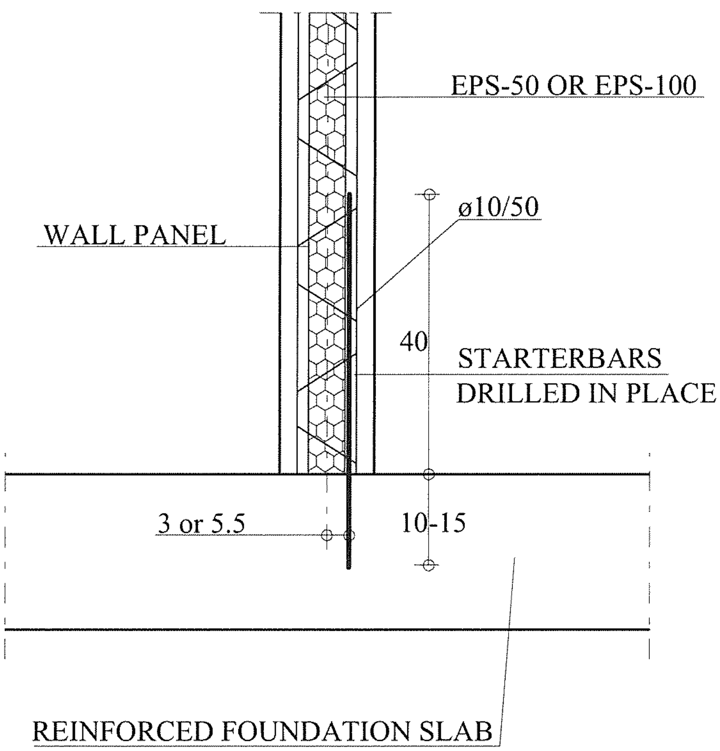


figure 11.1.a Connection between foundation slab and inside wall with starter bars

The axis of the starter bars is at a distance of 3 cm (EPS-50) or 5.5 cm (EPS-100) from the wall's axis. The drilling depth is 10 - 15 cm.

Connection Foundation - Outside Wall

FOUNDATION - OUTSIDE WALL

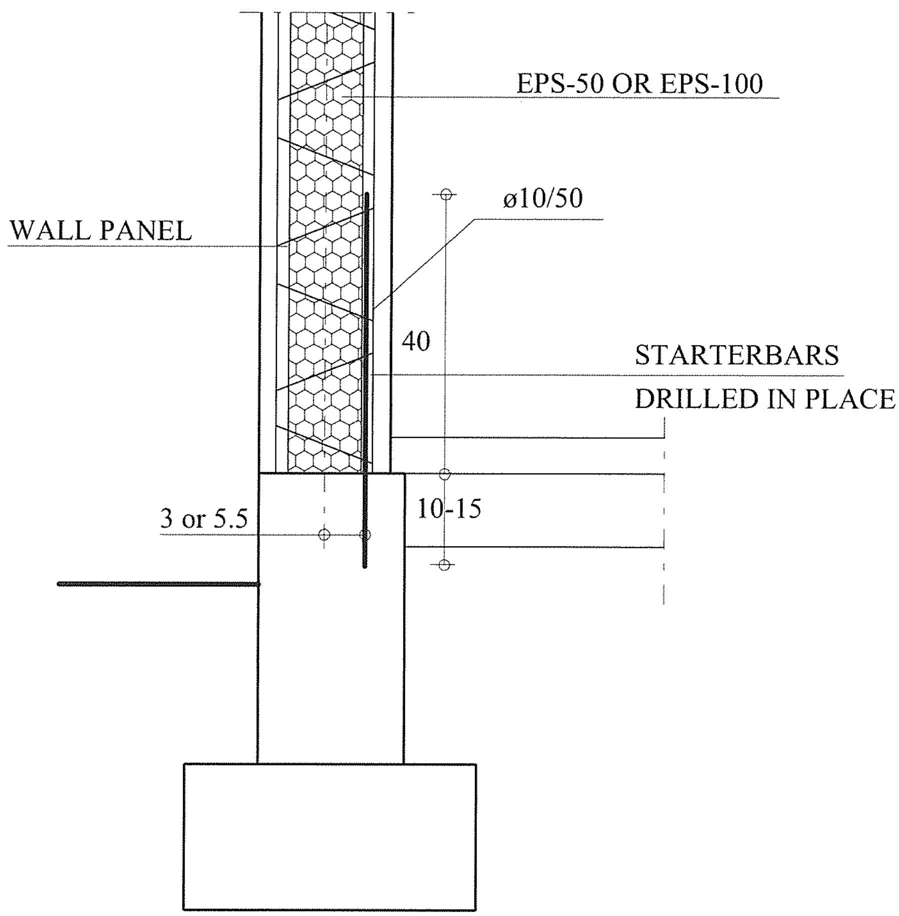
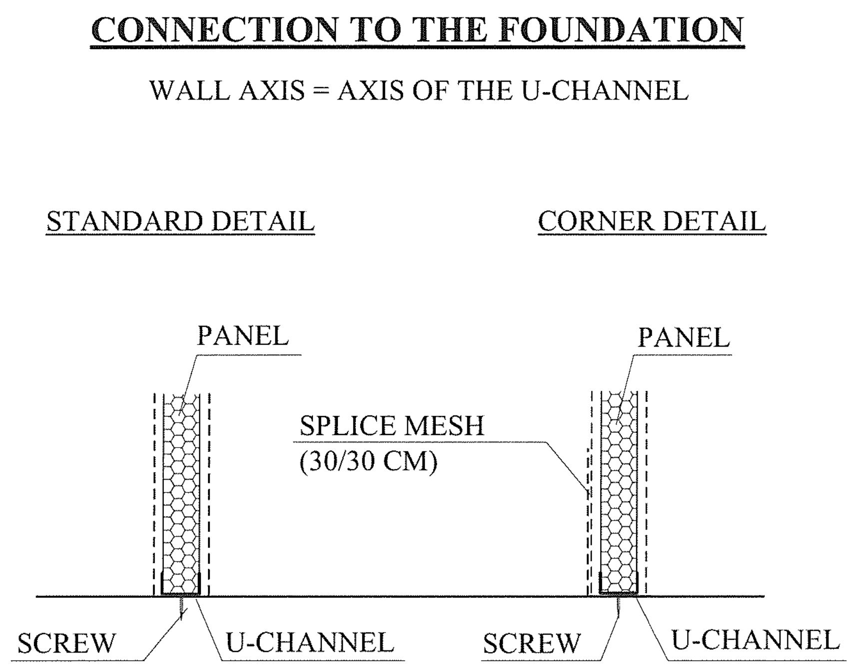


figure 11.1.b Connection between foundation and outside wall with starter bars (strip foundation)

The axis of the starter bars is at a distance of 3 cm (EPS-50) or 5.5 cm (EPS-100) from the wall's axis. The drilling depth is 10 - 15 cm. The outer edge of the finished wall is flush with the outer edge of the foundation.

Connection to Foundation with U-Channels



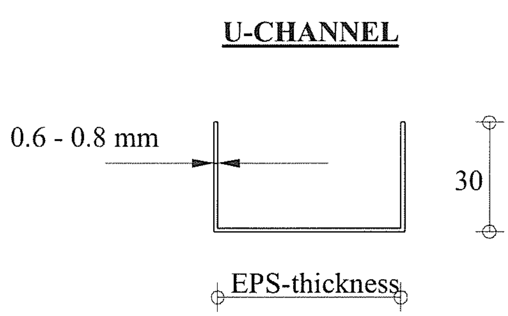


figure 11.1.c Connection between foundation and inside wall with U-channels

The axis of the U-channel corresponds to the wall's axis. The U-channel is screwed at a distance of 50 - 100 cm. In edges, long walls, at some intermediate points and next to door openings, an L-shaped splice mesh (30 × 30 cm) is clamped under the U-channel.

11.2. Connection Wall - Wall

Straight Panel Splice

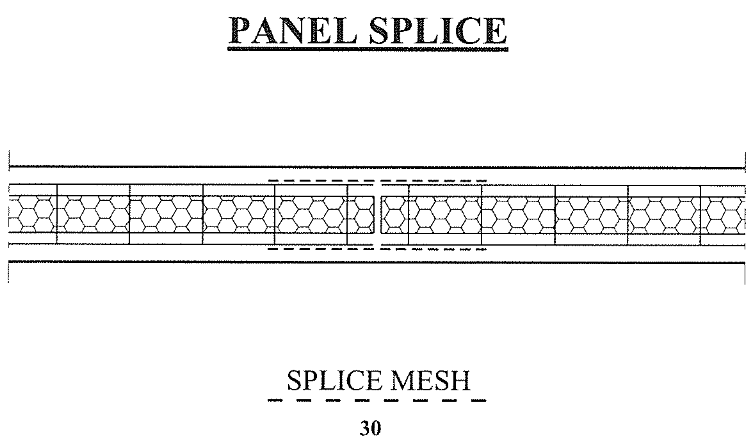


figure 11.2.a Straight panel splice

The length of splice mesh in straight panel splices is 30 cm.

Connection Wall - Cross Wall

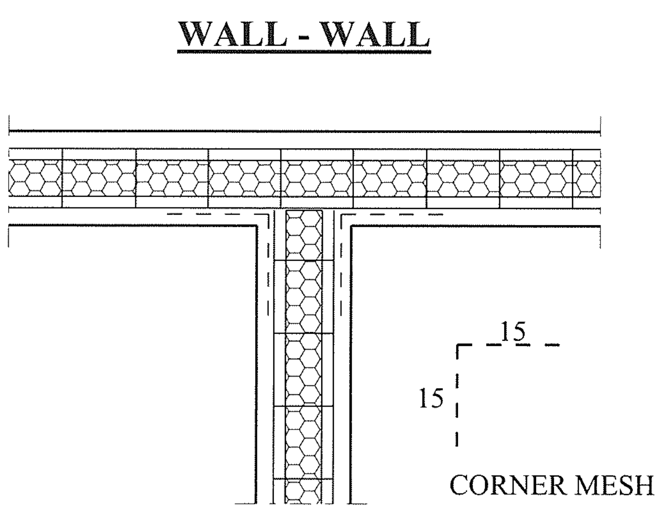


figure 11.2.b Inside panel splice

Splice mesh is fixed on both sides. Its length is 2 × 15 = 30 cm.

Outside Wall Corner

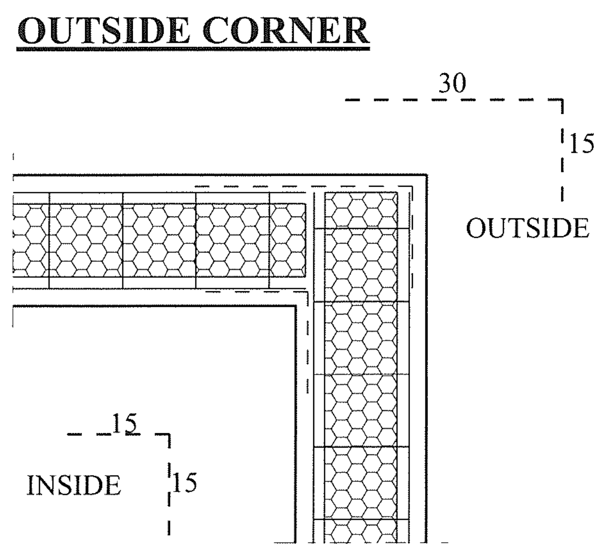


figure 11.2.c Outside wall corner

For wall panels of 50 mm EPS each, splice mesh with a width of 10 + 20 = 30 cm can be used for outside corners.

Sewage Pipe

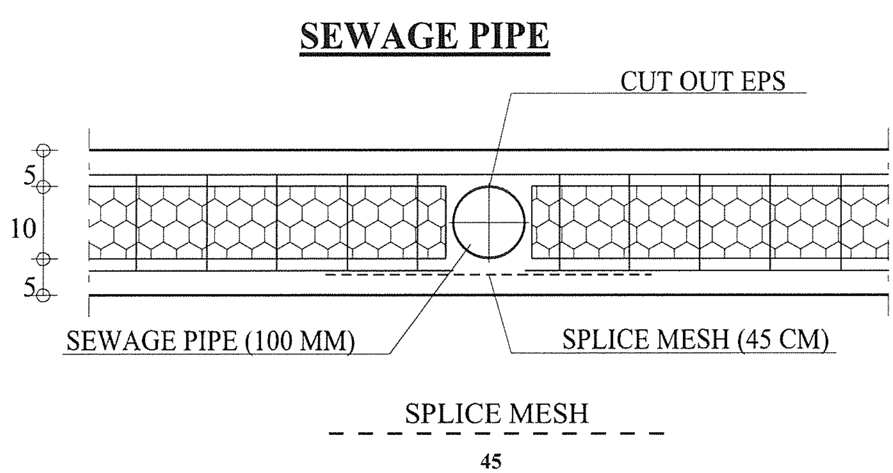


figure 11.2.d Sewage pipe

If necessary the whole EPS can be removed. In case of an outside wall the sewage pipe possibly has to be placed in front of the wall. The pipes for water and electric installations will be laid between EPS and cover mesh after the erection of the panels. Before laying thicker pipes possibly a groove in the EPS has to be burned out by a torch.

Wall Opening

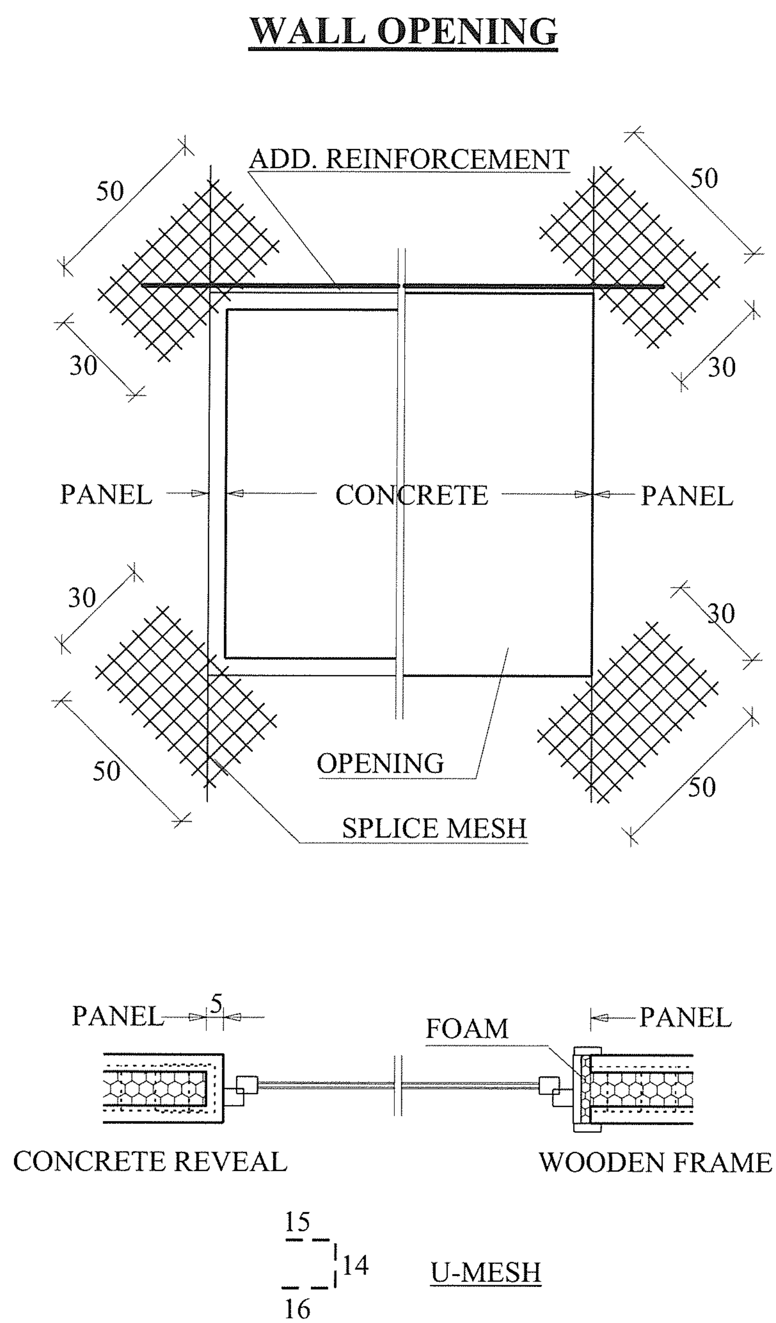


figure 11.2.e Wall opening

The splice mesh has to be fixed in all corners of the opening and has to have a length of at least 50 cm. The opening can be made with or without concrete reveal.

11.3. Connection Wall - Slab

Connection Slab - Outside Wall

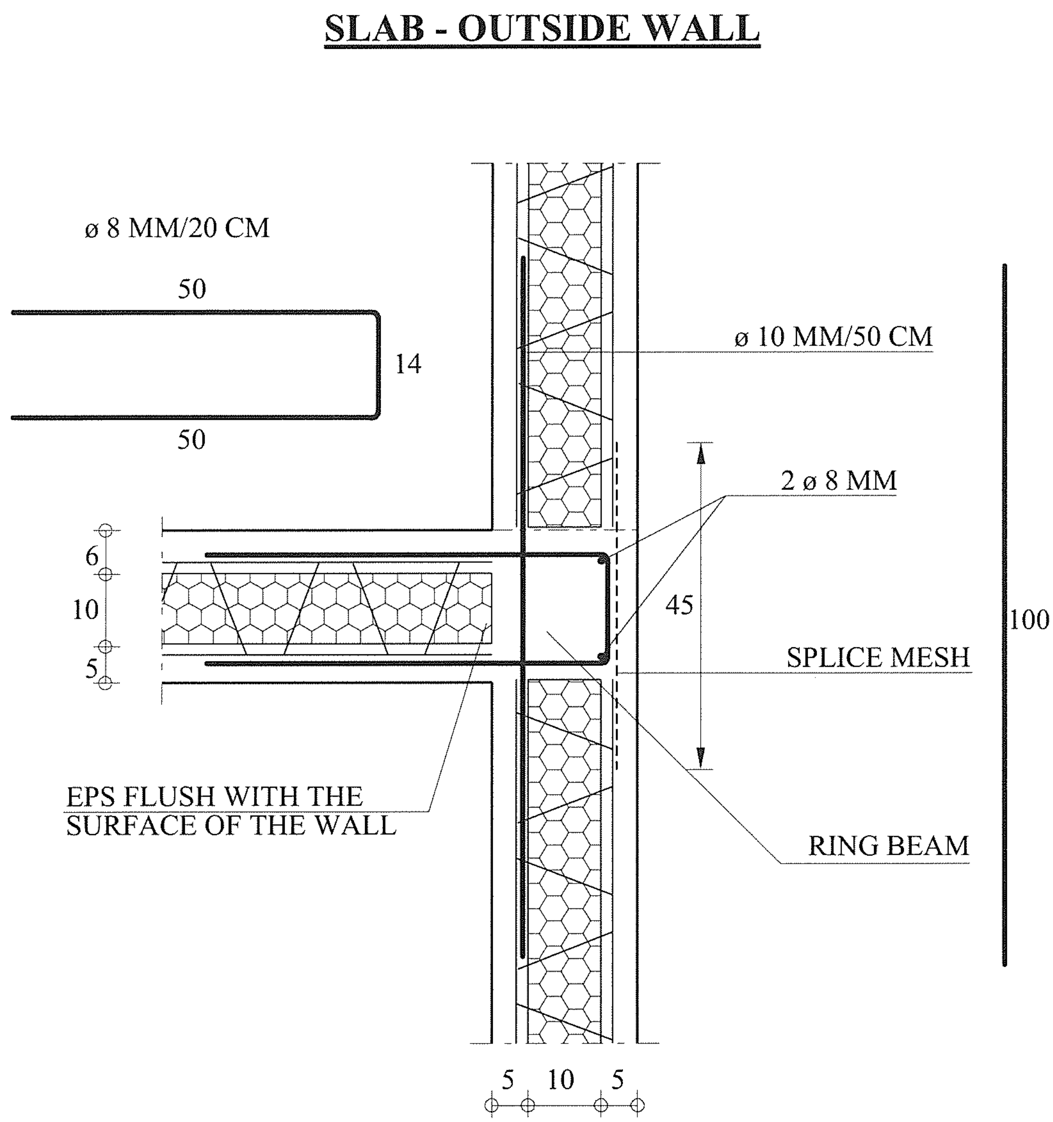


figure 11.3.a Connection slab - outside wall

The area of steel in the ring beam has to be at least 1.0 cm² (= 2 ø 8 mm). The distance of the stirrups at the support can be extended at the cross edge to up to 40 cm. The EPS of the slab panel has to be flush with the surface of the above wall.

Connection Slab - Outside Wall (Avoiding Thermal Bridging)

SLAB - OUTSIDE WALL

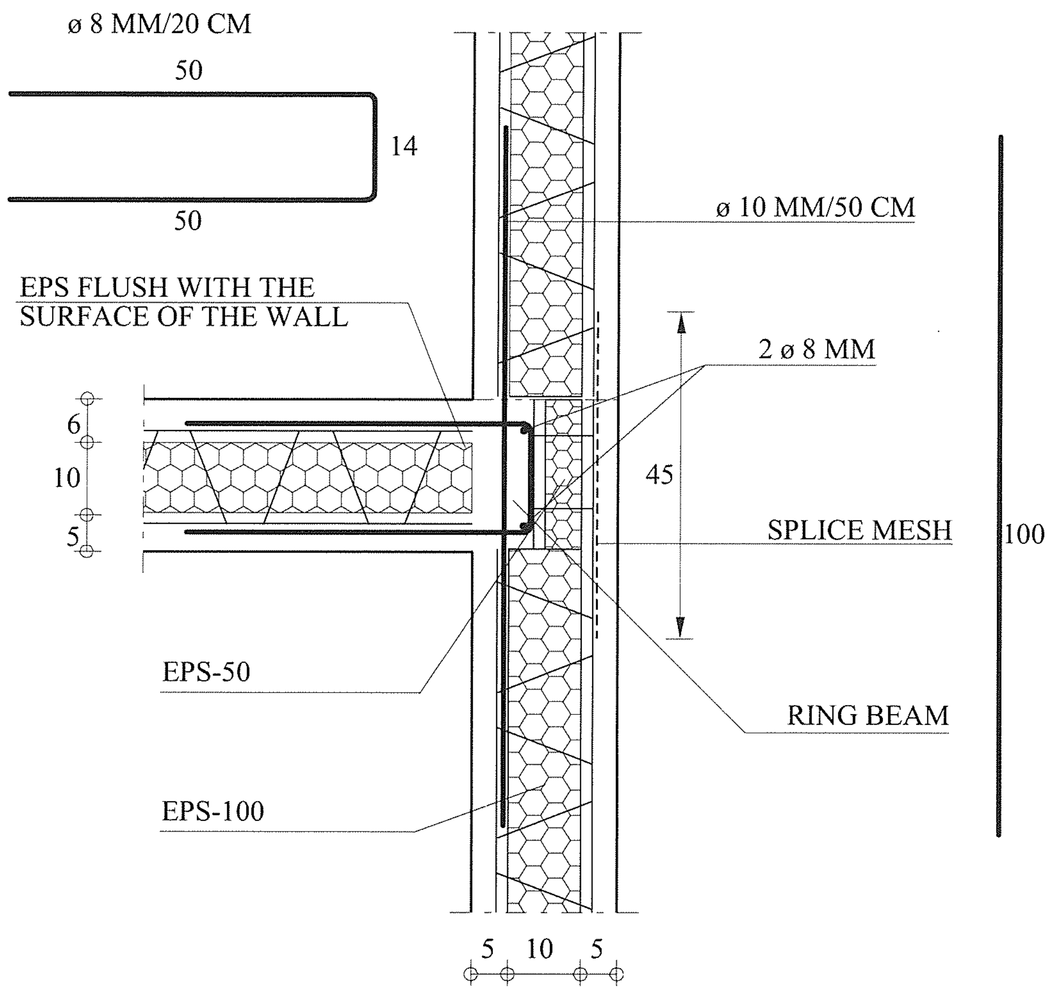


figure 11.3.b Connection slab – outside wall with thermal insulation

The area of steel in the ring beam has to be at least 1.0 cm² (= 2 ø 8 mm). In order to guarantee the distance between the additional thermal insulation (50 mm) and the reinforcement in the slab, a residual piece of a panel with 50 mm EPS can be used. The distance of the stirrups at the support can be extended at the cross edge to up to 40 cm. The EPS of the slab panel has to be flush with the surface of the above wall.

Connection Slab - Load-bearing Inside Wall

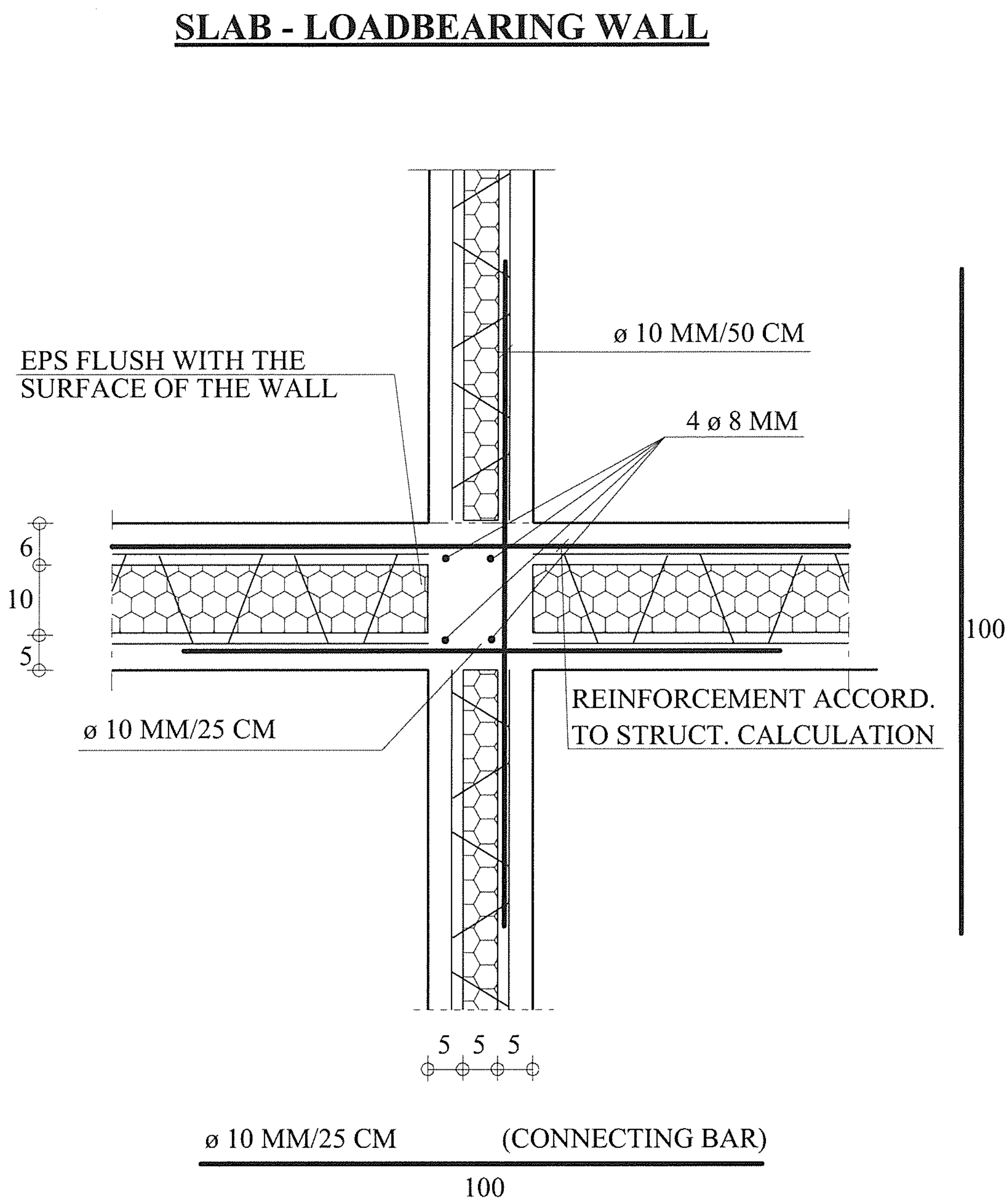


figure 11.3.c Connection slab - load-bearing inside wall

The connecting bars at the support transfer also compression forces during erection and are therefore chosen in most cases with a diameter of 10 mm. The EPS of the slab has to be flush with the surface of the above wall. A reinforced ring beam is not absolutely necessary.

Connection Inside Slab - Cantilever Slab

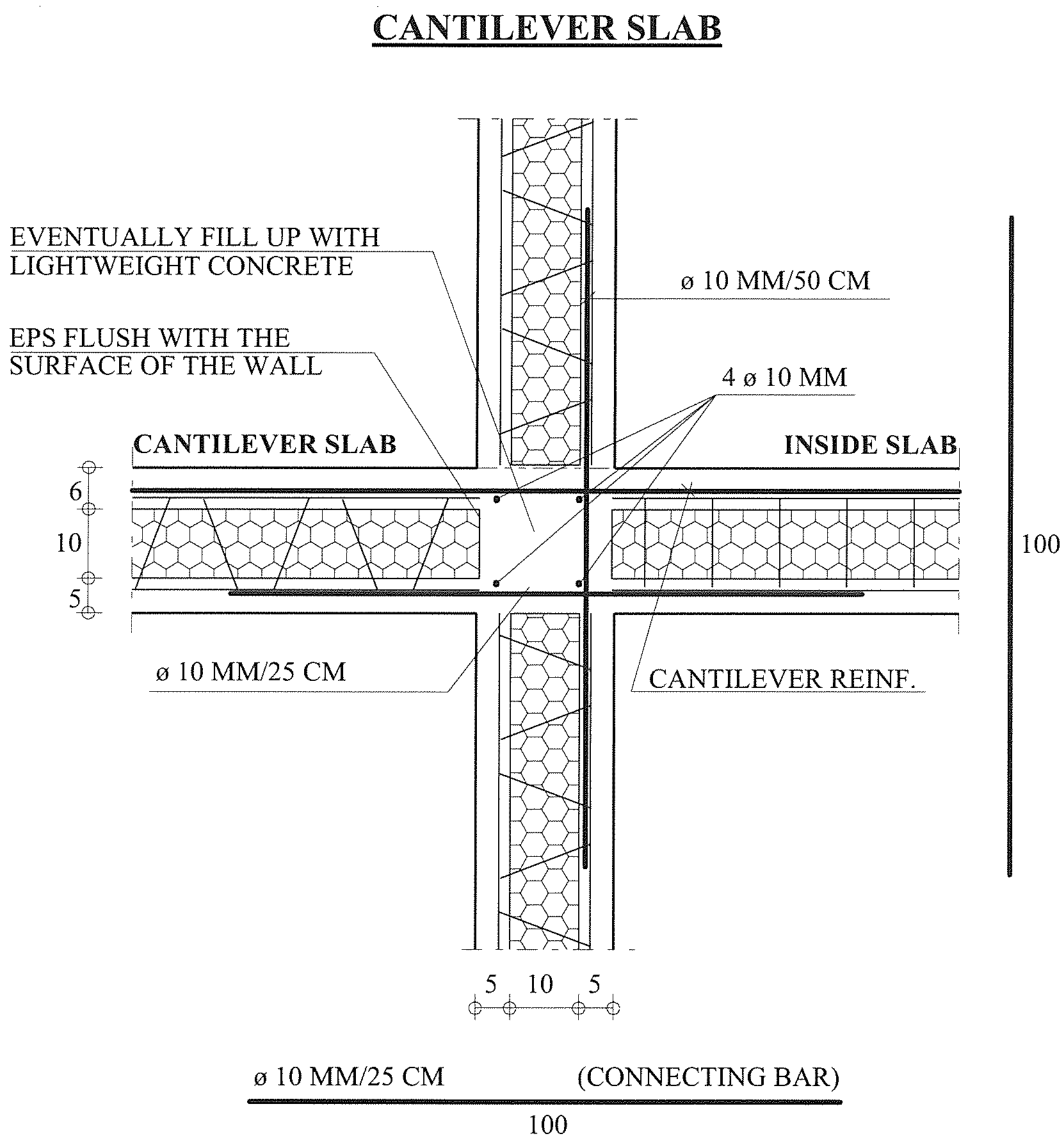


figure 11.3.d Connection of a cantilever slab to the cross edge

In order to avoid a thermal bridge, the ring beam can be filled up with lightweight concrete. Eventually a compression reinforcement has to be placed then. A reinforced ring beam is obligatory only at the connection of the cantilever slab to the cross edge of the inside slab. The EPS of the slab has to be flush with the surface of the above wall.

Connection Slab - Beam

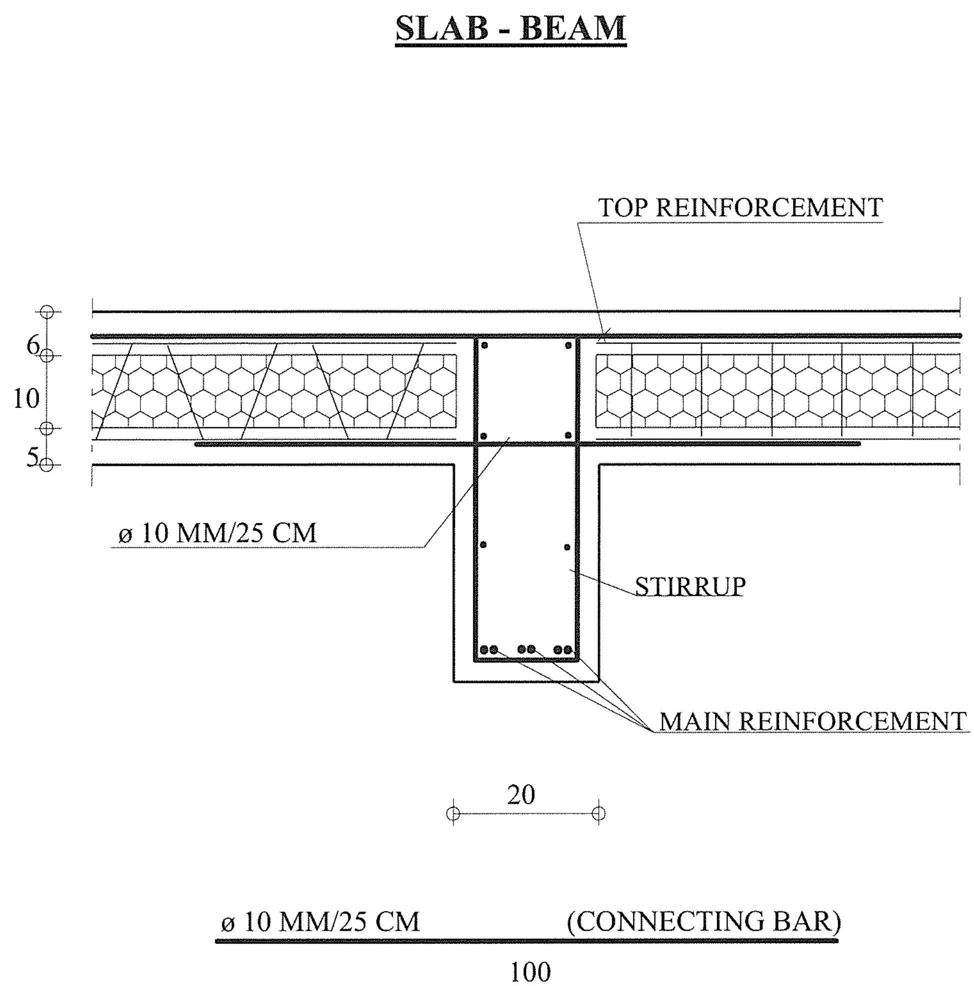


figure 11.3.e Connection slab-beam

The slab has to be connected to the beam by means of rebars on the top and bottom.

Connection Slab - Non Load-bearing Inside Wall

SLAB - NON LOADBEARING WALL

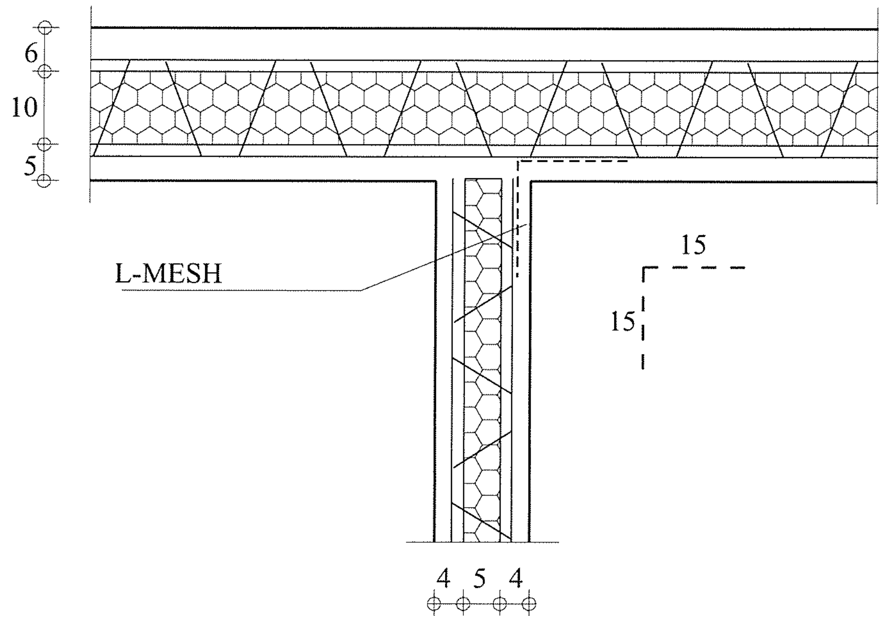


figure 11.3.f Connection slab - non load-bearing inside wall

The splice mesh is required only for alignment of the wall and is generally fixed on one side only. In a wall erected lengthwise to the slab, the splices on top of the slab have to be closed thoroughly with splice mesh. The concrete layers on top and bottom of the slab must not be reduced by the wall panels.

High Walls

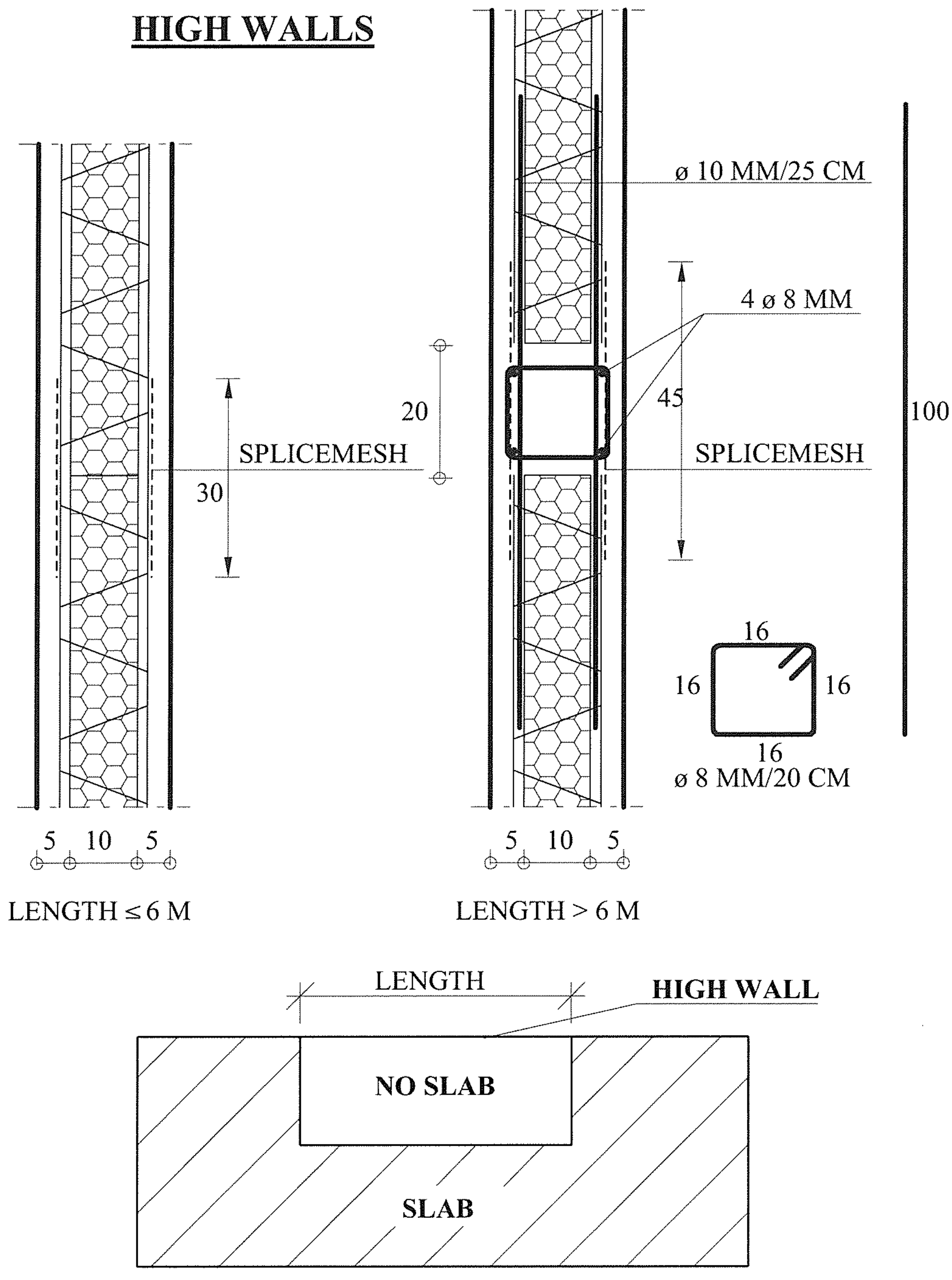


figure 11.3.g Walls of double storey height

Walls of double storey height require a stiffening ring beam only in case of great lengths without cross walls. If the wall is shorter than 6 m, this stiffening ring beam can be omitted.

11.4. Roof

Ridge Detail

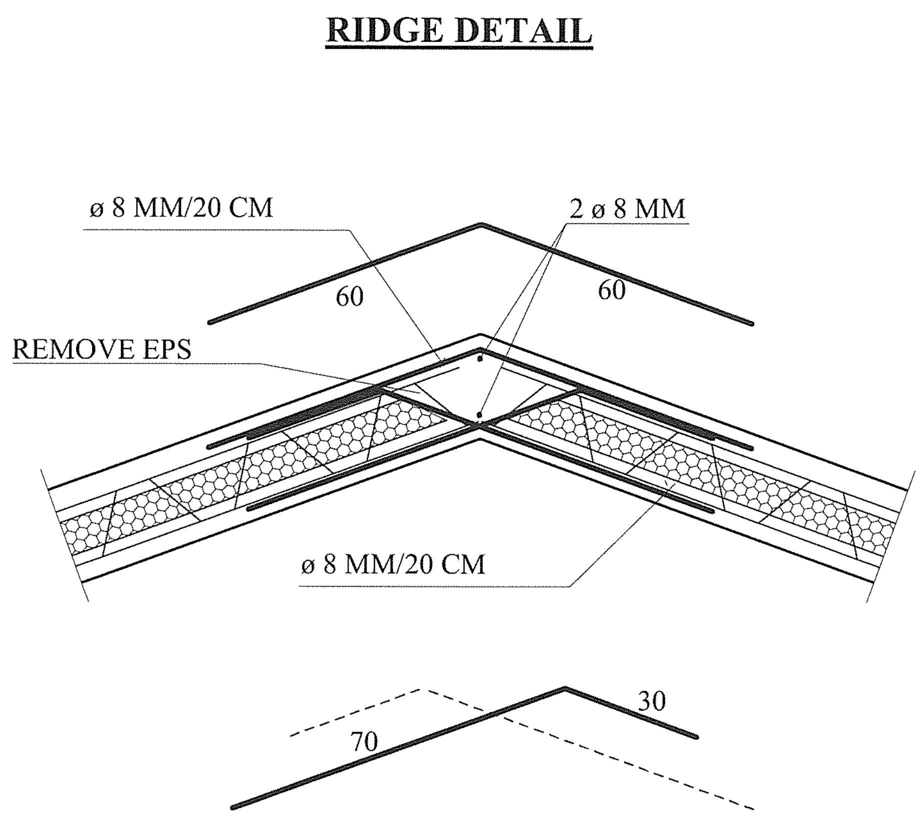


figure 11.4.a Ridge with connection reinforcement

In the area of the connection reinforcement, the EPS has to be removed to ensure the concrete cover.

Eaves Detail

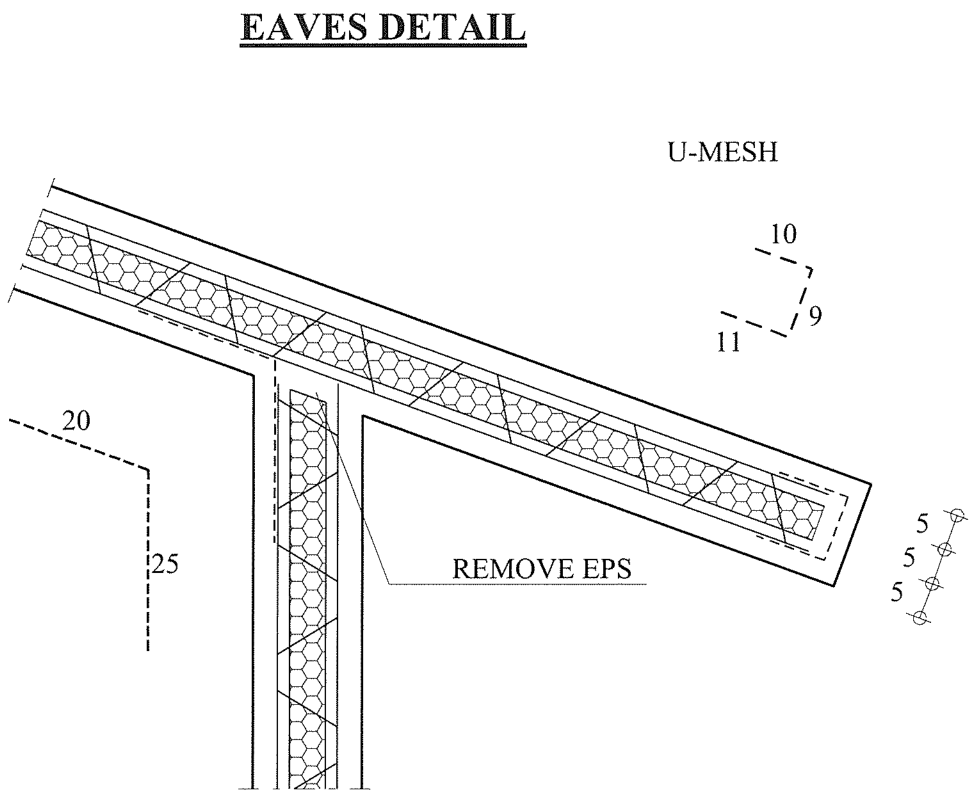


figure 11.4.b Design of the eave

In the area of the support, the EPS has to be removed or the panel length has to be chosen correspondingly shorter.

Overhang at Cross Edge

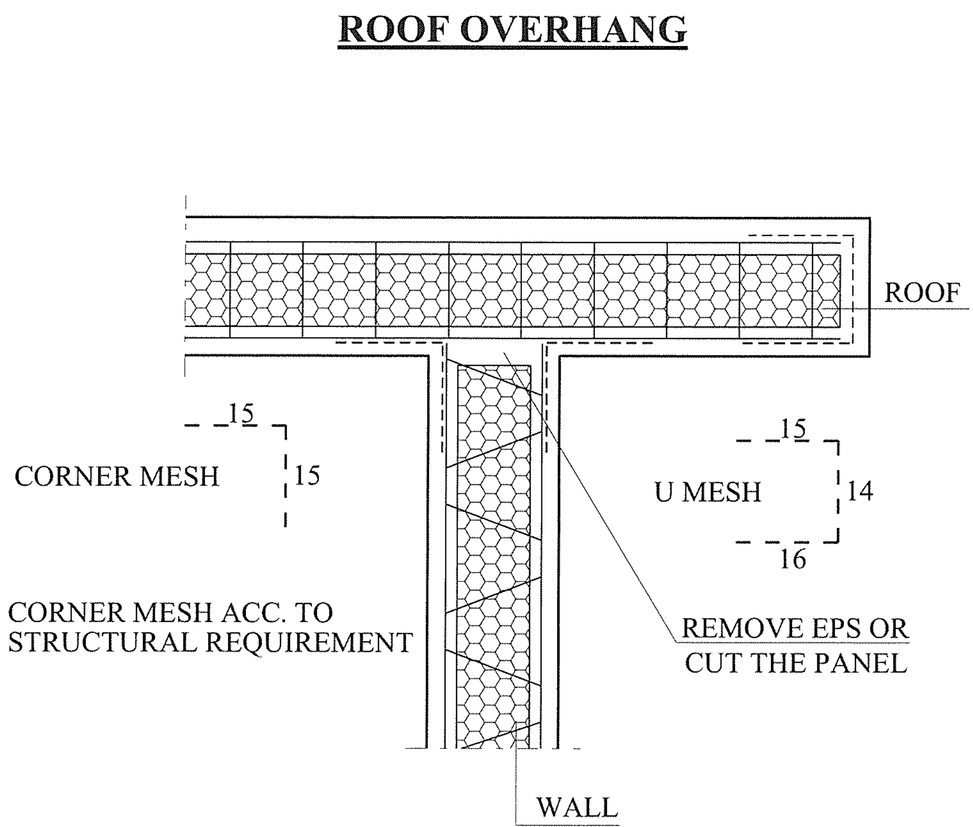


figure 11.4.c Roof overhang at cross edge

The thickness of the bottom concrete layer must be kept the same also in the area of the wall panels.

12. Concrete

12.1. Concrete Production

12.1.1. Basic Materials

12.1.1.1. Mixing on Site

Depending on the required concrete grade the aggregate is mixed for 3 - 4 minutes with about 300 kg cement and the required quantity of water in a compulsory mixer before the pump is charged. The data of the following table are sufficient at least for concrete grade B15 (fc = 10.5 N/mm²). Actual concrete grade depends also on the sieve curve of the aggregate and has to be found out by tests.

aggregate 0-4 mm	1,750.0 kg
cement	300.0 kg
water	150.0 kg
total	2,200.0 kg

table 12.1.1.1.a Mix ratio for concrete grade B15

12.1.1.2. Gradation Limits

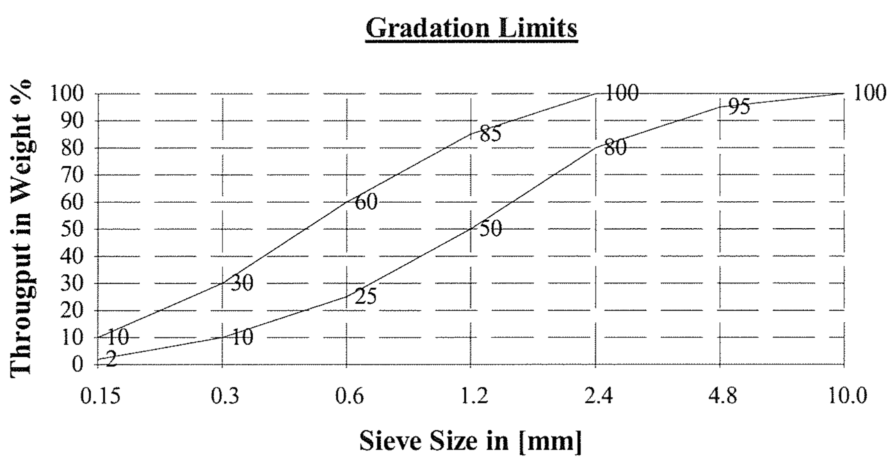


diagram 12.1.1.2.a Gradation limits recommended by ACI

Keeping to the correct sieve curve is not only a prerequisite to attain a good concrete quality but it is also decisive for the pumpability of the material. In order to achieve such a pumpability the aggregate has to contain a minimum quantity of superfines with a diameter of up to 0.125 mm. In the 0.125 mm sieve, the screenings should be at least 4 - 5 % and should not exceed 8 - 9 %. The superfines ensure that the concrete is capable of keeping the water and that it can be pumped through the hose. If there are not enough superfines - these include at long last also cement - they will have to be replaced by another material. In case of an aggregate that is extracted from lakes or rivers, there are nearly no fine grains.

The gradation limits according to diagram 12.2.1.2.a are taken from a recommendation by the American Standardization Institute. The portion of superfines in the lower part of the sieve curve is relatively small. This case requires to add other materials in order to achieve pumpability.

The gradation limits according to diagram 12.2.1.2.b explain the grain composition of the aggregate as recommended by Turbosol for their pumps.

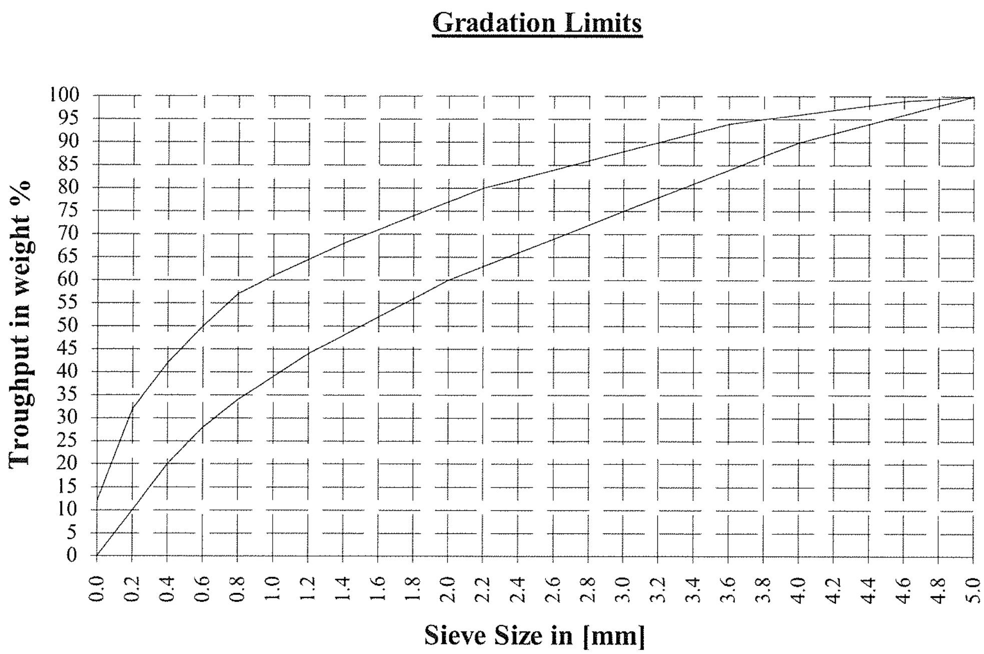


diagram 12.1.1.2.b Gradation limits recommended by Turbosol

12.1.1.3. Grain Size

The used grain size depends on the requirements on strength and performance of the pump. While dry-mix guns easily can process a maximum grain size of 8 mm in general, the grain size for most mortar pumps is limited to 4 or 5 mm at most. The final concrete strength of 10 to 15 N/mm² (= fc) necessary mostly for walls can also be achieved with a maximum grain size of 4 mm.

12.1.1.4. Cement Content

The shotcrete's cement content is about 300 kg/m³. This value ensures both sufficient strength and the pumpability of mortar. Due to a larger portion of water concrete with a bigger cement content is prone to shrink and, as a consequence, to form cracks.

12.1.1.5. Water/Cement Ratio

The ratio between the water's and cement's quantities does not only influence workability but also strength and rust protection of the reinforcement. If the water content is too high, air voids form and influence the concrete's quality. For reinforced concrete a water/cement ratio of 0.5 to 0.6 is recommended.

12.1.1.6. Ready-mix Concrete

The use of factory-made ready-mix concrete proves to be the definitely best solution. Both the aggregate's sieve curve and the cement quantity are largely ensured in this case. Therefore pumpability and quality remain constant within very tight limits.

The ready-mix concrete is supplied either in bags or, for large construction sites, in silos. In general, open and unprotected storing on site must be declined. Owing to the fact that the material may easily absorb the humidity from the air, the cement would set already after a couple of hours and it would be impossible to ensure the concrete's quality any longer. If the material is delivered in the morning and processed throughout the day, quality will be considerably deteriorated at the end of the working day.

12.1.2. Pumpability

Pumpability is one of the decisive criteria for the composition of concrete or mortar when applying the wet-mix method. This characteristic is not measurable but has to be found out by experience and a pump test. However, there are various simple tests that allow rather promising conclusions on the behavior of the material.

The **squeezing test** is very efficient. Take a handful mortar and squeeze the paste through your fingers by closing your fist. If neat cement grout only runs through your fingers and a squeezed-out sand lump is left in your palm, there will be danger of stopping-up the pump. This happens mostly in case of cement mortar made from river sand. Well pumpable mortar runs smoothly through your fingers and leaves only few small sand lumps behind in your fist.

Bleeding test: Fill one bucket with mortar full to the brim. After 15 minutes no water must have settled on the surface. Otherwise the mortar bleeds and is prone to stop up the pump.

Funnel test: Take a big petrol funnel whose lower opening has a diameter of about 3 cm. Pumpable mortar with a soft consistence will run through if the funnel is completely filled.

12.1.2.1. Stone Dust

The use of this material is a relatively inexpensive way to increase the content of superfines. Usually this is finely ground limestone. Approximately 5 - 10 % of the aggregate's weight (= 75 - 200 kg/m³) can be added. If the cement content is too high the cement can be replaced by stone dust up to a certain extent.

12.1.2.2. Fine Sand

The entire portion of superfines can be increased by adding sand containing the appropriate fine materials. However, this may hold the danger that the sieve curve lies beyond the gradation limits also in other sections and, as a consequence, that the final strength of the concrete is decreased if the sand content is too high.

12.1.2.3. Chemicals

Some special chemical additives (e.g. SikaPump from Sika or Rheobuild 3520 from Master Builders) may increase the concrete's pumpability considerably. According to the manufacturer's information no disadvantageous influences on strength may be expected. The recommended quantity of SikaPump is 0.5 - 1.0 % of the weight of the cement (= 1.5 - 3.0 kg/m³). The price of this materials, however, is relatively high.

12.1.2.4. Lime

Adding minor quantities of lime may substantially increase the concrete's pumpability. In order to avoid any negative effects on concrete quality the addition of lime can be recommended for final layers only.

12.2. Processing

Concrete is applied to the walls and to the slab's bottom side after the complete erection of wall and slab panels. In most cases this is done by means of shotcrete since the manual application of concrete or the application with hopper guns is rather time-consuming.

Shotcrete can be processed according to both wet-mix and dry-mix method. In most cases small pumps (30 - 50 l per minute) are used on account of small layer thicknesses and because of minor room sizes. Depending on local characteristic features an output of about 5 - 15 m³ a day can be achieved.

12.2.1. Dry-mix Method

A major advantage of the dry-mix method is the low sensitivity of the pumps to the composition of the starting material. The material is shot through the hose by means of compressed air and mixes with water at the nozzle only. This method helps to avoid blockings in the hose. Furthermore, it is possible to interrupt spraying for a short time and to do a restart without major risks.

The most important disadvantage of the dry-mix method is that an after-treatment of the concrete should be omitted and that the production of a smooth and even surface is not free of problems. That's why a concrete layer applied according to this method should be kept rough. The final layer has to be applied by a different method, such as finishing by mortar.

A risk that must not be underestimated is the use of pre-mixed material that is not kept in bags or silos. If the material is stored for several hours in the open, it may absorb the humidity from the air and start to set partly already before processing. Thus the shotcrete's quality would be decreased considerably during a working day.

Another disadvantage is the necessity to use a relatively big, and, as a consequence, a rather expensive compressor.

Among others, the following equipment has been used so far:

- Aliva 246
- Reed M20

In both cases the corresponding compressor must deliver 5 - 7 m³ per hour at 6 bar.

The rebound to be expected is 15 - 40 %. Thus it is about twice the one for wet-mixing. While the rebound of mortar pumps can be remixed with cement in parts and, as a consequence, some of it can be reused when using mortar pumps, the rebound of dry shotcrete pumps cannot be used again.

Besides, dust has become a rather nasty problem that can be observed especially in small rooms. Thus, wearing a mask is inevitable for the directly involved workers.

12.2.2. Wet-mix Method

Practical experience has shown that the wet-mix method is the much more favorable variant. It is possible to use relatively small and less costly pumps that are also known as mortar pumps (this implies the fluid transition from mortar to micro concrete). The compressor's capacity needs to be lower, too. When applying the dry-mix method the air flow's task is to shoot the dry mixture through the hose while, in case of the wet-mix method, the compressor's only task is to accelerate the concrete at the nozzle. Even the problematic after-treatment of the concrete is less complicated for wet-mix shotcrete than for dry-mix shotcrete. Screeding and smoothing the concrete is easily possible.

With up to 10 % the total amount of rebound is only half the one accumulated with the dry-mix method. Even the formation of dust is relatively small.

Machines for wet-mix concrete demand much more of the concrete. A crucial issue in this context is the concrete's pumpability. In order to achieve such pumpability the aggregate must keep a given sieve curve (see also item 12.2.1.2). Otherwise it will not be possible to avoid frequent blockings in the hose.

Among others the following pumps have been used so far:

- Putzmeister S5 (worm pump)
- PFT ZP 3 (worm pump)
- Putzmeister P13 (piston pump, 2 pistons)
- Turbosol Mini (piston pump, 1 piston)
- Quick Spray Pump (carrousel pump)

Machines with a worm pump are primarily suited for well pumpable mixtures with a maximum grain size of 4 to 5 mm. Piston pumps are capable of processing concrete with a maximum grain size of up to 8 mm, as well. Delivery pressure may reach 40 bar.

When using a carrousel pump (a roll presses against an elastic hose and conveys thus the material) the grain diameter is limited to 1.5 - 2 mm only. The generated pressure, too, is slightly lower than in case of the other two machine types. Of course, the attainable final strength itself is low accordingly.

Owing to the fact that the required compressors are very small, they have already been integrated in the pump in some cases (e.g. Turbosol Mini). For other cases (e.g. PFT ZP 3) it is possible to purchase upon request the corresponding compressor together with the machine.

12.2.3. Other Methods

12.2.3.1. Manual Processing

Masons apply the mortar in a conventional way. This case given, the performance is very low. Using this method makes sense for the application of thin final layers (5 - 10 mm) only. The fabrication of an entire concrete layer according to this method is a useful alternative in case of low wages only.

12.2.3.2. Hopper Guns

With this tool the concrete is filled into a hopper (5 - 10 l) from where it is shot with compressed air through the nozzles to the wall or slab. Since concrete is not pumped through a hose it is possible to neglect pumpability. Compared with manual application performance is considerably improved. The output, however, is still below the one achieved by shotcrete pumps.

Experience has shown that the daily output per hopper gun can be reckoned to be about 100 m^2 in case of a 2 cm thick concrete layer on the wall. For the bottom side of a slab 70 m^2 per hopper gun and day can be estimated.

12.2.4. Application

Concrete is applied always in at least 2 or 3 layers. The first layer gives the structure the required strength to transfer loads during construction. Thus it is possible to concrete the slab above and to possibly keep on erecting the walls of the next storey. The first layer is at least 2 cm thick (metered up to the panel's cover mesh). This concrete layer is screeded manually to avoid irregularities, but apart from that it is kept rough.

When applying dry-mix concrete it is useful to fabricate a thicker first layer and to limit the second layer to 10 to 15 mm. In this case the thin second layer serves to produce a smooth surface only. Basically, dry-mix concrete should not be refinished. That's why it is not advisable to produce the final layer with this method.

Another function of the first concrete layer is the sealing of small gaps. In a slab, for instance, there may be 4 - 5 cm wide slots in the shear reinforcement's area. Gaps of this width may be sealed with shotcrete by applying concrete to the side faces of the EPS.

12.2.4.1. Measuring of Layer Thicknesses

Measuring the concrete thickness is problematic especially for the first concrete layer. Its thickness has to be assessed in relation to the EPS' surface. However, in this stage the panel is not yet completely rigid and the measurement of the concrete layer thickness is somehow uncertain.

Measuring is simple if the first concrete layer has to reach the cover mesh. This implies a 2 cm thick concrete layer that is already sufficient to carry most loads during construction.

When using a thicker first layer with, for instance, a thickness of 4 cm, it is necessary to insert a well visible marking, such as nails that are pinned into the EPS and tied to the cover mesh. Their heads lie 4 cm above the EPS.

Another possibility are tensioned steel wires (known also as piano wires). These are tensioned horizontally with spacings of about 1.0 m and help the workers to screed the concrete roughly.

The second concrete layer is applied like conventional plaster within markings and screeded by a mason. If necessary, it is possible to apply onto this even face also a third thin and smooth layer. This layer, however, does not have any structural functions and, therefore, it must not be made from concrete. It is highly recommended to apply this dry mortar layer on the still moist surface of the second concrete layer. If the surface is not moist enough, wet it thoroughly first.

12.2.4.2. After-treatment

Very thin concrete layers are very sensitive to dry-out. Therefore a thorough after-treatment is essential. This includes at least regular wetting in the first week of construction. Covering the walls exposed to the sun (in the South and West) with a plastic foil is highly recommended. Another alternative is to use wet jute blankets. Otherwise an insufficient after-treatment leads to a premature dry-out and cracks.

Wash the surface of the first concrete layer thoroughly before you keep on concreting. Otherwise bonding of the two layers is not ensured. This risk is of special significance in view of considerable formation of dust when applying the dry-mix method.

Apart from the traditional methods the industry provides for the following:

- Application of an evaporation protection (curing) compound (Masterkure 112)
  When an evaporation protection compound is applied, sprinkling with water is no longer required. However, the applied curing agents have to be removed by means of steam before applying the next layer of shotcrete. These time-consuming procedures hinder the working sequence and contribute to an increase in costs.
- Internal curing (Meyco TCC 735)
  TCC 735 is a new, practical and cost-effective alternative for curing of shotcrete. The advantages of this new technology are impressive:
  - The time-consuming application and - in case of shotcrete applied in several layers - removal of the curing agents is no longer required.
  - Curing is guaranteed from the start of hydration on.
  - The bond strength between two layers is not affected

12.2.5. Top Concrete Layers on 3D Slabs

After the application of the first concrete layer to the walls and to the bottom side of the slab it is necessary to make the shuttering for the top concrete layer of the slab. These shutterings are required both in the area of the ring beam and in the joints between the walls and the slab. In most cases it is enough to fix the shuttering to the reinforcement by tie wire. However, for some rare cases only an additional shoring is necessary. Afterwards, concrete is applied on the top side of the slab. This concrete is applied as conventional concrete with a maximum grain size of 8 to 16 mm. Owing to the minimum clearance of the panels it is sufficient to meter the thickness of the top concrete layer directly from the upper edge of the EPS. Variations are limited to a couple of millimeters only and can be leveled in the finished building by the floor construction.

The mix ratio shown in table 12.2.5.a is sufficient at least for concrete grade B25 ($fc = 17.5\ N/mm^2$). The actual concrete grade depends also on the sieve curve of the aggregate and has to be determined by tests. However, an inferior concrete grade should not be used for the top concrete layer of a 3D slab.

aggregate 0-16 mm	1,725.0 kg
cement	350.0 kg
water	175.0 kg
total	2,250.0 kg

table 12.2.5.a Mix ratio for concrete grade B25

12.3. Problems Preventive

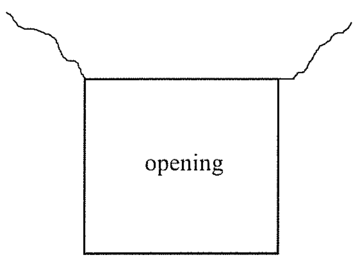
Occasionally some problems may occur in practice when the concrete is processed. These may lead to cracks. In very rare cases only cracks in concreted components are caused by general structural problems. The most frequent reason for such cracks is shrinkage or premature dry-out of concrete. Improper handling of the surface, as well, may lead to cracks. However, these cracks are mainly optical problems only. Really dangerous cracks are the exception and can be prevented usually by well-aimed countermeasures.

Paragraph 12.3.1 shows a list of some types of cracks, their causes and special remedial action. An examination of general measures to improve concrete quality is depicted in paragraph 12.3.2.

A prerequisite of all examples is, of course, that the construction has been made correctly and that particularly the panel joints are overlapped by splice mesh.

12.3.1. Some Types of Cracks

1.



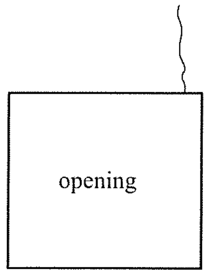
Type: 45°-crack; in some areas only 1 to 2 cm long.

Location: Corners of openings.

Reason: In general, these cracks are caused by shrinkage of concrete with a too high water or cement content. A bigger number of cracks on the sunny side is caused by insufficient treatment of the concrete after applying. If they are only 1 to 2 cm long, they most likely occurred during the installation of the window. These short cracks are found only in the thin mortar layer that covers the window's foaming seal.

Measures: Besides all general ways to improve concrete quality a solution to the problem may be found in a two-stage procedure, i.e. an area (approx. 30×30 cm) in the corners is kept free for the moment. Apply then a very dry mortar with a lower cement content by hand after shotcreting. Of course it is necessary to install the 45° mesh in the corners. Replacing this mesh by concentrated rebars does not improve the situation and can be considered satisfying in case of walls with a 50 mm concrete layer only.

2.



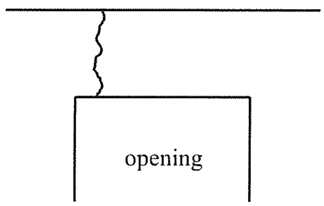
Type: Vertical crack in lintels.

Location: Close to the corners of an opening; sometimes in the area where the panels of the lintel are spliced.

Reason: When the concrete has already been applied to the lintel but is still wet, the panel cannot bear the load of the concrete and the lintel will sag.

Measure: If possible, door or window lintels should be made from one piece without any joints. It is necessary to support the lintel's panel before applying the concrete.

3.



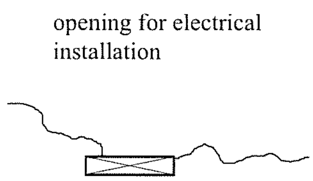
Type: Vertical crack as described in item 2.

Location: Lintel without a 3D slab; close to the corner of a door opening.

Reason: A door lintel without a 3D slab is not as stiff as a lintel connected to a 3D slab. The reason of the crack may be the same reason as described in item 2. In addition, slamming the door results in critical shocks in this area, as well, and may lead to a rather wide crack.

Measure: Door and window lintels without a 3D slab need to be reinforced by at least 2ø12 rebars on the top and bottom.

4.



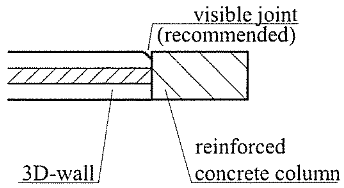
Type: Cracks in the area of openings.

Location: Openings for electrical installations in walls and slabs.

Reason: In general, the same reasons as shown under item 1. Due to the smaller size of these openings the number of cracks is considerably lower than in the window and door corners. In some cases these cracks may be caused by chiseling after applying the concrete.

Measure: In general, the same measures as described under item 1. A great number of these cracks indicates a serious problem with concrete quality or after-treatment. Chiseling should be avoided as long as the concrete is still young.

5.



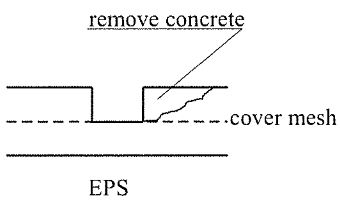
Type: Vertical crack in the wall.

Location: Connection between reinforced concrete column and 3D wall.

Reason: Different materials and a straight joint between the reinforced concrete column and the shotcrete. Reinforced concrete columns and 3D-walls show a different deformation behaviour.

Measure: The connecting reinforcement at the joint must consist of stirrups to absorb structural forces and additional splice mesh. It is recommended to provide also for a visible joint.

6.



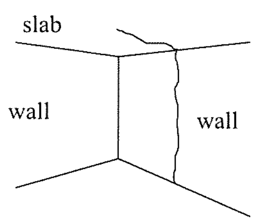
Type: Vertical crack in the wall.

Location: They occur in those places where the metal guides for plastering have been attached.

Reason: The metal guides were placed to measure the thickness of the conrete layer. After applying the concrete these guides were removed and the gap was sealed by concrete. These cracks are not real cracks. Owing to the fact that concrete has no adhesive properties, this joint will open even if shrinkage stress is very low.

Measure: A part of the concrete has to be removed to produce a rough and inclined surface (sketch, right-hand side).

7.



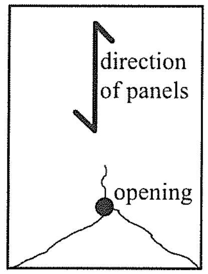
Type: Vertical crack in the wall that may reach into the slab.

Location: In walls and slabs.

Reason: The 3D wall panels weren't properly anchored to the foundation and slipped on account of the wet concrete's load. Such a crack may appear quite recognizably after applying the slab load.

Measure: Before applying the concrete it is necessary to anchor the panels properly in the foundation. If there are openings in the foundation below the wall it must be made sure that the panel is fastened properly.

8.



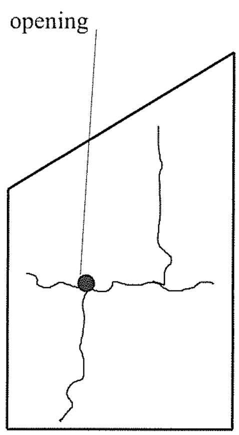
Type: Crack at the bottom side of a slab.

Location: In the area of main tensile stresses. In most cases they start from the openings for electrical installations.

Reason: The panels were placed in the longer direction of the slab. If the slab shape is almost a square, cracks will most likely occur. They mostly start from the openings for electrical installations.

Measure: The panels and additional reinforcement have to be placed always in the shorter direction of the slab. If the slab shape is almost a square, a negative reinforcement will reduce deflection. The openings can be handled as described in item 1.

9.



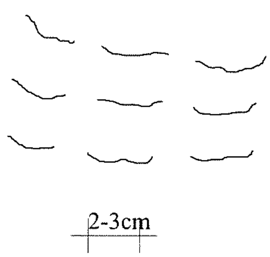
Type: Orthogonal cracks in walls and slabs.

Location: At any places in walls and slabs. These cracks start frequently from the openings.

Reason: Obviously these cracks are caused by shrinkage stresses. Owing to insufficient stiffening for shotcreting horizontal cracks in the panel joint may occur in tall walls that were abutted vertically.

Measure: Basically it is possible to minimize the number of these cracks by using a very dry material for the last concrete layer and by constantly wetting the concrete in the first week. Tall walls have to be braced properly before shotcreting to avoid vibrations.

10.



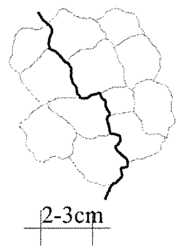
Type: Short horizontal cracks in walls.

Location: In the middle of the wall area.

Reason: The mason tried to smooth the surface after the concrete has already started to set.

Measure: The mason's job was not done correctly. The concrete work has to be finished before the concrete starts to set. The use of a plasticizer may be taken into consideration to extend this period.

11.



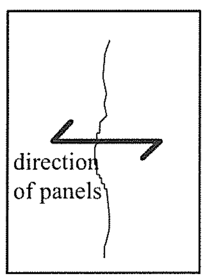
Type: Cracks in walls and slabs surrounded by clearly visible honeycomb-like structures.

Location: Somewhere on walls or slabs.

Reason: The honeycomb-like structure indicates the drying out of the concrete. A crack occurs in some parts of this structure. This clearly visible structure becomes evident only in case of concrete with a very high water content.

Measure: It is necessary to produce a concrete with a lower water/cement ratio. In some cases the mason smoothes the final layer with a steel trowel. However, this trowel sucks water up to the surface during lifting. Thus the water content becomes too high right at the surface. Therefore, wood or plastic trowels are to be preferred.

12.



Typ: Crack in cross direction of the slab.

Location: Approximately at midspan.

Reason: The shoring has been removed too early. When the crack appeared the shoring was placed again and the slab was cambered subsequently. By this subsequent cambering the width of the crack was enlarged only.

Measure: At least the middle row of shoring has to be kept until the concrete achieves its full strength. Cambering of the slab after applying the concrete doesn't make sense.

12.3.2. General Measures to avoid Cracks

- The concrete shall be as dry as possible to work with it. The best mixture (also premixed dry concrete in bags) can be ruined by adding too much water. The optimal water/cement-ratio is approximately 0.5. Values higher than 0.6 are to be avoided. To measure the w/c-ratio it is definitely recommended to use buckets. Adding water by feeling only is no sufficient way. If in some cases the concrete pump will not work properly a liquidizer shall be taken into consideration.

- The total amount of cement shall not exceed 300 kg/m³. With washed aggregate (sand from a river) the pumpability is not ensured. To increase the pumpability, admixtures like SikaPump are recommended. According to the manufacturer's information these additives increases the pumpability without any negative influence on the strength of the concrete. The adding of fine particles made out of crushed stones (stone dust) is possible and widely done, as well.

- The application of 3 layers of concrete is a sufficient way to avoid shrinkage cracks on the surface of the walls and slabs. The first two layers have to have the thickness required for structural purposes. The third layer is only a thin layer (some millimeters) made of dry mortar. If some cracks already exist the thickness of this final layer has to be increased to prevent these cracks from becoming visible. The final layer doesn't have to be concrete. A lime-cement mortar will be cheaper and easier to use. Especially in case of a thicker final layer in places where the panels are not exactly alligned, a lime-cement mortar is recommended.

- To be on the safe side it is possible to leave the most critical areas free of concrete and to apply a dry mortar manually after shotcreting. These critical areas are the corners of openings and possibly the small openings for electrical installations.

- The after-treatment of the surface is one of the most important measures to keep the concrete free of cracks. A considerably higher number of cracks in walls exposed to the sun shows clearly that the concrete was not kept wet enough during the first week (see also item 12.2.3.2). If the concrete dries out too quickly shrinkage stresses will increase and, as a result, cracks will most likely occur. Good after treatment means that the wall is wetted every two to three hours in the first three days after application, and at least twice a day for the rest of the week. Concrete applied short before weekend has to be covered by means of a PVC foil. This is important in particular for the second concrete layer of walls exposed to the sun.

- Another reason for cracks in some areas of a building is chiseling while the concrete is still young. The number of cracks corresponds to the scale of the chiseling work and the age of the concrete. Before the concrete is 7 - 10 days old chiseling jobs are to be reduced to a minimum. In general, heavy vibrations are slightly more dangerous for thin concrete layers than for plaster on a brick wall.

- The use of plastic fibers in the concrete may reduce the risk of cracks. Steel fibres are not recommended. They can cause a corrosion problem. Besides, the manufacturer of plaster pumps and mortar pumps recommend only plastic fibers but not steel fibers. However, this cannot be considered a cure-all to solve all problems. For practical reasons, the length of the fibers shall not exceed 15 - 20 mm. The number of fibers should lie between 0.8 and 1.0 kg/m³ concrete. However, with that length the admissible tensile strength of the concrete increases only slightly and an unfavorable mix-ratio and/or after-treatment will still lead to cracks. Longer fibers would have a bigger influence on tensile strength, but it is also more difficult to work with this concrete. After finishing the smooth surface, longer fibers could become visible and would be rather disturbing. On the other hand, the costs of the concrete will increase as well.
  Besides, all fibers lead to a slight decrease of pumpability of the material. If pumpability is already critical, additional fine particles (e.g. cement) would have to be added to maintain the pumpability.
  In general, a worm pump can be used in case of a fiber mortar with a maximum grain size of 2 - 4 mm. From a maximum grain size of 8 mm only piston pumps are a better alternative since worm pumps have got basic problems in pumping material of this grain size.
  As a rule, the properties of fresh concrete will change only insignificantly. If

pumpability is already critical, tests only will show whether concrete of this composition can still be pumped.

As a summary, 3 very important items have to be mentioned for keeping the concrete almost free of cracks:

1. The content of cement shall not exceed 300 kg/m^3.
2. The water/cement ratio has to be between 0.5 and 0.6.
3. The surface has to be kept wet for the first few days.

Appendix

A.1. Concrete Grades according to DIN

DIN 1045		Concrete Grades B15	B25	B35	B45	B55
fc		10.5	17.5	23.0	27.0	30.0
modulus		26,000	30,000	34,000	37,000	39,000
f_r		2.0	2.7	3.2	3.8	4.3
Slabs	τ_{011}	0.25 0.35	0.35 0.50	0.40 0.60	0.50 0.70	0.55 0.80
	τ_{02}	1.20	1.80	2.40	2.70	3.00
Beams	τ_{012}	0.50	0.75	1.00	1.10	1.25
	τ_{02}	1.20	1.80	2.40	2.70	3.00
	τ_{03}	2.00	3.00	4.00	4.50	5.00

table A.1.a Concrete grades according to DIN

i.e. fc ....specified compressive strength
modulus.......modulus of elasticity
f_r ....modulus of rupture
τ_{011}, τ_{012}, τ_{02}, τ_{03} ......limits of shear range 1, 2 and 3

All values are in N/mm².

A.2. Excel Files

The following files were drawn up with "Excel 4" and, to avoid any conversion problems, they were made out with very simple functions only. Especially the power function turned out to be a problem sometimes.

In the calculation sheets the values that need to be modified by the user are written in red. In the print-outs shown in this section these values are written in italic style. The calculated values are written in black.

The Excel files are subject to changes. Therefore the actual working sheets may differ slightly from the print-outs shown in this section.

A.2.1. Shear Strength of the Panel

In the calculation sheets SHEA_DIN.XLS and SHEA_ACI.XLS the following values are to be modified by the user:

e-horiz. .... Horizontal distance between the welding points of a diagonal (see figure 3.1.b).
100 diagonals per m² e-horiz. = 60 mm
200 diagonals per m² e-horiz. = 40 mm
step .... Distance between diagonals in lengthwise direction (see figure 3.1.b).
z/d....................... Relative size of internal lever arm referring to effective depth. The value 0.95 is recommended.
f,welding .... Maximum joint strength; In general the maximum strength of the welding joint is assumed to be 15.0 kN/cm².
safety against buckling.......... 2.05 under service load (DIN)
strength reduction factor .... 0.85 under factored load (ACI)
point of inters. .... 1=Yes, 0=No; Shall the restriction of figure 3.5.a, left side, apply? If the answer is yes, then the lever arm is limited with the distance between the centroid of tensile reinforcement and the diagonals' theoretical point of intersection.
lg_e/lg,diag .... Ratio between buckling length and free length of the diagonals. The value 0.75 is recommended.
mesh-EPS........... Clear distance between cover mesh and EPS.
ø cover mesh ...... Diameter of the cover mesh.
ø diagonal........... Diameter of the diagonals.

In view of the limitation of buckling stress due to a joint strength of 15 kN/cm² the values within the elastic zone ($\lambda \geq 75$) are taken into consideration only. Therefore a joint strength (f,welding) higher than 17.5 kN/cm² will lead to wrong results.

SHEAR STRENGTH OF 3D PANELS					
e-horiz [mm]		*40*	*60*	z/d =	*0.95*
step [mm]		*100*	*200*	f,welding =	*15*
inclination [°]		73.3	65.7	safety against buckling =	*2.05*
EPS	*100*	diagonals per m²		point of inters. =	*1*
concrete	layer	200	100	lg,e/l,diag =	*0.75*
50	mm	14.3	9.8	mesh/EPS [mm] =	*13*
60	mm	14.3	10.3	ø cover mesh [mm] =	*3*
70	mm	14.3	10.9	ø diagonal [mm] =	*3.8*
80	mm	14.3	10.9	EPS/center of mesh [mm] =	16.5
λ =		82.4	86.6		
fk =	[kN/cm²]	14.59	13.22		
F,diag =	[kN]	1.65	1.50		

table A.2.1.a Calculation sheet “Shear Strength of 3D Panels”

A.2.2. Design of Walls

In the calculation sheets WALL_DIN.XLS and WALL_ACI.XLS the following values are to be modified by the user:

eccentricity......... Eccentricity under service load.
safety factor........ For the calculation according to DIN we recommend safety factor 3.0.
β .... ratio between factored dead load and factored total load according to ACI
dimensions .... Thickness of concrete and EPS in [mm].

Calculation according to DIN is carried out up to a slenderness of $\lambda = 70$ as per paragraph 4.2.1. For a slenderness higher than 70 the method described in paragraph 4.2.2 is applied. The concrete grades B20 and B30 which are mentioned in the table, do not accord with DIN but they are widely used.

The table shows the loadbearing capacity of walls in kN/m.

LOADBEARING CAPACITY OF 3D WALLS ACCORDING TO DIN 1045					
eccentricity :	30	mm	area of concrete =	1000	[cm2/m]
safety factor :	3		center of gravity =	10	[cm]
concrete outside :	50	mm	moment of inertia =	58333	[cm4/m]
EPS :	100	mm	radius of inertia =	7.638	[cm]
concrete inside :	50	mm	max eccentricity =	7.5	[cm]

Concrete grade				Admissible axial load P [kN/m] for 3D walls Effective lenght of wall [m]									
				2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	4.75	5.00
B15	fc=	10.5	N/mm²	196	188	180	173	165	157	150	142	134	127
B20	fc=	14.5	N/mm²	270	260	249	238	228	217	207	196	185	175
B25	fc=	17.5	N/mm²	326	313	301	288	275	262	249	237	224	211

B10	fc=	7.0	N/mm²	
B15	fc=	10.5	N/mm²	
B20	*fc=*	*14.5*	*N/mm²*	*The concrete grades B20 and B30 do not*
B25	fc=	17.5	N/mm²	*accord with DIN but they are widely used.*
B30	*fc=*	*20.0*	*N/mm²*	
B35	fc=	23.0	N/mm²	
B5	fc=	3.5	N/mm²	

table A.2.2.a Calculation sheet "Loadbearing Capacity of 3D Walls"

A.2.3. Design of Beams

The calculation sheets BEAM_DIN.XLS and BEAM_ACI.XLS calculate beams made of 3D panels and additional reinforcement. The share of the panel‘s cover mesh has not been taken into consideration for flexural design (i.e. the panel is spliced). The shear design is carried out with the shear strength provided by the reinforcement only. Owing to the fact that small openings for electrical installations hardly effect the compression strut in the concrete but reduce the capacity to resist the inclined main tensile stresses, the shear strength provided by the concrete was neglected.

The maximum effective depth for design corresponds to the 0.5-fold length (DIN) or the 0.4-fold length (ACI) of the beam. Deep beams are not to be designed according to these tables.

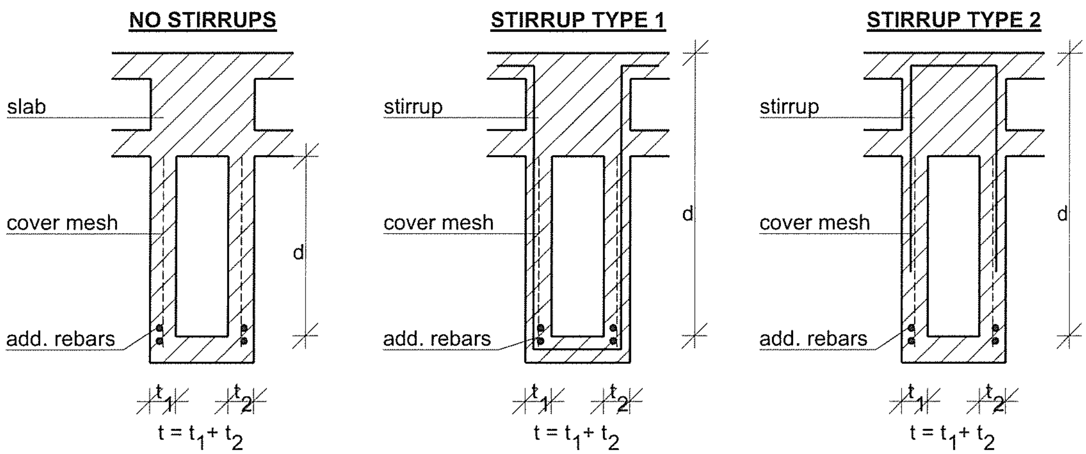


figure A.2.2.a Section of a slab-beam structure

The input values include steel grade and concrete grade, the dimensions of the beam and the number, type and diameter of the stirrups. The width of the concrete shells (=t) always refers to the total width of both shells (=t_1+t_2). If there are no stirrups the type has to be set to 0. In this case the effective depth has to be assumed to reach up to the upper edge of the panel and not to the upper edge of the slab. In case of an arrangement as per figure A.2.2.a, right side (stirrup type 2), the stirrup may not be considered additionally because it is seen as an elongation of the cover mesh only.

The tables show the maximum admissible load (DIN) or the ultimate load (ACI) in kN/m.

3D BEAMS ACCORDING TO DIN 1045

DIMENSIONS	clear length of beam	lg =	2.00	m
	width of the concrete shells	t =	10.00	cm
	concrete grade		B25	
REINFORCEMENT	cover mesh	ø =	3	mm
		e =	50	mm
	stirrups	ø =	8	mm
	type = 2	e =	200	mm
	steel grade	cover mesh =	500	N/mm²
		add. rebars =	420	N/mm²

MAX. ADMISSIBLE LOAD [KN/M]

add. rebars				Effective depth of the beam [cm]							
pcs.	/	ø[mm]	[cm²]	30	40	50	60	70	80	90	100
2	ø	8	1.01	11.0	14.8	18.6	22.4	26.2	30.0	33.8	37.6
2	ø	10	1.57	16.7	22.6	28.6	34.6	40.5	46.5	52.4	58.4
4	ø	8	2.01	20.9	28.6	36.2	43.8	51.4	59.1	66.7	74.3
2	ø	12	2.26	23.3	31.9	40.4	49.0	57.6	66.2	74.8	83.3
4	ø	10	3.14	27.1	38.4	51.2	65.8	78.7	90.6	102.5	114.4
4	ø	12	4.52	27.1	38.4	51.2	65.8	82.6	102.3	125.6	153.5
6	ø	12	6.79	27.1	38.4	51.2	65.8	82.6	102.3	125.6	153.5

Concrete (DIN)

W28	fc	τ03
	[kN/cm²]	[kN/cm²]
B15	1.05	0.20
B20	*1.45*	*0.25*
B25	1.75	0.30
B30	*2.00*	*0.35*
B35	2.30	0.40
B45	2.70	0.50
B55	3.00	0.60

cover mesh :	1.41	cm²/m
stirrups :	2.51	cm²/m
horiz. shear force :	141.37	kN/m
concrete grade	B25	
fc =	1.75	kN/cm²
τ03 =	0.30	kN/cm²

B20 and B30 are not defined in DIN 1045, but they are widely used

table A.2.3.a Calculation sheet "Loadbearing Capacity of 3D Beams"

A.2.4. Design of Slabs

The design of slabs will be carried out by the working sheets SLAB_DIN.XLS and SLAB_ACI.XLS. The data about the dimensions and the shear strength provided by the panel have to be determined as a constant value by the user. The concrete grades are different for the bottom and top concrete layers. At least the determination of the negative reinforcement is required additionally. Due to the fact that a 3D slab is preferably carried out as a simply supported member the negative reinforcement has to be assumed by the user. This reinforcement is not taken into consideration for the design of the tensile reinforcement. Design is split up into 3 parts.

- Flexural design
  Generally the design is carried out as a simply supported slab. The negative reinforcement is not accounted for. In case of a calculation according to DIN and ACI the limit for flexural strength is determined as per section 2.5.
  The tables show the required additional reinforcement.
- Shear design
  Shear design takes place in several steps. If the shear strength of the panel is exceeded one or two stirrups made of U-shaped splice mesh are added to the panel. If this is not sufficient a reinforced concrete girder is placed between the panels.
  The table shows the number of U-meshes or the required stirrups and, in the line below, the required length of the additional shear reinforcement.

3D-slab without girders

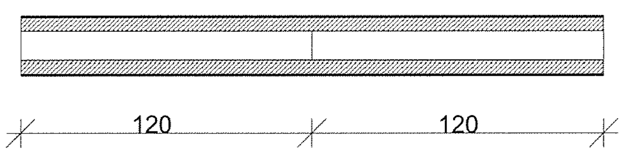
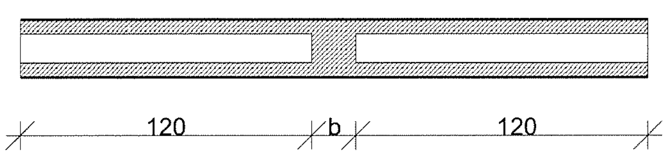


figure A.2.4.a Section of a 3D slab with and without girder

3D-slab with girders



- Deflection

  Deflection is computed according to the curvature method (DIN) or by the method described in ACI. The curvature method is carried out according to section 6.1. In both cases the share of shear deflection of a panel with 100 diagonals per m² (ø 3.8 mm) can be added.

  If the admissible deflection is exceeded the slab will be restrained according to the assumed negative reinforcement. However, the fixed-end moment must not exceed the following values:

  one side is restrained $M_{MIN} = -ql^2/8$

  two sides are restrained $M_{MIN} = -ql^2/12$

  The tables show the number of restrained edges that are necessary to observe admissible deflection.

3D SLABS ACCORDING TO DIN 1045

SLAB	concrete on top side :	*60*	mm
	EPS :	*100*	mm
	concrete on bottom side	*50*	mm
	concrete grade top:	*B25*	
	concrete grade bottom:	*B15*	
LOADS	slab :	2.75	kN/m²
	floor construction :	*2.50*	kN/m²
	dead load :	5.25	kN/m²
	live load :	*4.75*	kN/m²
	total load :	10.00	kN/m²
REINFORCEMENT	ø - cover mesh:	*3*	mm
	size - cover mesh:	*50*	mm
	steel grade-cover mesh	*500*	N/mm²
	steel grade-add. rebars:	*500*	N/mm²
	upper reinforcement:	*2.51*	cm²/m
	adm. deflection:	length /	*200*

Clear length [m]	*2.50*	*3.00*	*3.50*	*4.00*	*4.50*	*5.00*	*5.50*
Moment Mu [kNm/m]	9.11	12.80	17.11	22.05	27.61	33.80	40.61
Bottom reinforcement	ø 6 / 30	ø 6 / 24	ø 8 / 24	ø 8 / 15	ø 8 / 10	ø 10 / 12	ø 10 / 10
Shear force [kN/m]	12.50	15.00	17.50	20.00	22.50	25.00	27.50
Shear reinforcement	panel only	1 U-mesh	1 U-mesh	1 U-mesh	2 U-mesh.	2 U-mesh.	ø 8 / 20
Length of shear reinf.[m]	-	0.07	0.32	0.57	0.82	1.07	-
max. Deflection [cm]	0.38	0.74	1.75	1.42	2.31	2.58	not allowed
Upper reinforcement	-	-	-	1 side	1 side	2 sides	not allowed

table A.2.4.a Calculation sheet "Design of 3D Slabs"

Reference

Among others, the theoretical principles were taken from the following works of reference:

DIN 1045
ACI 318/89 - documented version
ÖNORM series B4200 and B4600

Leonhardt - *Vorlesungen über Massivbau* (Lectures on Reinforced Concrete Constructions)
Bareš - *Scheiben und Platten* (Diaphragms and Slabs)
Böhm, Geiger, Valentin - *Stahlbetonbau* (Reinforced Concrete Constructions)
Lohmeyer - *Stahlbetonbau* (Reinforced Concrete Constructions)
Issues of the Magazine of the *Deutscher Ausschuß für Stahlbetonbau* (German Commission for Reinforced Concrete Constructions)
Ferguson, Breen, Jirsa - Reinforced Concrete Fundamentals

The data of the different tables and diagrams were determined on the basis of EVG calculations and tests. The tests were made in the framework of different test series between 1985 and 1997 mainly at the Graz Institute of Technology. Various other tests were made at research centers in Austria and abroad. These, among others, include tests on the resistance to earthquake forces at the University of Bogotá, Colombia.

The model calculations relating to the load-bearing behavior of 3D cross sections subject to single loads were made with the program *Allplot* by *Nemetschek*, Munich.

The detailed drawings were made with the program *MegaCAD* by *MegaTech*, Berlin.

The basic data of the diagrams of chapter 6 (deflection) were determined by EVG programs.

Diagrams and tables were drawn up with the program *Microsoft Excel.*

The Manual on hand emphasizes primarily the design differences between 3D components and load-bearing structures made of traditional reinforced concrete. Therefore, the knowledge of traditional design rules for reinforced concrete construction is a prerequisite.

The Manual's structure and the included examples were drawn up mainly on the basis of questions asked by users of the 3D system. All examples form a representative cross section of practical 3D design.

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