Demountable Construction
Analysis of the behaviour of a
1:5 scale floor bay: Part 1

Series C: Monotonically increasing and repeating load

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DEMOUNTABLE CONSTRUCTION

Analysis of the behaviour of a 1:5 scale floor bay;
Part 1
Series C-monotonically increasing and repeating load

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\( A_c \) cross-sectional area of concrete
\( A_s \) cross-sectional area of steel
\( E_c \) modulus of elasticity of concrete in compression
\( E_s \) modulus of elasticity of steel in tension
\( EI \) stiffness
\( F \) point load
\( H \) horizontal force
\( M \) bending moment
\( N_c \) direct compressive force in concrete
\( N_s \) direct tensile force in steel
\( Q \) total horizontal load
\( R \) reaction force
\( V \) shear force
\( V_u \) ultimate shear force
\( f_c \) design value of compressive strength of concrete
\( f_s \) design value of tensile strength of steel
\( n \) number of cycles
\( s \) shear deformation
\( w \) crack width
\( x \) depth of compressive zone
\( \delta \) deflection
\( \varepsilon_c \) strain of concrete
\( \varepsilon_{cy} \) strain of concrete at \( f_c \)
\( \varepsilon_{cu} \) maximum compressive strain of concrete
\( \varepsilon_s \) strain of steel
\( \varepsilon_{sy} \) strain of steel at \( f_s \)
\( \varepsilon_{su} \) maximum strain of steel
\( \varphi \) rotation
\( \chi \) curvature
\( \sigma_c \) normal stress occurring in the concrete
\( \sigma_s \) normal stress occurring in the steel
\( \tau \) shear stress
\( \mu \) quotient of shear stress to normal stress \((=\tau/\sigma)\)
SUMMARY

With a view to the rational use of energy and raw materials, on the one hand, and having regard to the environmentally detrimental effects of demolition (nuisance caused by demolition work, problems with the disposal of rubble), on the other, it will have to be established by research to what extent demountable construction methods offer a meaningful alternative to the methods now in current use.

By "demountable construction" is understood a building method characterized by the fact that the structural connections are so designed that the component parts of the structure can be dismantled (demounted) intact or with only minimal damage and are, if possible, suitable for re-use.

Within the context of such research the behaviour of a demountable floor bay, constructed to a 1:5 scale, was investigated in the Stevin Laboratory of the Delft University of Technology. This floor bay was composed of reinforced concrete hollow core slabs supported on beams which in turn rested on columns.

The results of the research, which was carried out in collaboration with CUR-VB Committee D7 "Demountable construction", are given in Stevin Report No. 5-84-4 [1].

The present report gives an analysis of the results of the measurements relating to part of the research, namely, tests performed in series C with a monotonically increasing and with a repeating load applied within the plane of the floor bay.

A general introductory chapter is followed, in Chapter 2, by an examination of the action of the forces and the deformations in a floor bay composed of precast concrete units. Chapter 3 then reviews the simplified mathematical models that may suitably be used for a structural analysis. The variants of
these models are elaborated in Appendices B, C and D.

Next, on the basis of a comparison with the results of the measurements, a choice was made from among these variants. The results of the calculations in Appendices B to H are assembled and summarized in tables and graphs in Chapter 4.

In Chapter 5 the values calculated in this way for a number of relevant patterns of behaviour are compared with the measured values. An analysis of the behaviour of the floor bay is then given in Chapter 6.

From the analysis it emerges more particularly that the shear transmission of force at the joints is of major significance in determining the behaviour of the floor bay.

At a transverse joint with predominantly bending moment and a small transverse (shear) force, the measured and the calculated values of the longitudinal forces are found to be in good agreement. But at joints where the transverse (shear) force predominates, the measured values of the longitudinal forces differ considerably from the calculated values.

The loadbearing capacity of the floor bay is found to be determined to a considerable degree by the shear capacity of the joints and therefore by the roughness of the side faces of the floor units.
1. INTRODUCTION

In 1977 the CUR-VB initiated the development of structures that are simple to dismantle or demount and therefore referred to as "demountable structures". In the then following years a large number of tests were performed on a floor bay consisting of precast concrete units. This floor structure is characterized by the fact that the forces are transmitted solely via the joints and with the aid of simple connections between the units, not through in-situ concrete or an in-situ concrete topping.

The "diaphragm action" of this floor - constructed to a 1:5 scale - under horizontal loading in its own plane was investigated. The set-up and execution of the research are described in Stevin Report No. 5-84-4 [1], which also contains an overview of the results obtained in the tests.

The object of the research was to gain more insight into the pattern and action of the forces in a floor bay consisting entirely of precast concrete units. The calculation of the internal forces and the deformations of such a structure is highly complex. An accurate analysis is impracticable without the aid of a computer.

All the same, for estimating the dimensions of the precast units, the forces acting in the connections and the loadbearing capacity of the floor bay at the design stage, and for analysing the stability of the building, the designer requires a simple calculation method that is sufficiently reliable for preliminary use at this stage.

For employing a computer program for the final calculations the designer should have at his disposal sufficient input data to enable the behaviour of the structure to be determined. Also, at that stage of the work he will require a simple method of calculation for verifying the validity of the results yielded by the computer.
The aim of the present report is to contribute to finding the answers to the questions arising from these considerations. At the same time it must be realized that, having regard to the nature of the research, this report can do neither more nor less than add to the understanding of the behaviour of a rigid floor bay. In order to proceed from here to generally-valid design rules will require, on the one hand, detailed research into the transmission of force in joints between precast floor slabs and, on the other, the development of a theoretical model with which such transmission of force can be described.

In this report it will be investigated to what extent existing theoretical models such as those which have been developed by other investigators — Walraven, Tassios, and others — are applicable to these tests.

It is perhaps needless to add that, although the tests were performed against the background of the design of demountable structures, the results are valid also in the context of precast concrete construction in general, where rationalization and efficiencies are of primary importance.
2. ACTION OF FORCES AND DEFORMATION IN THE FLOOR BAY

The detailing of a floor bay, which must be able to transmit the horizontal loads to the vertical shear walls depends to a considerable extent on the forces acting in that floor bay. Of course, this action of the forces is governed by the manner in which the floor is constructed or assembled. This chapter is concerned with the analysis of the floor bay tested in the Stevin Laboratory and fully described in Stevin Report No. 5-84-4 "Demountable construction. Description and results of tests on a demountable floor bay constructed to a 1:5 scale", January 1984 [1]. The layout and constructional features of the bay are shown in Fig. 2.1.

It is composed of longitudinal beams and transverse floor units (hollow core slabs). The latter rest on strips of felt interposed between them and their bearings on the beams, which in turn bear on the column heads likewise through interposed felt strips. The gaps between the floor units themselves and also those between the slabs and the beams are filled with low-strength mortar.

The longitudinal beams are interconnected by means of threaded steel bars (M6) and anchor sockets (M24) embedded in the ends of the beams. The column heads are supported on the steel testing rig through roller bearings so as to obtain virtually frictionless bearing conditions.

The shear walls along the short sides of the floor bay, on the grid lines 1 and 14, are simulated by steel strips with a degree of rigidity equivalent to that developed in an actual building.

In the tests performed in series C the influence of the following parameters was investigated:
(a) the boundary conditions at the ends of the floor;
(b) the tensile force at the ends of the floor;
(c) the imposed cracking at grid line 2 or 13;
(d) the manner of loading.
Fig. 2.1 Plan and details of the floor bay.
The boundary conditions at the ends relate to the connection of the floor bay to the simulated shear walls along the short sides of the bay. This connection was varied in ten different ways.

In a number of tests a constant tensile force was applied in the longitudinal direction of the floor, as shown in Fig. 2.2. The magnitude of the total tensile force was 2.24 or 3.04 kN.

In a number of tests a crack was deliberately formed over the full length of the joints at grid line 2 or 13 before testing commenced. At grid line 2 the crack width was about 0.3mm and at grid line 13 it was about 0.2mm.

The floor bay was subjected to horizontal loading in its own plane, as shown in Fig. 2.1a. The following modes of load application were applied:
- monotonically increasing load;
- repeating load.

The loads were applied with the aid of pneumatic jacks.

\[ F_2 \quad F_3 \quad F_2 \quad F_3 \]

\[ \text{Fig. 2.2 Tensile forces at ends of floor bay.} \]
More detailed information on these parameters is given in Stevin Report No. 5-84-4 already referred to.

In considering the action of the forces and the deformation the following will be examined:

1. Bending moments (section 2.1)
2. Deflection of the longitudinal edge (section 2.2)
3. Deformation of the ends of the floor (section 2.3)
4. Longitudinal shear forces (section 2.4)
5. Transverse shear forces (section 2.5)

2.1 Bending moments

The horizontal loading produces bending moments in the floor bay. In every conceivable cross-section these moments produce a strain distribution which depends on the kind of materials comprised in the section.

At the grid lines 2 to 13 of the floor bay, where the connections between the longitudinal beams are located, the section consists entirely of joint filling material. If flexural cracks occur, the strain diagram will be as shown in Fig. 2.3b. With the aid of the stress-strain diagrams of the materials employed it is possible to calculate the stress distribution, which is shown in Fig. 2.3c. The longitudinal forces in the longitudinal beam-to-beam connections and the compressive stresses in the extreme fibres can be calculated from the equilibrium conditions $\sum H=0$ and $\sum M=0$.

Between each pair of transverse grid lines is a continuous beam, so that the cross-section of the floor bay will remain uncracked there for a long time. Because of this the strain distribution and the associated stress distribution will be very different from those occurring at the grid lines. These distributions are shown schematically in Fig. 2.4b and c.

At the longitudinal beams the stress distribution exhibits a discontinuity due to the difference in stiffness between these beams, the jointing material and
Fig. 2.3 Strain and stress distribution in cracked section.

Fig. 2.4 Strain and stress distribution in uncracked section.
the floor units. Here too, just as for the sections at the grid lines, the strains and the stresses at the extreme fibres associated with a particular horizontal loading can be calculated.

The two strain distributions which have thus been obtained can serve as a starting point for a "manual calculation" of the horizontal deflection of the floor bay. The approach adopted for the purposes is an approximation in which the shear deformation of the longitudinal joints is neglected.

Since the section is certainly not homogeneous, there is every likelihood that discontinuities ("jumps") occur in the strain distribution at the longitudinal joints. The magnitude of these discontinuities will depend on the magnitude of the shear forces in the longitudinal joints and on the shear deformation of the joint. In order to determine the magnitude of the discontinuities in the strain diagram it is necessary to know the relation between the shear stress, the normal stress, the shear displacement and the crack width of the mortar joint. See Fig. 2.5.

![Fig. 2.5 Strain diagrams for uncracked and cracked sections.](image)
2.2 Deflection of the longitudinal edge

The horizontal deflection of the longitudinal edge of the floor bay depends to a considerable extent on the stiffness of the component parts of the bay. For the purpose of calculating the deflection the floor bay can be schematized as shown in Fig. 2.6. It is conceived as being composed of flexurally rigid segments of 600 mm width between the grid lines and of flexible segments of 120 mm width at the grid lines. This 120 mm width has been adopted because this dimension corresponds to the length of the connecting bars between the longitudinal beams.

![Fig. 2.6 Schematization of the floor bay.](image)

For analysing the flexible segments the calculation may be based on the usual stress analysis at a section, as schematized in Fig. 2.3. The distribution of forces at the section can be calculated on the assumption that plane sections remain plane. It is then possible to determine the curvature $\chi_1$ of the flexible elements: $\chi_1 = \tau_1 / x$.

The distribution of forces can similarly be calculated for the rigid segments between the grid lines (see Fig. 2.4). The longitudinal beams are assumed to be
uncracked and the joints to be unable to transmit tensile forces. The curvature $x_2$ of the rigid segments can then be calculated from the strain distribution.

The horizontal deflection of the longitudinal edge can now be determined from the curvature diagram of the floor bay, as shown in Fig. 2.7. The deflection at point D7 of the edge is equal to the moment due to the shaded area at that point.

2.3 Deformation of the ends of the floor

The deformation of the ends of the floor bay at grid lines 1 and 14 comprises a rotation $\phi$ and a horizontal displacement, as indicated in Fig. 2.7. The rotation can be calculated from the deflection of the longitudinal edge at grid line 2 divided by the distance between grid lines 1 and 2, i.e.,

$$\phi = \frac{\delta_2}{720} \text{ rad.}$$

The horizontal displacement of point D1 is influenced by the elongation of the longitudinal edge and furthermore by the curvature of that edge. Since the elongation of the connection between the longitudinal beams is the dominant influence on the horizontal displacement of D1, the other factors will be neglected in the numerical analysis.

These matters are considered in more detail in Section G1 of Appendix G, and summarized in Section 4.5.
2.4 Shear forces in the longitudinal joints

As shown schematically in Fig. 2.9, the shear forces will be transmitted from the longitudinal beams via the longitudinal joints to the ends of the floor units.

Fig. 2.9 Forces acting in the floor bay.

For calculating the longitudinal shear stresses the floor bay is divided into elements equal in width to the floor units (hollow core slabs), as shown in
Fig. 2.10. The magnitude of the shear forces depends on the difference in force acting on the left and on the right of the element under consideration. As a result of applying a section at a longitudinal joint, e.g., at grid line A, a floor sub-element is obtained (see Fig. 2.10d) in which the shear force $V_{AB}$ is in equilibrium with the normal (longitudinal) forces $N_{c1(l)}$ and $N_{c1(r)}$ on the left and right of this sub-element.

The shear force calculated in this way is assumed to be uniformly distributed over the joint considered. This is of course an approximation, but it will be more accurate according as the floor element chosen for the analysis is smaller. Another reason why this approach is no more than an approximation is that the shear deformation in the longitudinal joints is neglected.

![Diagram of forces acting in a floor segment](image)

The shear forces in the other longitudinal joints can similarly be calculated from the conditions of internal equilibrium.

2.5 Shear forces in the transverse joints

At a joint where both a bending moment and a transverse (shear) force occur the stress distribution will be as shown schematically in Fig. 2.11.
At the grid lines the connection between the longitudinal beams coincides with a transverse joint between the floor units. The shear force must therefore be transmitted entirely by the joint. At a cracked joint this force will be transmitted entirely by the compressive zone. The magnitude of the shear force that can thus be transmitted will depend on the $\tau-\sigma$ relation of the joint construction. If this relation is known, it is possible to calculate the shear force for which shear displacement is just about to occur.

Fig. 2.11 Stress distribution in a transverse joint without shear displacement.

If shear displacement occurs, it will be attended by a widening of the crack in consequence of the wedge action within the rough crack face. However, the increase in crack width will be prevented by the longitudinal connections provided, so that additional tensile forces are produced in these connections. These in turn produce additional compressive forces in the joint, with the result that the shear resistance is increased. This phenomenon is here called "shear friction".

The stress distribution at the transverse joint could then be as shown schematically in Fig. 2.12.

Determining the actual stress distribution by "manual" calculation is no longer practicable, since the $\tau-\sigma-s-w$ relation depends on the initial crack width and
this width varies over the section. For this reason the stiffness with respect to shear will vary over the length of the section. The shear stress can be expected to be concentrated at the original compressive zone because the shear stiffness is greatest there.

Fig. 2.12 Stress distribution in a transverse joint with shear displacement.
3. VARIANTS FOR A STRUCTURAL ANALYSIS

In Chapter 2 the action of the forces and the deformation in the floor bay have been analysed on the basis of theoretical considerations. In Chapters 4, 5 and 6 this approach will be numerically elaborated with a view to comparing the theoretically adopted fundamental points with the experimentally determined values and moreover with the results obtained with the ZEFE computer program [6]. With that program the behaviour of the floor bay has been analysed with the aid of the finite element method.

The object of this analysis is to find out under what conditions a highly simplified method of calculation is sufficiently reliable for predicting the behaviour of the floor bay.

Its behaviour depends to a high degree on the properties of the component materials. The properties of the materials and precast units employed can be determined in two ways:

a) from the results of tests,

b) from design rules given in the code of practice VB 74/84 [9].

In both cases the reports [1,6,7,8] and the data contained in Appendix I have been used. The material properties employed in the structural analysis are described in Appendix A.

Besides depending on the material properties, the distribution of forces depends also on the location of the section. In the calculation a distinction is therefore drawn between a section in the floor bay at a grid line and a section in the portion between two grid lines. Three methods of analysing the distribution of forces at the grid lines are investigated in sec. B1, B2 and B3 of Appendix B. These calculations relate to the following variants:

Variant A: the distribution of forces was calculated for the case where the section coincides with the transverse joint at a grid line, as indicated in
Fig. 3.1a. The material properties were determined experimentally.

- Variant B: this calculation was performed in the same way as for variant A, but the material properties were obtained from measured strength properties with the aid of the rules in VB 74/84.

- Variant C: the distribution of forces was calculated on the basis of the average material properties over a slab strip, as indicated in Fig. 3.1b. The width of the strip was taken as 120 mm, corresponding to the length of the longitudinal connections between the beams. The material properties were determined experimentally.

Fig. 3.1 Section through floor bay at a grid line.

The results of the calculations for the variants A, B and C are summarized in Chapter 4 and in Sec. B4 of Appendix B.

In the portion of the floor bay between grid lines the distribution of forces
was calculated in two ways. This distribution is investigated in Sec. C1 and C2 of Appendix C. These calculations relate to the following variants:

- Variant D: the distribution of forces was calculated for the case where the section coincides with a mortar joint, as indicated in Fig. 3.2.a. The material properties were determined experimentally.
- Variant E: the distribution of forces was calculated by considering a slab strip as shown in Fig. 3.2b. In this case the strip was taken as 240 mm wide, corresponding to the width of a floor unit. The material properties were determined experimentally.

![Diagram](image)

**Fig. 3.2 Section through floor bay between grid lines.**

The results of the calculations for the variants D and E are summarized in Chapter 4 and in Sec. C3 of Appendix C.
4. RESULTS OF A STRUCTURAL ANALYSIS BASED ON A SIMPLIFIED MODEL

Calculations based on a simplified schematization of the structure are presented in Appendices B to H. The fundamentals of this schematization are described in Sec. 2.1 to 2.5. The results of the calculations are summarized in this chapter and relate successively to the following behaviour patterns:

a. Longitudinal forces at grid line 7  
(b. Longitudinal forces between grid lines 7 and 8  
(c. Stiffness of the floor bay  
(d. Deflection of the longitudinal edge  
(e. Deformation of the ends  
(f. Crack width in transverse joints  
g. Shear stresses in the longitudinal joints  
h. Longitudinal forces at grid line 2  
i. Shear forces that can be resisted at grid line 2  
j. Ultimate load

4.1 Longitudinal forces at grid line 7

In Appendix B the longitudinal forces at grid line 7 have been calculated for three variants. In Sec. B7 of that appendix the results of the calculations are compared with the measured values obtained in the tests and with the values yielded by calculations performed with the ZEFE computer program. On the basis of that comparison it is explained there why, for analysing the measured data, only variant A will be used.

The principal features of variant A will now be summarized.

The distribution of the forces at grid line 7 in the cracked state is shown schematically in Fig. 4.1.
Fig. 4.1 Distribution of forces at grid line 7 for variant A.

The calculated values of the longitudinal forces on attainment of the yield point in the longitudinal (beam-to-beam) connections and for a total horizontal load $Q=6$kN are given in Table 4.1.

For this value of $Q$ the tensile forces acting in the longitudinal connections have been plotted for the variants A, B and C in Fig. 4.2.

<table>
<thead>
<tr>
<th></th>
<th>for $F_{\text{tot}}=0$kN</th>
<th></th>
<th>for $F_{\text{tot}}=3.04$kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$Q=73.2$kN</td>
<td>$Q=6$kN</td>
<td>$Q=70.7$kN</td>
</tr>
<tr>
<td>-------</td>
<td>(kN)</td>
<td>(kN)</td>
<td>(kN)</td>
</tr>
<tr>
<td>$N_{s1}$</td>
<td>26.00</td>
<td>2.13</td>
<td>26.00</td>
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<tr>
<td>$N_{s2}$</td>
<td>11.66</td>
<td>0.96</td>
<td>11.79</td>
</tr>
<tr>
<td>$N_{s3}$</td>
<td>5.92</td>
<td>0.49</td>
<td>6.10</td>
</tr>
<tr>
<td>$N_{c1}$</td>
<td>6.81</td>
<td>0.56</td>
<td>6.57</td>
</tr>
<tr>
<td>$N_{c2}$</td>
<td>36.75</td>
<td>3.01</td>
<td>34.28</td>
</tr>
</tbody>
</table>

Table 4.1 Overview of longitudinal forces at grid line for variant A.
4.2 Longitudinal forces between grid lines 7 and 8

In Appendix C the longitudinal forces between the grid lines 7 and 8 have been calculated for two variants. From Fig. C4 of that appendix it appears that the difference between the results of the variants D and E in so far as the curvature is concerned is very small. Variant D has been chosen for the purpose of further comparison of the measured and the calculated values.

The distribution of the forces at a section through a transverse joint between the grid lines 7 and C is shown in Fig. 4.3.

The calculated values of the longitudinal forces on attainment of the yield point in the longitudinal (beam-to-beam) connections at grid line 7 (for Q=73.2kN) and for a total horizontal load Q=6 kN are given in Table 4.2.
Fig. 4.3 Distribution of forces between grid lines 7 and 8 for variant D.

<table>
<thead>
<tr>
<th></th>
<th>Longitudinal force for</th>
<th>Variant D</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>$Q=73.2,\text{kN}$</td>
<td>$Q=6,\text{kN}$</td>
</tr>
<tr>
<td></td>
<td>(kN)</td>
<td>(kN)</td>
</tr>
<tr>
<td>$N_{c1}$</td>
<td>27.05</td>
<td>2.22</td>
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<tr>
<td>$N_{c2}$</td>
<td>6.21</td>
<td>0.51</td>
</tr>
<tr>
<td>$N_{c3}$</td>
<td>0.85</td>
<td>0.07</td>
</tr>
<tr>
<td>$N_{c4}$</td>
<td>3.03</td>
<td>0.25</td>
</tr>
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<td>$N_{c5}$</td>
<td>0.06</td>
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<tr>
<td>$N_{c6}$</td>
<td>6.59</td>
<td>0.54</td>
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<tr>
<td>$N_{c7}$</td>
<td>30.61</td>
<td>2.51</td>
</tr>
</tbody>
</table>

Table 4.2 Longitudinal forces between grid lines 7 and 8.

4.3 Stiffness of the floor bay

For calculating the horizontal deflection of the floor bay the schematization shown in Fig. 4.4 is adopted. As already described in Sec.2.2, the bay is
conceived as consisting of a number of flexurally rigid segments (of 600mm width) between the grid lines and of flexible segments (of 120mm width) at the grid lines.

Fig. 4.4 Schematization of the floor bay.

The curvature diagram is shown in Fig. 4.5. The curvatures have been calculated in Appendices B and C.

Fig. 4.5 Curvature diagram of the floor bay.

In Fig. 4.6, for the flexible and for the rigid segments, the moments have been plotted against the curvature, for the variants A to E, as calculated in Appendices B and C. The numerical values of the curvatures (\( \chi \)) and stiffnesses (\( EI \)) are assembled in Table 4.3.
For the purpose of the further comparison of the calculated and the measured values only the variants A and D will be considered, as explained in Sec. B7 of Appendix B and Sec. D4 of Appendix D.

Fig. 4.6 M-X diagram.

<table>
<thead>
<tr>
<th></th>
<th>yielding of longitudinal connection</th>
<th>yielding of beam reinforcement</th>
<th>cracking moment of longitudinal beam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>M (kNm)</td>
<td>92.19</td>
<td>98.89</td>
<td>105.97</td>
</tr>
<tr>
<td>Q (kN)</td>
<td>73.2</td>
<td>78.5</td>
<td>84.1</td>
</tr>
<tr>
<td>$\alpha$ (10^{-9} , l/mm)</td>
<td>2517</td>
<td>2303</td>
<td>2114</td>
</tr>
<tr>
<td>EI (kNm²)</td>
<td>36 600</td>
<td>42 900</td>
<td>50 100</td>
</tr>
</tbody>
</table>

Table 4.3 Curvatures and stiffnesses for variants A to E.
4.4 Deflection of the longitudinal edge

In Sec.D1 of Appendix D the horizontal deflection of the longitudinal edge at point D7 has been calculated for three variants. The results of these calculations are represented in a Q-δ diagram in fig. 4.7.

![Q-δ diagram at point D7](image)

Fig. 4.7 Q-δ diagram at point D7.

For the purpose of the further comparison of the calculated and the measured values only the variants A and D will be considered, as explained in Sec.B7 of Appendix B and Sec.D4 of Appendix D.
4.5 Deformation of the ends at grid lines 1 and 14

In Sec. G1 of Appendix G the deformations of the ends at the grid lines 1 and 14 have been calculated for a total horizontal load \( Q = 6 \text{kN} \) and \( F = 0 \text{kN} \). The results of this calculation are represented graphically in a deformation diagram in Fig. 4.8.

Fig. 4.8 Calculated deformation of grid line 1 for \( Q = 6 \text{kN} \).

The calculated displacements at the grid lines A to D and at grid line 1 or 14 for a total horizontal load \( Q = 6 \text{kN} \) are assembled in Table 4.4.

<table>
<thead>
<tr>
<th>grid line</th>
<th>( \Delta l ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>- 0.150</td>
</tr>
<tr>
<td>B</td>
<td>+ 0.010</td>
</tr>
<tr>
<td>C</td>
<td>+ 0.073</td>
</tr>
<tr>
<td>D</td>
<td>+ 0.233</td>
</tr>
</tbody>
</table>

Table 4.4 Calculated displacement of the longitudinal beams at grid line 1 or 14 for \( Q = 6 \text{kN} \).
4.6 Crack width in transverse joints

In Sec.H1 of Appendix H the crack widths in the transverse joints near grid line D have been calculated from the elongation of the longitudinal (beam-to-beam) connections for a total horizontal load of 6kN.

The results of this calculation have been plotted in Fig. 4.9.a for a floor bay without tensile force at the ends and in Fig. 4.9.b for a floor bay with a tensile force (F=3.04kN) at the ends.

Fig. 4.9 Crack width in transverse joints at grid line D for Q=6kN.

The calculated crack widths near grid line D for a total horizontal load Q=6kN are assembled in Table 4.5.
Table 4.5 Overview of crack widths in transverse joints at grid line D for Q=6kN.

4.7 Shear stresses in the longitudinal joints

The shear stresses and shear forces in the longitudinal joints at the grid lines A, B, C and D have been calculated in Appendix E. The magnitude of the calculated shear forces depends on the normal forces which can occur on each side of two successive sections. The magnitude of these normal forces depends on the variants which were adopted as the basis for the structural calculations.

In order to gain insight into the effect of the choice of variant upon the results of the shear stress calculations, two combinations of variants have been investigated in Appendix E:

a. the combination of the variants A and D;
b. the combination of the variants C and E.

The results of the calculations are given for the combination A + D in Fig. 4.10 and the for the combination C + E in Fig. 4.11. In these diagrams the abbreviation AAP (for example) denotes:
the longitudinal joint at grid line A on the side nearest grid line B.
Fig. 4.10 Overview of shear stresses in longitudinal joints for Q=6kN, variants A+D.
Fig. 4.11 Overview of shear stresses in longitudinal joints for $Q=6kN$, variants $C + E$. 
4.8 Longitudinal forces at grid line 2

In Sec. Fl and F2 of Appendix F the longitudinal forces have been calculated for the following cases:

a. without tensile force at the ends, for:
   a.1. limit state: failure due to shear at grid line 2;
   b. with tensile force (F=3.04kN) at the ends:
      b.1. limit state: zero stress at gridline A;
      b.2. limit state: failure due to shear at grid line 2.

The results of this calculation are assembled in Table 4.6.

<table>
<thead>
<tr>
<th></th>
<th>loading case</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a.l</td>
</tr>
<tr>
<td>total load (kN)</td>
<td>36.06</td>
</tr>
<tr>
<td>moment at grid line 2 (kNm)</td>
<td>12.98</td>
</tr>
<tr>
<td>longitudinal forces:</td>
<td></td>
</tr>
<tr>
<td>Ns1 (kN)</td>
<td>3.66</td>
</tr>
<tr>
<td>Ns2 (kN)</td>
<td>1.64</td>
</tr>
<tr>
<td>Ns3 (kN)</td>
<td>0.83</td>
</tr>
<tr>
<td>Nc (kN)</td>
<td>6.13</td>
</tr>
</tbody>
</table>

Table 4.6 Longitudinal forces at grid line 2.

In Figs. 4.12 and 4.13 the data in Table 4.6 have been plotted in two kinds of diagram. In Figs. 4.12.a and 4.13.a the tensile forces in the longitudinal (beam-to-beam) connections have been plotted against the bending moment at grid line 2 and the total horizontal load on the floor bay, respectively. In Figs. 4.12.b and 4.13.b the tensile forces have been plotted in a force diagram for a total horizontal load 6, 12, 18 and 24kN.

Fig. 4.12 relates to a floor bay without tensile forces acting at the ends, Fig. 4.13 to a floor bay with a total tensile force F=3.04 at the ends.
Fig. 4.12 Longitudinal forces at grid line 2 without tensile force at the ends.

Fig. 4.13 Longitudinal forces at grid line 2 with tensile force \((F=3.04\text{kN})\) at the ends.
In Fig. 4.14 the sum of the tensile forces at grid line 2 has been plotted against the bending moment at grid line 2 and against the total horizontal load on the bay, respectively – both for the case where no tensile forces are acting at the ends and for the case where a total tensile force $F=3.04$ is acting there.

Fig. 4.14 Some of tensile forces at grid line 2.

4.9 Shear forces that can be resisted at grid line 2

From the analysis presented in Sec. 2.5 it appears that the magnitude of the ultimate shear force in a transverse joint depends on the stress distribution which occurs in that joint.

Initially, in consequence of the bending moment, a stress distribution as shown in Fig. 2.11 will occur. If the maximum shear force that can be resisted in this situation is attained, then, depending on the roughness of the faces of the cracked joint, the so-called "shear friction" effect may occur, as a result of which a stress distribution as shown in Fig. 2.12 will develop.
In the above-mentioned cases it is presumed that the shear stress will be resisted solely by the compressive zone. But if predominantly tensile forces occur in a cracked joint, there will, up to a certain value of the bending moment, be no compressive zone. Yet the joint may then still be able to resist some shear force in consequence of the interlock effect at the joint.

In the Sec. F2 of Appendix F it has been calculated at what value of the bending moment there is still just no compressive zone in a floor bay in which a tensile force of 3.04 kN is acting at the ends. It emerges from this calculation that for this load \(Q=6.25\) kN an average shear stress \(\tau=0.0027\) N/mm\(^2\) has to be resisted by the cracked joint.

Owing to lack of sufficient information on the \(\tau-G-w-s\) relationship of a cracked joint, it is, at this stage of research, not yet possible to calculate the stress distribution on the basis of the "shear friction" model. Accordingly, in Appendix F the analysis for the shear forces that can be resisted has been confined to the mechanism shown in Fig. 2.11. There those forces at grid line 2 have been calculated for the following cases:

a. without tensile force at the ends of the floor bay;
b. with tensile force \(F=3.04\) kN at the ends.

The results of these calculations for the above-mentioned cases are assembled in Table 4.7.

<table>
<thead>
<tr>
<th>loading case</th>
<th>shear force that can be resisted (V_u) (kN)</th>
<th>ultimate load (Q_u) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>18.03</td>
<td>36.1</td>
</tr>
<tr>
<td>b</td>
<td>13.5</td>
<td>27.0</td>
</tr>
</tbody>
</table>

Table 4.7 Overview of shear forces that can be resisted and ultimate load.
4.10 Ultimate load

By the calculated ultimate load is understood the load at which, according to the calculations, failure occurs in consequence of either of the following phenomena:

a. failure in bending due to yielding of the longitudinal connection;
b. failure in shear at grid line 2.

For both these cases the calculation has been performed for the floor bay without tensile forces at the ends (F=0) and with tensile forces at the ends (F=3.04kN).

The calculated values of the ultimate load and the corresponding failure modes are given in Table 4.8.

<table>
<thead>
<tr>
<th>failure mode</th>
<th>ultimate load for F=0 (kN)</th>
<th>ultimate load for F=3.04kN (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>yielding of the longitudinal connection</td>
<td>73.2</td>
<td>70.7</td>
</tr>
<tr>
<td>shear at grid line 2</td>
<td>36.1</td>
<td>27.0</td>
</tr>
</tbody>
</table>

Table 4.8 Overview of calculated ultimate loads.

In Fig. 4.15 the calculated ultimate loads have been plotted in a diagram in which the values on the horizontal axis represent the tensile force at D7, while those on the vertical axis represent the bending moment at grid line 7 and the total horizontal load, respectively. This diagram relates to the values calculated for variant A.
Fig. 4.15 Relationship between calculated tensile force at D7 and bending moment at grid line 7.
5. COMPARISON OF THE CALCULATED RESULTS WITH THE TEST RESULTS OF SERIES C

The results of an analysis, as calculated in Appendices B to H, have been assembled in Chapter 4. This analysis was successively concerned with:

- a. Longitudinal forces at grid line 7
- b. Longitudinal forces between grid lines 7 and 8
- c. Stiffness of the floor bay
- d. Deflection of the longitudinal edge
- e. Deformation of the ends
- f. Crack width in transverse joints
- g. Shear stresses in the longitudinal joints
- h. Longitudinal forces at grid line 2
- i. Shear force that can be resisted at grid line 2
- j. Ultimate load

The results of the calculations cannot for all the above-mentioned cases be directly compared with the results of the measurements. A direct comparison is possible only for the cases: a, d, e, f, h, i and j.

The cases b, c and g can be judged only in an indirect way. In connection with this the reliability of the schematization of the structure plays an important part. This aspect will be further considered in Sec. 6.5.

The effect of the boundary conditions at the ends of the bay upon the behaviour of the bay is not considered in the present chapter. By "boundary conditions" is here understood the manner in which the ends of the floor bay are connected to the simulated shear walls along the short sides thereof. In Appendix K the effect of these conditions is represented in four diagrams for the following behaviour patterns:

- a. deflection of the longitudinal edge;
- b. tensile force in the connection between the beams at point D7;
- c. tensile force in the connection between the beams at point D2;
- d. average deformation of the ends of the bay at grid lines 1 and 14.
In these diagrams the measured values have been plotted for a total horizontal load $Q=6kN$. It appears from these diagrams that, for that load, the effect of the boundary conditions on the behaviour of the floor bay is negligible. Whether this assertion is valid also for higher values of the load is not certain, because the necessary data for comparison are lacking. It cannot be ruled out that with increasing magnitude of the load the effect of the boundary conditions will likewise increase.

In this chapter the calculated and the measured values for the following behaviour patterns will be compared with one another:

a. longitudinal forces in the transverse joints 

b. deflection of the longitudinal edge 

c. deformation of the ends of the bay at grid lines 1 and 14 

d. deformation of the transverse joints 

e. ultimate load 

5.1 Longitudinal forces in the transverse joints

5.1.1 Longitudinal forces at grid line 7

Fig. 5.1 shows the relationship between the tensile force in the longitudinal connection at point D7 and the bending moment at grid line 7. The corresponding total horizontal load is also indicated on the vertical axis of the diagram.

In this diagram the values calculated with the aid of the ZEFE computer program [6] and those obtained by "manual" calculation, as assembled in Table 4.1, are compared with the measured values obtained in the tests C401 and C206. Two lines have been plotted for the "manual" calculation, namely, one for the case where there is no tensile force and one for the case where a tensile force $F=3.04kN$ is acting at the ends. Two failure criteria are indicated on both these lines:

a. if failure due to shear at grid line 2 is the governing condition; 

b. if failure is due to the yield point being attained in the longitudinal connection at D7.
The results for test C401 relate to a floor without a preformed crack at grid line 2 and without tensile force at the ends. In test C206 the floor has a preformed crack at grid line 2 and there is a tensile force $F=3.04\,\text{kN}$ acting at the ends. The ZEFE computer program relates to a floor without tensile force at the ends.

![Graph](image)

Fig. 5.1 Relationship between tensile forces at D7 and bending moment at grid line 7.

Fig. 5.2 shows the longitudinal forces at grid line 7 for a horizontal load $Q=6\,\text{kN}$. In this diagram the ZEFE calculated values as well as the "manually" calculated values are compared with the measured values obtained in tests C401 and C206.

Fig. 5.2.a relates to the floor bay without tensile force at the ends, while Fig. 5.2.b represents the case where a tensile force $F=3.04\,\text{kN}$ is acting at the ends.
In Fig. 5.3 the longitudinal forces at grid line 7 are represented for a total horizontal load of 12, 18, 24 and 30kN, respectively. In each diagram the "manually" calculated values are compared with the corresponding measured values obtained in test C206. The measured and the calculated values are assembled in Table 5.1.

<table>
<thead>
<tr>
<th>load Q (kN)</th>
<th>moment grid line</th>
<th>manual calculation</th>
<th>measured (C206)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$N_{s1}$ (kN)</td>
<td>$N_{s1}$ (kN)</td>
</tr>
<tr>
<td>6</td>
<td>7.56</td>
<td>3.02</td>
<td>2.40</td>
</tr>
<tr>
<td>12</td>
<td>15.12</td>
<td>5.15</td>
<td>4.28</td>
</tr>
<tr>
<td>18</td>
<td>22.68</td>
<td>7.28</td>
<td>6.29</td>
</tr>
<tr>
<td>24</td>
<td>30.24</td>
<td>9.41</td>
<td>7.86</td>
</tr>
<tr>
<td>30</td>
<td>37.80</td>
<td>11.54</td>
<td>8.52</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$N_{s2}$ (kN)</td>
<td>$N_{s2}$ (kN)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.50</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.45</td>
<td>2.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.41</td>
<td>3.08</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.36</td>
<td>4.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.32</td>
<td>6.86</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$N_{s3}$ (kN)</td>
<td>$N_{s3}$ (kN)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.89</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.38</td>
<td>1.08</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.86</td>
<td>1.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.34</td>
<td>2.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.82</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.1 Overview of calculated and measured values of longitudinal forces at grid line 7 in test C206.

Fig. 5.2 Longitudinal forces at grid line 7 for a total horizontal load $Q=6kN$. 
Fig. 5.3 Longitudinal forces at grid line 7 for a total horizontal load of 12, 18, 24 and 30kN in test C206.
Fig. 5.4 Relationship between tensile forces and moments at grid line 7 in test C206.
In Fig. 5.4 the longitudinal forces at the points B7, C7 and D7 have been plotted against the bending moment at grid line 7. In these diagrams the "manually" calculated values are compared with the corresponding measured values obtained from test C206.

5.1.2 Longitudinal forces at grid line 5

In Fig. 5.5 the longitudinal forces at grid line 5 are represented for a total horizontal load of 12, 18, 24 and 30kN, respectively. In each diagram the "manually" calculated values are compared with the corresponding measured values obtained in test C206. The measured and the calculated values are assembled in Table 5.2.

In Fig. 5.6 the longitudinal forces at the points B5, C5 and D5 have been plotted against the bending moment at grid line 5. In these diagrams the "manually" calculated values are compared with the corresponding measured values obtained in test C206.

<table>
<thead>
<tr>
<th>load Q (kN)</th>
<th>moment grid line (kNm)</th>
<th>manual calculation</th>
<th>measured (C206)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( N_{s1} ) (kN)</td>
<td>( N_{s2} ) (kN)</td>
</tr>
<tr>
<td>6</td>
<td>6.48</td>
<td>2.72</td>
<td>1.37</td>
</tr>
<tr>
<td>12</td>
<td>12.96</td>
<td>4.54</td>
<td>2.18</td>
</tr>
<tr>
<td>18</td>
<td>19.44</td>
<td>6.37</td>
<td>3.01</td>
</tr>
<tr>
<td>24</td>
<td>25.92</td>
<td>8.20</td>
<td>3.82</td>
</tr>
<tr>
<td>30</td>
<td>32.40</td>
<td>10.02</td>
<td>4.63</td>
</tr>
</tbody>
</table>

Table 5.2 Overview of calculated and measured values of longitudinal forces at grid line 5 in test C206.
Fig. 5.5 Longitudinal forces at grid line 5 for a total horizontal load of 12, 18, 24 and 30kN in test C206.
Fig. 5.6 Relationship between tensile forces and moments at grid line 5 in test C206.
5.1.3 Longitudinal forces at grid lines 2 and 13

In Fig. 5.7 the longitudinal forces at the grid lines 2 and 13 are represented for a total horizontal load of 12, 18, 24 and 30kN, respectively. In each diagram the "manually" calculated values are compared with the corresponding values obtained in test C206. The calculated and the measured values are assembled in Table 5.3.

At the start of the test there was, at grid line 2, a 0.3mm wide crack extending along the full length of the joint, whereas the joint at grid line 13 was still uncracked at the start of testing. The first flexural crack at grid line 13 was formed at a total horizontal load Q=13kN.

<table>
<thead>
<tr>
<th>load (kN)</th>
<th>6</th>
<th>12</th>
<th>18</th>
<th>24</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q grid line 2 or 13 (kNm)</td>
<td>2.16</td>
<td>4.32</td>
<td>6.48</td>
<td>8.64</td>
<td>10.80</td>
</tr>
<tr>
<td>manual calculation N2</td>
<td>1.49</td>
<td>2.11</td>
<td>2.72</td>
<td>3.33</td>
<td>3.94</td>
</tr>
<tr>
<td>manual calculation N3</td>
<td>0.88</td>
<td>1.09</td>
<td>1.37</td>
<td>1.64</td>
<td>1.91</td>
</tr>
<tr>
<td>measured in grid line 2</td>
<td>1.17</td>
<td>1.89</td>
<td>2.89</td>
<td>4.32</td>
<td>6.22</td>
</tr>
<tr>
<td>measured in grid line 13</td>
<td>0.75</td>
<td>1.01</td>
<td>1.46</td>
<td>1.97</td>
<td>2.02</td>
</tr>
<tr>
<td>measured in grid line 13</td>
<td>0.91</td>
<td>1.07</td>
<td>1.88</td>
<td>2.55</td>
<td>2.52</td>
</tr>
<tr>
<td>measured in grid line 13</td>
<td>0.01</td>
<td>0.06</td>
<td>1.37</td>
<td>2.46</td>
<td>4.12</td>
</tr>
</tbody>
</table>

Table 5.3 Overview of calculated and measured values of longitudinal forces at grid lines 2 and 13 in test C206.

In Fig. 5.8 the longitudinal forces at the points B2, B13, C2, C13, D2 and D13 have been plotted against the bending moments at grid line 2 or 13. In these diagrams the "manually" calculated values are compared with the corresponding measured values obtained in test C206.

During the test (directly after the application of the tensile forces at the ends of the floor bay, as seems probable) the longitudinal connection at point A2 became detached. This could account for the different behaviour displayed by the longitudinal forces at B2 (see Fig. 5.8.a).
Fig. 5.7 Longitudinal forces at grid lines 2 and 13 for a total horizontal load of 12, 18, 24 and 30kN in test C206.
Fig. 5.8 Relationship between tensile forces and moments at grid lines 2 and 13 in test C206.
Fig. 5.9 Relationship between tensile forces and moments at grid line 13 in test C306.
In Fig. 5.9 the longitudinal forces at grid line 13 have, for test C306, similarly been plotted against the bending moments at the grid lines A, B, C and D. In these diagrams the calculated values are compared with the measured values. Test C306 relates to the bay subjected to repeated horizontal loading and a tensile force \( F = 3.04 \text{kN} \) at the ends. The transverse joint at grid line 13 had, before testing commenced, been provided with a preformed crack about 0.2mm in width.

Table 5.4 presents an overview of the calculated and the measured forces.

<table>
<thead>
<tr>
<th>reaction grid line 14</th>
<th>moment grid line 13</th>
<th>manual calculation</th>
<th>measured in grid line 13 (C306)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(kN)</td>
<td>(kNm)</td>
<td>( N_1 )</td>
</tr>
<tr>
<td>3</td>
<td>2.16</td>
<td>1.49 0.88 0.64</td>
<td>1.09</td>
</tr>
<tr>
<td>6</td>
<td>4.32</td>
<td>2.11 1.09 0.69</td>
<td>1.94</td>
</tr>
<tr>
<td>9</td>
<td>6.48</td>
<td>2.72 1.37 0.82</td>
<td>2.83</td>
</tr>
</tbody>
</table>

Table 5.4 Overview of calculated and measured values of longitudinal forces at grid line 13 in test C306.

5.1.4 Internal and external moments at the transverse joints

The relationship between the horizontal and the internal and the external bending moment, respectively, at some transverse joints is shown in two diagrams in Fig. 5.10. Fig. 5.10.a relates to the transverse joints at the grid lines 2 and 13 in test C206, while Fig. 5.10.b relates to those at the grid lines 5 and 7 in that same test.

In Fig. 5.11 the internal and the external moments at the grid lines 2 and 13 in the tests C206 and C306 are compared with one another. In test C206 the floor bay was loaded with a monotonically increasing load; in test C306 a repeating load was applied.
The external moment has been calculated from the sum of the reactions at the grid lines 1 and 14. Expressed as a formula, the external moment is:

for grid lines 2 and 13:

\[ M_{\text{ext}} = \frac{R_1+R_{14}}{2} \times 0.72 = 0.36 \ (R_1+R_{14}) \]

for grid lines 5:

\[ M_{\text{ext}} = 1.08 \ (R_1+R_{14}) \]

for grid line 7:

\[ M_{\text{ext}} = 1.26 \ (R_1+R_{14}) \]

The internal moment at the transverse joints cannot be accurately determined with the aid of the measured results because the location of the resultant compressive force cannot be ascertained by measurement. In order nevertheless to get some idea of the magnitude of the internal moment, the resultant compressive force is assumed to coincide with grid line A, in which case the highest possible value of this moment is obtained. The internal moment is then expressed by:

\[ M_{\text{int}} = N_{s1} \times 2.88 + N_{s2} \times 1.68 + N_{s3} \times 1.20 - F \times 1.44 \]

Table 5.5 presents an overview of the internal and the external moments for a total horizontal load \( Q=6, 12, 18, 24 \) and \( 30 \) kN. The measured values in this table relate to test C206.
Table 5.5 Overview of internal and external moments at grid lines 2, 5, 7 and 13 in test C206.
Fig. 5.10 Relationship between internal and external moment at grid lines 2, 5, 7 and 13, test C206.
5.1.5 Longitudinal forces at grid line D

The tensile forces in the longitudinal beam-to-beam connections at grid line D have been plotted for horizontal loads of 6, 18 and 30kN, respectively, in Fig. 5.12. In this diagram the "manually" calculated values are compared with the measured values obtained in test C206. The calculated and the measured values are assembled in table 5.6.
Table 5.6 Overview of calculated and measured values of longitudinal forces at grid line D, test C206.

<table>
<thead>
<tr>
<th>grid line</th>
<th>manual calculation for load:</th>
<th>measured (test C206) for load:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q=6kN</td>
<td>Q=18kN</td>
</tr>
<tr>
<td>2</td>
<td>1.49</td>
<td>2.72</td>
</tr>
<tr>
<td>3</td>
<td>2.02</td>
<td>4.23</td>
</tr>
<tr>
<td>4</td>
<td>2.42</td>
<td>5.44</td>
</tr>
<tr>
<td>5</td>
<td>2.72</td>
<td>6.36</td>
</tr>
<tr>
<td>6</td>
<td>2.92</td>
<td>6.97</td>
</tr>
<tr>
<td>7</td>
<td>3.02</td>
<td>7.27</td>
</tr>
<tr>
<td>13</td>
<td>1.49</td>
<td>2.72</td>
</tr>
</tbody>
</table>

Fig. 5.12 Tensile forces in longitudinal connection at grid line D, test C206.
5.2 Deflection of the longitudinal edge

In two diagrams in Fig. 5.13 the relationship between the deflection at point D7 and the total horizontal displacement is represented. In these diagrams the calculated values are compared with the measured values obtained in the tests C401, C601 and C206. Fig. 5.13.a relates to the floor bay without tensile force at the ends, while the bay with a tensile force $F=3.04\text{kN}$ at the ends is envisaged in Fig. 5.13.b.

![Diagram](image)

Fig. 5.13 Relationship between total horizontal load and deflection at D7.

The calculation of the deflection of the longitudinal edge has been performed in Appendix D and is summarized in Sec.4.4.

The deflection at grid line D for a total horizontal load $Q=6\text{kN}$ is shown in Fig. 5.14. This diagram represents the relationship between the calculated and the measured deflection obtained in the tests C401 and C601. Both of these relate to the floor bay without tensile force at the ends. As distinct from test C401, in test C601 a crack was formed at grid line 2 before testing commenced.
Fig. 5.14 Deflection of grid line D for a load $Q=6\,\text{kN}$ in tests C401 and C601.

Fig. 5.15 shows the relationship between the calculated deflection of the longitudinal edge and the measured deflection obtained in test C206 for different values of the load $Q$, namely, 6, 12, 18, 24 and 30 kN. Test C206 relates to the floor bay in which a tensile force $F=3.04\,\text{kN}$ is acting at the ends and there is a preformed crack at grid line 2.

In judging the measured deflection curve it should be borne in mind that part of the deflection may be due to shear displacement at the transverse joints. In test C206 this displacement occurred only at grid line 2. Its effect upon the deflection curve is represented in Figs. 5.15.b–e by a corrected deflection curve, which therefore represents the deflection that would have occurred if there had been no shear displacement.
Fig. 5.15 Relationship between calculated and measured deflection at grid line D in test C206.
5.3 Deformation of the ends at grid lines 1 and 14

Fig. 5.16 shows the deformation at grid line 1 for a total horizontal load \(Q=6\text{kN}\), without tensile force at the ends of the floor bay. In this diagram the calculated values are compared with the measured values obtained in the tests C401, C106 and C601. In test C401 there was, at this value of the load, as yet no flexural crack at grid line 2. In the other two tests there was a preformed crack at grid line 2; this approximately 0.3mm wide crack had been formed over the full length of the transverse joint.

The calculation of the deformation at the ends of the bay has been carried out in Sec.G1 of Appendix G and summarized in Sec.4.5.
Fig. 5.16 Deformation of the end of the floor bay at grid line 1 for Q=6kN.

5.4 Deformation of the transverse joints

5.4.1 Crack width in transverse joints at grid line D

The crack width at grid line D in the transverse joints, for a total horizontal load of 6kN, is shown in two diagrams in Fig. 5.17. Fig. 5.17.a relates to the floor bay without tensile force acting at the ends. In this diagram the calculated values are compared with the measured values obtained in the tests C401 and C601. Fig. 5.17.b relates to the bay subjected to a tensile force (F=3.04kN) at the ends. Here the calculated values are compared with the measured values obtained in the tests C801 and C206. The crack width calculation is given in Sec.H1 and Sec.H2 of Appendix H.

The relatively large crack widths at grid line 2 for the tests C601, C801 and C206 are due to the fact a preformed crack existed at that grid line before testing commenced. At the load in question there was as yet no cracking at grid line 5.
5.4.2 Deformation of the transverse joint at grid line 5

In Fig. 5.19, for test C401, the shear deformations and crack width at grid line 5, between the grid lines C and D, has been plotted against the shear force acting at grid line 5. In Fig. 5.20 these data have, for test C206, been plotted for six measuring points located on grid line 5. In these diagrams the measured values of the crack widths are compared with the values as calculated in Sec.H.3.1 of Appendix H.

The location of the measuring points along a transverse joint is indicated in Fig. 5.18. These points in this diagram and in the further diagrams are coded as follows:
5-CD = a point on grid line 5, in the area between grid lines C and D, near grid line C;
5-DC = a point on grid line 5, in the area between grid lines C and D, near grid line D.

Fig. 5.17 Crack width in transverse joints at grid line D for Q=6kN.
Fig. 5.18 Location of measuring points in transverse joints.

Fig. 5.19 Relationship between shear force and shear deformation and between shear force and crack width at grid line 5, test C401.
Fig. 5.20 Relationship between shear force and shear deformation and between shear force and crack width at grid line 5, test C206.
Fig. 5.20 (continued)
5.4.3 Deformation of the transverse joints at grid lines 2 or 13

In Fig. 5.21, for test C401, the shear deformations and crack width at grid line 2, between the grid lines C and D, has been plotted against the shear force acting at grid line 2. In Fig. 5.22 these data have, for test C206, been plotted for six measuring points located on grid line 2, as indicated in Fig. 5.18. In these diagrams the measured values of the crack width are compared with the calculated values, the calculations for which are presented in Sec. H.2.1 and Sec. H.2.2 of Appendix H.

![Fig. 5.21 Relationship between shear force and shear deformation and between shear force and crack width, test C401.](image)
Fig. 5.22 Relationship between shear force and shear deformation and between shear force and crack width at grid line 2, test C206.
Fig. 5.22 (cont.)

Shear force (kN)

Shear force (kN)

Shear force (kN)

Shear force (kN)

Shear force (kN)

Shear force (kN)

Crack width (mm)

Crack width (mm)

Crack width (mm)
In Fig. 5.23, for test C306, the shear forces at grid line 13 have, in six diagrams, been plotted against the shear deformation at the measuring points along that grid line between the grid lines A to D.

The measured values for grid line 2 in test C206 and those for grid line 13 in test C306 are comparable with one another because in both cases there was a preformed crack before the start of testing and in both cases the floor bay was loaded with a tensile force of total magnitude $F=3.04\text{kN}$ at the ends. Furthermore, a monotonically increasing load was applied in test C206, a repeating load in test C306.

In fig. 5.24, for test C306, the shear force at grid line 13 has, in six diagrams, been plotted against the crack width at the measuring points along that grid line between the grid lines A to D. In these diagrams the measured crack widths are compared with the widths calculated in Sec. H.2.2 of Appendix H.

In Figs. 5.25 and 5.26 the interrelationship of the shear stress, the normal (compressive) stress, the shear deformation and the crack width is represented in a combination of four diagrams. Fig. 5.25 relates to the transverse joint at grid line 2 in test C206 (monotonically increasing load), while Fig. 5.26 relates to the transverse joint at grid line 13 in test C306 (repeating load).

The shear stress has been calculated from the measured external forces divided by the total cross-sectional area of the joint:

$$\tau = \frac{V}{40 \times 2940} \text{ (N/mm}^2\text{)}$$

The normal stress has been calculated from the sum of the measured longitudinal
forces at the transverse joint divided by the total cross-sectional area of the joint:

\[ \sigma_c = \frac{N_{s1} + N_{s2} + N_{s3} + N_{s4}}{40 \times 2940} \text{ (N/mm}^2\text{)} \]

By shear deformation is here understood the average of all the shear deformations measured along the transverse joint.

By crack width is here understood the average of all the normal deformations measured along the transverse joint.
Fig. 5.23 Relationship between shear force and shear deformation at grid line 13, test C306.
Fig. 5.23 (cont.)
Fig. 5.24 Relationship between shear force and crack width at grid line 13, test C306.
Fig. 5.25 Relationship between shear stress, normal stress, shear deformation and crack width at grid line 2, test C206.

Fig. 5.26 Relationship between shear stress, normal stress, shear deformation and crack width at grid line 13, test C306.
5.5 Ultimate load

In Fig. 5.27 the calculated and the measured values of the ultimate load have been plotted in a diagram in which the values on the horizontal axis represent the tensile force at D7, while those on the vertical axis represent the moment at grid line 7 and the total horizontal load, respectively. The measured value obtained in test C206 relates to the floor bay with a tensile force $F=3.04\text{kN}$ acting at the ends. In the diagram this value is compared with the values calculated for the cases with and without tensile force at the ends.

The calculated ultimate load relates to failure due to shear at grid line 2. The calculation has been carried out in Appendix F and is summarized in Sec. 4.9 and 4.10.

An overview of the calculated and the measured values of the ultimate load is given in Table 5.7.

![Fig. 5.27 Overview of calculated and measured values of the ultimate load.](image)
Table 5.7 Overview of ultimate loads.

In Fig. 5.28 the shear force $V_u$ at failure (ultimate shear force) has been plotted against the logarithm of the number of load cycles. It is not yet possible, on the basis of these data, to make a pronouncement as to the number of cycles at a particular loading level. The line connecting the values for C206 and C306 indicates a tentative relationship between the ultimate shear force and the number of cycles to failure.

<table>
<thead>
<tr>
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<th>shear force $V_u$ for:</th>
<th>manual calculation (kN)</th>
<th>measured for C206 (kN)</th>
<th>measured for C306 (kN)</th>
<th>number of cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>monotonically increasing load</td>
<td>13.5</td>
<td>13.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>repeating load</td>
<td>-</td>
<td>-</td>
<td>10.2</td>
<td>6200</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 5.28 Relationship between ultimate shear force $V_u$ and number of cycles.
6. ANALYSIS OF THE CALCULATED AND THE MEASURED RESULTS

The results of the analysis presented in Appendices B to H have been assembled in Chapter 4, and then, in Chapter 5, these data have been compared with the results obtained in the tests. The relevant values are presented in tables and diagrams. Now, in the present chapter, these data will be further analysed and some comments be given. More particularly, the following behaviour patterns will be considered here:

- Sec. 6.1: Longitudinal forces in the transverse joints at grid lines 7 and 5;
- Sec. 6.2: Longitudinal forces at grid line D;
- Sec. 6.3: Longitudinal forces in the transverse joints at grid lines 2 and 13;
- Sec. 6.4: Deformation behaviour in the transverse joints;
- Sec. 6.5: Shear forces in the longitudinal joints;
- Sec. 6.6: Deflection of the longitudinal edge;
- Sec. 6.7: Deformation of the ends of the floor bay;
- Sec. 6.8: Ultimate load.

Since all the behaviour patterns have been measured on one and the same floor bay, it is necessary to exercise some caution in claiming general validity for the conclusions arrived at. The results are, in the first instance, valid only for the floor that was investigated and give only qualitative indications as to the behaviour of a floor bay in general.

Among others, one reason why it is not possible to make generally-valid statements is that the behaviour pattern is dependent, on the one hand, upon the quality of the jointing mortar and the design features of the joint and, on the other hand, upon the ratio between bending moment and shear force in a joint. In the floor under investigation the largest moments always occur in combination with the smallest shear forces. From the series of tests performed on this structure it cannot be concluded whether different ratios of moments and shear forces might not yield different results.
In this chapter the calculated values will be compared with the measured values of some tests which are to be regarded as representative, without taking account of the boundary conditions at the ends of the floor bay. This latter omission is justified because it appears from the introductory part of Chapter 5 that the boundary conditions at the ends have only a minor effect on the behaviour of the floor.

6.1 Longitudinal forces in the transverse joints at grid lines 7 and 5

By longitudinal forces is here understood the tensile forces acting in the connection between two beams.

6.1.1 Longitudinal forces at grid line 7

In the calculation of the longitudinal forces in the floor with the aid of a simplified model and with the ZEFE computer program the materials have been assumed to be characterized by linear behaviour. This being so, in a floor bay without tensile force applied at its ends there must theoretically exist a linear relationship between the bending moment and the tensile forces at grid line 7. There must then theoretically also exist a linear relationship between the longitudinal forces themselves. Figs. 5.1. and 5.2. illustrate this.

From the relationship between the measured and the calculated values as represented in Figs. 5.2 and 5.3 it appears that the linear relation is reasonably satisfactory up to a horizontal load Q=18kN. At higher values of Q the tensile force at grid line D lags behind that occurring at grid lines B and C. This phenomenon is clearly manifested in Fig. 5.4.

As was to be expected, it is apparent from Fig. 6.1. that, despite the non-linear behaviour of the longitudinal forces, the internal equilibrium is preserved.

It cannot be clearly ascertained from the tests what caused the deviant behaviour of the longitudinal forces. It may be due to shear deformation and/or
cracking in the longitudinal joints between the beams and the end faces of the floor units (hollow core slabs).

In view of the calculated values of the shear stresses in the longitudinal joints (see Sec. 4.7) it can be assumed that locally large shear stresses occurred (see Figs. 4.10 and 4.11) at the instant of failure of the floor. From Fig. 6.1, where the observed cracking of the floor bay in test C206 is represented, it appears that a fairly long crack developed at grid line B, between the grid lines 5 and 12, as well as a short crack at grid line C between the grid lines 6 and 8. It cannot be ascertained at what value of the load these cracks were formed and whether there is any tie-up between these cracks and the non-linear behaviour of the longitudinal forces at grid line 7.

![Diagram](image)

Fig. 6.1 Cracking in floor bay, test C206.

Finally, it appears from Figs. 5.1 and 5.2 that there is good agreement between the values calculated with the aid of the simplified model and those calculated with the ZEFE program.

6.1.2 Longitudinal forces at grid line 5

From Figs. 5.5, 5.6 and 5.10.b it appears that what has been stated with regard to grid line 7 in the previous section of this chapter is also applicable to grid line 5. The only, and not very important, difference is that the joint at grid line 5 remained uncracked up to a load Q=13kN, so that tensile forces could develop only after this load had been reached.
6.2 Longitudinal forces at grid line D

It appears from Fig. 5.12 that the measured behaviour of the longitudinal forces at grid line D at certain loads deviates quite considerably from the calculated behaviour. The deviant behaviour in the case of a total horizontal load $Q=6kN$ can indeed be explained by the fact that not all the flexural cracks had fully developed at that load. For $Q=18kN$ there is in fact good agreement between measurement and calculation. This tallies with what has already been pointed out in Sec.6.1.1, namely, that up to $Q=18kN$ there is a reasonably good linear relationship between moments and longitudinal forces. With further increase of the load the measured behaviour deviates considerably from the calculated values. At $Q=30kN$ there is relatively little difference between the forces themselves. This phenomenon indicates that the floor behaves at first like a beam, but that after the load has reaches $Q=18kN$ the floor behaves more and more like an arch with tie-bar. This phenomenon is undoubtedly in part also due to the cracking that occurs in the longitudinal joints.

From Fig. 5.12 it also appears that for a load $Q=30kN$ the measured tensile forces at the points D4, D5, D6 and D7 lag behind the calculated values. From Figs. 5.3 and 5.5 it can be concluded that therefore the longitudinal forces at the grid lines B and C become larger than their calculated values. This phenomenon, too, is probably caused by cracking in the longitudinal joints.

From the foregoing examination of the behaviour of the longitudinal connections it can be concluded that it is advisable to adopt the longitudinal connection designed for the maximum moment also for the other longitudinal connections along the same longitudinal grid line of the floor bay. It is moreover advisable, in detailing the structure, to ensure that after cracking of the longitudinal joints the shear displacement will remain as small as possible. This can be achieved by providing suitable transverse reinforcement. In Sec. 6.5 it will be seen that the same conclusion can be drawn from the analysis of the shear forces in the longitudinal joints.
6.3 Longitudinal forces in the transverse joints at grid lines 2 and 13

It appears from Figs. 5.7 and 5.8 that the measured behaviour of the longitudinal forces at grid line 2 (when there is a preformed crack) differs considerably from the calculated values. A similarly deviant behaviour, though less pronounced, occurs also at grid line 13 with only a flexural crack.

From Fig. 5.8.c it appears that the behaviour of the longitudinal forces at the points D2 and D13 is identical. Evidently the initial crack width affects the magnitude of the longitudinal forces, but hardly affects their pattern of behaviour.

From examining Figs. 5.7 and 5.8.a it should be taken into consideration that the longitudinal connection at A2 had become detached at the commencement of test C206. This accounts for the deviant behaviour of the longitudinal force at B2.

From Fig. 5.8 it also appears that the longitudinal forces at the points B2, C2 and D2 after the attainment of a certain load are considerably larger than the calculated values. This notable difference in behaviour is also apparent from Fig. 5.10.a, where the external and the internal moments have been made comparable with one another in an M-Q diagram. At the joint with the preformed crack the internal moment is found to be in reasonably good agreement with the external moment up a load of about Q=12kN. With further load increase the internal moment undergoes a relatively large increase. Because the internal and the external moment must always be equal, this means that the starting point for the calculation of the internal moment cannot be correct. This implies in turn that the stress distribution and distribution of forces as shown in Fig. 6.2, on which the analysis of the structure is based, is no longer applicable.

What pattern of forces did in fact occur can be described only in approximate general terms. Presumably, the shear force will at first be resisted only by
the compressive zone, as shown in Fig. 6.2.

The magnitude of the shear force that can be resisted, while as yet no shear displacement occurs, depends on the \( \tau-\sigma \) relationship for the mortar joint. The \( \tau-\sigma \) diagram in Fig. 6.2 applies to an uncracked joint. No data are as yet available for a cracked joint.

![Fig. 6.2 Stress distribution in a transverse joint without shear displacement.](image)

When the shear force that can be resisted is exceeded, a shear displacement will occur. If then, simultaneously with this displacement, the crack becomes wider, extra tensile forces will develop in the longitudinal connections in consequence of wedge action. These extra tensile forces give rise to extra compressive forces in the joint, as a result of which the shear load capacity may further increase. This phenomenon is called "shear friction".

The stress distribution in the transverse joint will then take on a very different appearance and possibly acquire the shape shown schematically in Fig. 6.3. As appears from Fig. 5.8, this phenomenon does not arise at grid line 13 (with a flexural crack), because at grid line 2 the floor had failed in shear before its shear load capacity as envisaged in Fig. 6.2 was reached. The situation at grid line 2 was different. From Fig. 5.10.a it may be concluded that already at a load \( Q=12\text{kN} \) the limit value of Fig. 6.2 had been reached. The fact that nevertheless failure occurred only when \( Q \) had reached a value of 30kN must be attributed to the "shear friction" effect.
For the "shear friction" model to occur, the crack faces have to be sufficiently rough. Because the crack will usually occur between the joint filling material and the precast floor unit, the sides of the joint should possess a sufficient degree of natural roughness or otherwise should be suitably profiled. The required roughness will depend to a great extent on the width of the crack which may be formed by causes other than the bending moment (e.g., shrinkage, creep, restraint of deformation). In certain cases it may therefore be necessary to provide the lateral faces of the floor units with a profiling whose projections/indentations are of a depth corresponding to the anticipated crack width.

The above analysis relates to the floor subjected to monotonically increasing load. In Fig. 5.9 the effect of a repeating load is represented, as was applied in test C306. The transverse joint under consideration, at grid line 13, had been provided with a preformed crack about 0.2mm in width before testing commenced. Just as in C206, in this test the floor bay was loaded with a tensile force totalling F=3.04kN at its ends.

It appears from Fig. 5.9.a that with increasing magnitude of the load a tensile force develops in the longitudinal connection at point A13. However, in
consequence of the bending moment, a compressive force should occur at that point, since it is located in the compressive zone of the section. From Figs. 5.9.b and 5.9.c it likewise emerges that the increase in the measured longitudinal forces is considerably greater than the increase in the calculated forces. Only the measured longitudinal forces at point D13 are in reasonably good agreement with the calculated values.

From this behaviour, too, it is apparent that the schematization in Fig. 6.2 is not valid for calculation of the distribution of forces in the transverse joints. Under the monotonically increasing load applied in test C206 it was found that this schematization was indeed adequate up to a load Q=12kN. On the other hand, it was totally unsatisfactory under repeating load, and it can be presumed that soon after the first few loading cycles had been applied to the structure, the schematization shown in Fig. 6.3 took over instead.

This is also manifested in Fig. 5.11, where the internal and the external moment are compared with each other. It is notable that the internal moment associated with monotonically increasing load differs only little from that associated with repeating load up to a shear force of about 11 kN. After this there is a considerable difference. Under monotonically increasing load the longitudinal forces continue to increase with the load, though not linearly. Under repeating load the tensile forces increase greatly, even when the magnitude of the load is not increased.

The deformation behaviour in a transverse joint will be analysed in the next section of this chapter.

6.4 Deformation behaviour in the transverse joints

Since the floor bay failed in shear, it is of real importance to obtain information on the magnitude of the ultimate shear force and the distribution of this force along the transverse joint. It is not possible to determine the shear stresses by direct measurement. Therefore the shear deformations and the
crack width were measured at a number of points along a transverse joint. With the aid of the data given in Sec. 5.4 it will now be attempted to obtain some insight into the shearing mechanism of the transverse joints. For this purpose the behaviour of the joints at the grid lines 2 and 13 will be considered, both for monotonically increasing and for repeating load.

6.4.1 Deformation behaviour at grid lines 2 and 13

From the analysis given in Sec. 6.3 it has already become apparent that the calculated longitudinal forces at grid line 2, based on a stress distribution as shown in Fig. 6.2, are not in agreement with the measured values. This is more particularly the case at a joint with a preformed crack.

In connection with a stress distribution as in Fig. 6.2 it is assumed that the bending moment is the governing factor and that the shear deformation is negligible. The results of the measurements show, however, both with regard to grid line 2 in test C206 and grid line 13 in test C306, that this assumption is not in conformity with the actual behaviour. In order to be able to describe this behaviour it is necessary to schematize a cracked joint in such a way that a theoretical model can be derived from this schematization.

In the literature a number of models have been developed for describing the transfer of force across a crack, by Walraven [4] and Tassios [5], among others. Walraven's model is shown in Fig. 6.4, while that proposed by Tassios is shown in Fig. 6.5.

Both are based on cracking in monolithic concrete, not on cracking in a joint between two precast concrete floor units.

In order to make the measured data obtained in tests C206 and C306 comparable with the analyses of Walraven and Tassios, the values given in Sec. 5.4 have been rearranged in five interdependent diagrams presented in Figs. 6.6 and 6.7. They express the relationship between the average shear stress, the average normal stress, the average shear deformation, the average increase in crack width, and the quotient of shear stress and normal stress (\(\mu = \tau/\sigma\)). The average
shear stress has been calculated from the measured external forces divided by the total cross-sectional area of the transverse joint:

\[ \tau = \frac{V}{40 \times 2940} \ (N/mm^2) \]

The average normal stress has been calculated from the measured longitudinal forces in the transverse joint divided by the total cross-sectional area of that joint:

\[ \sigma_c = \frac{N_{s1} + N_{s2} + N_{s3} + N_{s4}}{40 \times 2940} \ (N/mm^2) \]

Fig. 6.4 Crack model according to Walraven [4].

Fig. 6.5 Crack model according to Tassios [5].
and crack width for monotonically increasing load, Test C206.

Fig. 6.6 Relationship between shear stress, normal stress, shear deformation, and crack width.

Test C206
**Fig. 6.7** Relationship between shear stress, normal stress, shear deformation, and crack width for repeating load, test C306.
By shear deformation is here understood the average of all the measured shear deformations along the transverse joint. By crack width is here understood the average of all the increases in crack width measured along the transverse joint.

Figs. 6.6.a and 6.7.a represent the relationship between the average shear stress and the average shear deformation for monotonically increasing load and for repeating load, respectively. The two diagrams are not directly comparable with each other because the shear deformations are plotted to different scales. The relationship between the average normal stress and the average shear deformation for monotonically increasing load and repeating load, respectively, is shown in Figs. 6.6.b and 6.7.b. The relationship between the average shear deformation and the average crack width is shown in Figs. 6.6.c and 6.7.c. In Figs. 6.6.d and 6.7.d the average normal stress has been plotted against the quotient \( \mu = \tau / \sigma \). Finally, in Figs. 6.6.e and 6.7.e the average normal stress has been plotted against the average crack width. The measured crack widths are in fact increases in crack width because, in both tests, there was a preformed crack - of 0.3mm and 0.2mm width respectively - before the tensile forces were applied to the end of the floor bay.

The following can be inferred from a comparison of the diagrams presented in Figs. 6.6 and 6.7 with regard to the shear behaviour of a transverse joint:

a. From Figs. 6.6.a and 6.7.a it appears that with increasing load the shear stiffness \( (k = \tau / s) \) decreases.

b. From Figs. 6.6.a and 6.7.a it appears that with repeating load the shear stiffness decreases considerably:
- Under monotonically increasing load there occurred brittle fracture at an average shear stress of 0.118 N/mm² and a shear deformation of 0.97mm; hence \( k = \tau / s = 0.118 / 0.97 = 0.122 \) N/mm².
- Under repeating load there occurred "plastic" deformation after 6200 load cycles of varying level. At that instant the average shear stress was 0.086 N/mm² and the average shear deformation was 5.50mm; hence \( k = \tau / s = 0.086 / 5.50 = 0.016 \) N/mm².
c. From Figs. 6.6.d and 6.7.d it appears that with increasing load the curve for $\tau/G$ approaches a limit:
- At the instant of failure this quotient $\mu = \tau/G$ under monotonically increasing load was equal to 1.3. This value can be regarded as the coefficient of friction.
- Under repeating load the value of this quotient at the instant of termination of the test $\mu$ was 0.9.
- At the instant when "yielding" of the joint occurred, the value attained by $\mu$ was 1.6;

d. From Figs. 6.6.c and 6.7.c it appears that both with monotonically increasing load and with repeating load the relationship between crack width and shear deformation is reasonably linear;
e. It also appears from these diagrams that there is a considerable difference in the ratio of crack width to shear deformation between these two types of loading:
- Under monotonically increasing load the ratio between crack width increase and shear deformation is $\Delta w \approx 0.17s$.
- Under repeating load the ratio between crack width increase and shear deformation is: $\Delta w \approx 0.04s$.

Besides a considerable difference in the ratio between $\Delta w$ and $s$, under repeating load the shear deformation is found to be many times greater than under monotonically increasing load.

The data yielded by the tests are not sufficient to justify making a choice between the crack models of Walraven and Tassios, respectively, or developing an alternative model. For that purpose the requisite data should be available from the research carried out by CUR-VB Committee C43 "Joints in precast concrete floors" [2].

6.5 Shear forces in the longitudinal joints

The calculated shear stresses in the longitudinal joints between the beams and the ends of the floor units at the grid lines A, B, C and D are summarized in
Sec. 4.7. The calculation has been carried out with two combinations of variants,

namely:

a. combination of the variants A and D;
b. combination of the variants C and E.

As it was not possible in the tests to measure the shear forces in the longitudinal joints, no direct conclusions as to the reliability of the schematization of the structure can be drawn from the calculated results. From an examination of the cracking pattern of the floor it is only possible to deduce whether particular calculated shear stresses may have occurred.

On comparing Figs. 4.10 and 4.11 with each other it appears that the schematization of the structure is of major influence on the calculated shear stress behaviour in the longitudinal joints:

- from the calculation with the aid of the variants A and D it emerges that the largest shear stresses occur near grid line A at the centre of the floor bay;
- from the calculation with the aid of the variants C and E it emerges that the largest shear stresses occur near the grid lines C and D and are fairly uniformly distributed over the full length of the floor.

Fig. 6.8 Shear deformation in the longitudinal joints as calculated with the ZEFE computer program [6].
Both results differ considerably from the stress distribution occurring in a beam consisting of a homogeneous material, for in such a beam the maximum shear stress occurs in the middle of the cross-section at the position where the maximum shear force occurs. In a floor structure composed of precast concrete units the maximum shear stresses occur directly beside the longitudinal (beam-to-beam) connections. This behaviour pattern also emerges from the results of the calculations performed with the ZEFE computer program, as appears from Fig. 6.8. The fact that in that case the highest values of the shear stresses occur near grid line A must be attributed to the fact that in the mathematical model the joint between the beams was unable to transmit compressive forces.

In the calculation with variants A + D the largest shear stress for a load $Q=6\text{kN}$: $\tau = 0.215 \text{ N/mm}^2$.

This means that at the ultimate load $Q_u = 30.3\text{kN}$ the shear stress is:

$$\tau = \frac{30.3}{6} \times 0.215 = 1.09 \text{ N/mm}^2$$

In the calculation with variants C + E the largest shear stress for a load $Q=6\text{kN}$ is: $\tau = 0.063 \text{ N/mm}^2$

This means that at the ultimate load $Q_u = 30.3\text{kN}$ the shear stress is:

$$\tau = \frac{30.3}{6} \times 0.063 = 0.318 \text{ N/mm}^2$$

The results of both calculations are high in comparison with the shear stress that can be resisted at grid line 2. The average shear stress at failure at this grid line is:

$$\tau_u = \frac{Q/2 - Q/24}{40 \times 2940} = 0.118 \text{ N/mm}^2$$

This value is in good agreement with the splitting tensile strength of the jointing mortar, which according to Table 2.5 of Stevin Report 5-84-4 [1] is: $f_{sp} = 0.1 \text{ N/mm}^2$. 

The cracking pattern in Fig. 6.1 shows furthermore that in the middle of the floor bay cracking in the longitudinal joints occurred only at the grid lines B and C, not at the grid lines A and D, as might have been expected on the basis of the structural analysis. Quite probably, in consequence of the shear deformation in the longitudinal joints, there occurred a redistribution of forces so that the stresses remained below the permissible values.

In analysing the longitudinal forces as well as the deflection of the longitudinal edge it appeared justified to conclude that the combination of variants A+D gives calculated results which agree most closely with the measured values. This was found not to be the case in calculating the shear stresses in the longitudinal joints, however. For that purpose the combinations C+E gives a more acceptable result. From the fact that the calculation based on the combination of variants A+D agrees best with the measured deflection values of the longitudinal edge it can be concluded that this combination can also most suitably be adopted for calculating the stiffness of the floor and for the longitudinal forces between the grid lines 7 and 8.

From the above considerations it can be concluded that mathematical models for calculating the longitudinal shear stresses must be used with due caution. Whatever mathematical model is adopted, it is evident that considerable shear stresses are liable to occur in the longitudinal joints and that there is a high risk of cracking.

In detailing the structure it is therefore necessary to ensure that cracking in the longitudinal joints will not lead to their immediate failure. This can be achieved by providing suitable transverse reinforcement so that extra shear force can be transmitted in consequence of the "shear friction" effect as described for grid line 2 in Sec. 6.3. This connector reinforcement is also needed to enable tensile forces due to the loading (e.g., wind suction) to be transmitted to the floor.
6.6 Deflection of the longitudinal edge

The relationship between the calculated and the measured value of the deflection of the longitudinal edge of the floor bay in the tests C401, C601 and C206 has been described in Sec. 5.2. As shown in Figs. 5.13 and 5.14, for a total horizontal load \( Q = 6 \text{kN} \) the deflection found in test C601 is in good agreement with the calculated value, but that the deflection found in test C401 lags behind. An important reason for the difference in deflection in test C401 must be sought in the fact that at this load the joints at several grid lines were still uncracked.

It appears from Fig. 5.15 that with increasing magnitude of the load the deviation with regard to the calculated values increases considerably (test C206). This is in part due to the shear displacement that occurs in the preformed crack at grid line 2, as indicated in this diagram. The difference between measured and calculated values is not thereby fully explained. Hence the starting point adopted for the calculation, in which only the flexural deformation is taken into account, is not entirely in conformity with the actual behaviour.

The actual deflection mechanism can be explained as follows:

The floor bay can be conceived as divided into 13 segments separated by the grid lines 1 to 14. Each of these segments can be regarded as infinitely rigid, while the deformations and displacements are considered to be concentrated largely in the transverse joints at these grid lines. This assumption is reasonable because it appears from the summary in Sec. 4.3 that the stiffness of the segment between two grid lines is about 40 times as high as the stiffness at each such grid line itself. Two kinds of deformation at a transverse joint are to be distinguished:

a. deformations perpendicular to the joint in consequence of bending moments and normal forces;
b. deformations parallel to the joint due to shear forces.

The deflection shown schematically in Fig. 6.9 is due to the deformations perpendicular to the joints. The theoretical deflection curves for the floor bay
have been determined on the basis of this conception in Appendix D. An extra deflection as schematized in Fig. 6.10 is caused by the deformations parallel to the joints.

Fig. 6.9 Deflection curve due to moments and normal forces.

Fig. 6.10 Deflection curve due to shear deformations.

From the description of the shear behaviour of a transverse joint (see Sec. 5.4.2 and 6.3) it appears that more particularly in a joint with a preformed crack relatively large shear deformations may occur. In the case of a crack due to bending moments the shear deformation is considerably less, but not entirely negligible, as appears from Fig. 5.20 for grid line 5 in test C206.

The results presented in figs. 5.3 and 5.4 are also of importance in judging the deflection curves. The fact that there is no linear relationship between the load and the tensile forces in the longitudinal connections means that the stiffness of the floor bay has a non-linear distribution, as has been assumed in the structural analysis. Although it is difficult to quantify it numerically, it can be presumed to have an unfavourable effect on deflection.
Finally, the results present in Fig. 5.12 should be taken into consideration in this analysis. From that diagram it appears that with increasing load the floor behaves more and more in the manner of a tied arch. This means that the overall extension of the tie-rod will be greater than was supposed on the basis of the calculation. In consequence of this the deflection will also be greater than the calculation indicates.

So it must be concluded that the calculated deflection found by taking account only of the effect of the bending moments will be less than the actual deflection, the reason being that the shear deformations are not negligible. This is more particularly relevant in cases where cracks occur in consequence of tensile forces and/or deformation restraint. With the ZEFE computer program it is indeed possible in principle to take shear deformation into account. Appropriate input data are not yet available, however.

6.7 Deformation of the ends of the floor bay

The relationship between the calculated and the measured deflection of the ends of the floor bay at the grid lines 1 and 14 is given in Sec. 5.3. From Fig. 5.16 it appears that there is reasonably good agreement between the measured and the calculated values for the tests C401, C601 and C106. The basis adopted was that there exists a linear relationship between the displacements at the grid lines A, B, C and D. As the diagram shows, this is indeed the case for test C601, but not for C401 and C106. In these last-mentioned tests the displacements at the grid lines B and C are almost equal, so that the deformation of grid line 1 is no longer linear, but has become S-shaped. Such a shape indicates that the shear force has an important effect on the deformation behaviour of the ends of the bay. This may have been the cause of cracking in the longitudinal joints between the grid lines 1 and 2, and the grid lines 13 and 14, as appears from Fig. 6.1.

6.8 Ultimate load

The relationship between the measured and the calculated ultimate load is
represented in Sec. 5.5 and in Fig. 5.27. At first sight this diagram suggests that there is reasonably good agreement. All the same, this result must be viewed with due caution.

The calculation for the limit state of failure due to shear at grid line 2 has been performed in Appendix F, based on a distribution of forces at that grid line as shown in Fig. 6.2. The shear force that can be resisted at grid line 2 has been calculated from the stress distribution in the compressive zone and the $\tau-\sigma$ diagram according to Fig. A4 in Appendix A.

From the analysis given in Sec. 6.3, however, it emerges that the stress distribution as shown in Fig. 6.2 is not in accordance with the actual stress distribution in the transverse joint. More particularly at a joint with a preformed crack the shear deformation in the joint is found to be of considerable influence on the stress distribution in that joint and therefore also on the magnitude of the load that can be resisted. From this analysis it appears that the $\tau-\sigma$ relationship alone cannot adequately describe the actual shear behaviour; for that, a $\tau-\sigma$-$s$-$w$ relationship for the joint must be known.

Since the tests C206 and C306 both had a preformed crack (at grid line 2 and grid 13, respectively), the structural analysis is in fact not valid for those tests. It must accordingly be concluded that the similarity was due purely to chance.

In the present stage of research it is not yet possible to predict the ultimate load (failure load) on the basis of a calculation. Although it is possible, basing oneself on the tests C206 and C306, to establish a $\tau-\sigma$-$s$-$w$ relationship (see Figs. 5.25 and 5.26), these diagrams are relevant only to the floor bay investigated and offer no scope for developing a generally-valid formula. For this it will be necessary to carry out further research.

From Fig. 5.28 it is apparent that the number of cycles of repeating load is of influence on the magnitude of the ultimate load. Under such conditions a reduction in the ultimate load is found to occur if failure is due to shear in a transverse joint. Whether this assertion is valid also for other modes of failure cannot be deduced from the tests. The number of tests which resulted in failure is moreover too small to enable the influence of the number of load cycles to be quantified.
7. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions can be drawn from the analysis presented in Chapter 6:

1. The manner in which the floor bay is connected to shear walls at its ends is of little influence on the magnitude of the longitudinal forces in the longitudinal (beam-to-beam) connections, the horizontal deflection of the longitudinal edge and the deformation of the ends of the bay.

2. The measured tensile forces acting in the longitudinal connections in a transverse joint with predominantly bending moment are in reasonably good agreement with the values calculated from a structural analysis. By this is understood a calculation based on a stress distribution due to bending moments and normal forces (if any), neglecting the shear behaviour. The above-mentioned agreement between measured and calculated values exists up to a load $Q=18kN$ (approx.). With further increase of the load the measured behaviour of the longitudinal forces at grid line D deviates more and more from the calculated values. At a load $Q=30kN$ there is relatively little difference between the tensile forces at grid line D. Apparently the floor behaves increasingly like a tied arch from $Q=18kN$ onwards. Hence it is recommended that the longitudinal connection as designed for the maximum moment should be adopted also for the other longitudinal connections located on the same longitudinal grid line.

3. In joints with predominantly shear force the actual longitudinal forces may differ considerably from those which can be expected from a structural analysis for bending. This difference is due to the fact that the stress distribution in a (transverse) joint which is loaded only in bending is very different from that in a joint loaded predominantly in shear where the displacement parallel
to the joint plays an important part. This is more particularly true of joints which have cracked in consequence of shrinkage, creep, deformation restraint, etc.

4. Repeating load results in a considerable increase in shear deformation in comparison with a joint subjected to monotonically increasing load.

5. The stress distribution in a joint in which shear displacement can occur depends on the relationship between the shear stress $\tau$, the normal stress $\sigma$, the shear deformation $s$ and the crack width $w$. This relationship is represented schematically in Fig. 7.1.

![Diagram](image)

**Fig. 7.1** Schematic representation of $\tau$-$\sigma$-$s$-$w$ relationship in a cracked joint.

For lack of characteristic data it was not possible to predict the stress distribution, and therefore also not the longitudinal forces, on the basis of a structural analysis. Further research should supply these data for monotonically increasing load, repeating load and alternating load.

6. The load capacity of a transverse joint with no, or only a small, bending moment is determined by the "shear friction" effect and is thus to a great extent dependent on the roughness of the faces of the joint. It is recommended that the natural (as-cast) roughness of the floor units be increased by
suitably profiling their lateral faces. The depth of the profiling should correspond to the anticipated width of the crack due to shrinkage, creep, imposed deformation, etc.

7. The longitudinal joints should likewise be so detailed that efficient transfer of shear forces is achieved. To ensure this, connector reinforcement perpendicular to the longitudinal joints must be installed. This reinforcement is also needed for the transmission of tensile forces arising from loading (e.g., wind suction) to the floor.

The theoretical shear stresses in the longitudinal joints are highly dependent on the mathematical model adopted for a structural analysis based on a simplified model of the structure. The results of a "manual" calculation should therefore be accepted with due caution.

8. The horizontal deflection of the longitudinal edge of the floor bay is in reasonably good agreement with the value calculated from a structural analysis based on pure bending, for those cases where only flexural cracks develop in the transverse joints. If deformations due to shear can additionally occur in a cracked joint, the deflection will be greater than the calculated value.

9. The deformations of the ends of the floor bay are in reasonably good agreement with the values that a structural analysis for bending can be expected to yield.

10. Under monotonically increasing load a brittle fracture occurred as a result of shearing at grid line 2 at an average shear stress of 0.118 N/mm$^2$. Under repeating load, after 6200 cycles of varying level, failure occurred as a result of shearing at grid line 13 at an average shear stress of 0.086 N/mm$^2$.

11. As characteristic data relating to a cracked joint are as yet unavailable, it is in the present stage of this research not possible to calculate the ultimate load for the floor bay due to shear.
REFERENCES


APPENDIX A

A. MATERIAL PROPERTIES

The properties of the materials and of the precast units employed were determined in two ways:

a. from results of measurements performed in tests;
b. from design rules given in the code of practice VB 1974/1984 [9].

The properties envisaged in (a) were used in the calculations for the variants A, C, D and E, while those in (b) were used for variant B.

A1. Material properties determined from results of measurements

\[ \begin{align*}
\sigma_c & \quad \text{where:} \\
\epsilon_{cy} & = 0,101 \% \\
\epsilon_{cu} & = 0,35 \% \\
f_c & = 23,5 \text{ N/mm}^2 \\
E_c & = 23300 \text{ N/mm}^2 \\
\end{align*} \]

Fig. A1. $\sigma-\epsilon$ diagram for normal weight concrete of longitudinal beams.

a. Normal weight concrete for longitudinal beams:

The material properties of the normal weight (gravel aggregate) concrete used for these components are given in [1]. The following data stated there are more particularly of importance:

- average cube compressive strength:
  \[ f_{ccm} = 30.8 \text{ N/mm}^2 \]
- average prism compressive strength:
  \[ f_{cpm} = 23.5 \text{ N/mm}^2 \]
- average modulus of elasticity:
  \[ E_c = 23300 \text{ N/mm}^2 \]
The stress-strain diagram for the longitudinal beams can be defined as in Fig. A1.

b. Normalweight concrete for floor units:
The properties of the floor units (hollow core slabs) are described in Appendix I. The stress-strain diagram for these units can be defined as in Fig. A2.

\[
\begin{align*}
\sigma_c &= f_c \\
\varepsilon &= \varepsilon_c \\
\varepsilon_{cy} &= 0.0664 \times \arctan(E_c)
\end{align*}
\]

where:
\[
\begin{align*}
f_c &= 9.5 \text{ N/mm}^2 \\
E_c &= 14300 \text{ N/mm}^2 \\
\varepsilon_{cy} &= 0.0664 \\
\end{align*}
\]

Fig. A2. $\sigma$-$\varepsilon$ diagram for floor units (measured.)

c. Mortar joints:

\[
\begin{align*}
\sigma_c &= f_c \\
\varepsilon &= \varepsilon_c \\
\varepsilon_{cy} &= 1.5 \\
\varepsilon_{cu} &= 3.0 \\
\end{align*}
\]

where:
\[
\begin{align*}
f_c &= 24 \text{ N/mm}^2 \\
E_c &= 1600 \text{ N/mm}^2 \\
\varepsilon_{cy} &= 1.5 \% \\
\varepsilon_{cu} &= 3.0 \% \\
\end{align*}
\]

Fig. A3. $\sigma$-$\varepsilon$ diagram for mortar jointing (measured).

The properties of the jointing mortar have been described in [8]. A distinction should be drawn between the properties of the mortar itself and those of the mortar actually in situ in the joint between two precast units. Because of the restraint in the joint, the mortar can in fact resist considerably higher
stresses than when subjected to a uniaxial state of stress.

The properties of the mortar itself, measured on prism specimens (70mmx70mmx250mm), are as follows [8]:
- prism compressive strength:
  \( f_{cp} = 2.1 \text{ N/mm}^2 \)
- modulus of elasticity:
  \( E_c = 3600 \text{ N/mm}^2 \)
These values were determined after 14 days' hardening.

According to [8] the properties of the mortar can be defined by a stress-strain diagram as in Fig. A3 and furthermore by a \( \tau - \sigma \) diagram as in Fig. A4.

![Fig. A4. \( \tau - \sigma \) diagram for mortar jointing](image)

The parabolic curve in Fig. A4 can be represented by:
\[
t = T_0 + T_1 \sigma + T_2 \sigma^2 \text{ expressed in N/mm}^2
\]
where:
\( T_0 = 0.768 \text{ N/mm}^2 \)
\( T_1 = -0.736 \)
\( T_2 = -0.032 \text{ mm}^2/N \)

d. Connection between longitudinal beams:
The cross-sectional area of the longitudinal connecting bar is:
\( A_s = 22.6 \text{ mm}^2 \)
The stress-strain diagram of the connection can, according to [6], be defined as in Fig. A5.
where:
\[ f_s = 1150 \text{ N/mm}^2 \]
\[ E_s = 210 \times 10^3 \text{ N/mm}^2 \]
\[ \varepsilon_{sy} = 0.548 \% \]
\[ \varepsilon_{su} = 3.0 \% \]

Fig. A5. \( \sigma-\varepsilon \) diagram for longitudinal connection.


a. Normalweight concrete for longitudinal beams:
Starting from a cube strength \( f_c = 30.8 \text{ N/mm}^2 \) the values obtained in accordance with clauses A.201.3.1 and A.202.5.1 of VB 1974/1984 are:
\[ f_c = 0.8 \times 30.8 = 24.6 \text{ N/mm}^2 \]
\[ E_c = 0.9 \times (1800 - 4 \times 30.8) \sqrt{10 \times 30.8} = 26500 \text{ N/mm}^2 \]

The stress-strain diagram for the longitudinal beams can be defined as in Fig. A6.

where:
\[ f_c = 24.6 \text{ N/mm}^2 \]
\[ E_c = 26500 \text{ N/mm}^2 \]
\[ \varepsilon_{cy} = 0.0928 \% \]
\[ \varepsilon_{cu} = 0.35 \% \]

Fig. A6. \( \sigma-\varepsilon \) diagram for normalweight concrete of longitudinal beams.
b. Normalweight concrete for floor units:

For calculating the stress-strain diagram with the aid of the VB 1974/1984 rules the material properties as given in [1] were adopted. The following data stated there are more particularly of importance:

- **Average cube compressive strength:**
  \[ f_{ccm} = 36.6 \text{ N/mm}^2 \]

- **Average prism compressive strength:**
  \[ f_{cpm} = 29.2 \text{ N/mm}^2 \]

- **Average modulus of elasticity:**
  \[ E_c = 31400 \text{ N/mm}^2 \]

Adopting a cube strength \( f_c = 36.6 \text{ N/mm}^2 \) and basing oneself on the code of practice and the dimensions of the floor units, the properties of these units can be approximately calculated. For this purpose the units (hollow core slabs) are schematized as indicated in Fig. A7.

![Fig. A7. Schematization of floor units.](image-url)
Calculation of the average modulus of elasticity:

\[ \sigma_{\text{average}} = 0.48 \, E_c \]

According to VB 1974/1984, clauses A.201.3.1 and A.202.5.1:

\[ E_c = 0.9(1800-4\times36.6) \times 36.6 = 28500 \, \text{N/mm}^2 \]
\[ f_c = 0.8 \times 36.6 = 29.3 \, \text{N/mm}^2 \]

Therefore:

\[ \sigma_{\text{average}} = 0.48 \times 28500 = 13680 \, \text{N/mm}^2 \]

The gross stress at failure is:

\[ f_c(\text{gross}) = \frac{12}{40} \times f_c = \frac{12}{40} \times 29.3 = 8.8 \, \text{N/mm}^2 \]

On the basis of this calculation the stress–strain diagram for the floor units can be defined as in Fig. A8.

From a comparison of this diagram with Fig. A2 it appears that there is good agreement between the material properties of these units as determined from the results of measurements and those determined with the aid of the VB 1974/1984 rules.
c. Mortar joints

The properties of the mortar joint can also be determined by calculation with the rules given in VB 1974/1984, clause C.503.3.1. The calculation is based on the properties measured on prisms as reported in [8]:

\[ f_{cp} = 2.1 \text{ N/mm}^2 \]

According to clause C.715.3.1 the design value of the strength of the mortar joint is:

\[ f_{cv} = \eta_0 \beta f_c \]

where:

\[ \eta_0 = 1 \]

\[ f_c = \text{compressive strength of the adjacent units} = 8.8 \text{ N/mm}^2 \]

\[ \beta = \frac{5(1-k)+\delta^2}{5(1-k)+k\delta^2} \]

The joint can be schematized as follows:

```
+----------------------------------------+
|                                 | v=6 |
|                                 |-----|
|                                 |     |
|                                 |     |
|                                 |     |
|                                 |     |
|                                 |     |
|                                 |     |
+----------------------------------------+
```

\[ \delta = \frac{a}{v} = \frac{40}{6} = 6.67 \]

\[ k = \frac{\eta_{cm} f_{cp}}{f_c} = \frac{2.1}{8.8} = 0.24 \]

\[ \beta = \frac{5(1-0.24)+6.67^2}{5(1-0.24)+0.24*6.67^2} = 0.80 \]

\[ f_{cv} = 0.80 \times 8.8 = 7.0 \text{ N/mm}^2 \]

VB 1974/1984 gives no rules for calculating the modulus of elasticity of mortar joint.

Therefore the value of \( E_c \) measured on a 70\( \text{mm} \times 70\text{mm} \times 250\text{mm} \) prism, as reported in [8], will be adopted, namely:

\[ E_c = 3600 \text{ N/mm}^2 \]

The properties of the mortar joint can now be defined by a stress-strain diagram as in Fig. A9.
Fig. A9. σ-ε diagram for mortar joint (according to VB 74/84).

From a comparison of this diagram with Fig. A3 it appears that there is considerable difference between the properties of the joint as determined from the results of measurements and those determined with the aid of the VB 1974/1984 rules. The latter turn out to be very conservative for this type of joint and therefore on the safe side.

where:

\[ f_c = 4.1 \text{ N/mm}^2 \]
\[ E_c = 3600 \text{ N/mm}^2 \]
\[ \varepsilon_{cy} = 0.114 \% \]
\[ \varepsilon_{cu} = 0.35 \% \]
APPENDIX B

B. ANALYSIS OF THE ACTION OF FORCES AT GRID LINE 7

For the variants mentioned in Section 3 the stresses and longitudinal forces will be calculated for various limit states. The associated loading, curvature and stiffness will then be determined from those calculated values. The relation between the bending moment at grid line 7 and the total horizontal load follows from the schematic representation shown here:

\[ F = \frac{1}{12}Q \text{ (where } Q \text{ is the total horizontal load)} \]
\[ M = \frac{1}{2}Q \times 6 \times 0.72 - \frac{1}{12}Q(1+2+3+4+5) = 1.26Q \]
Therefore: \( Q = \frac{M}{1.26} \)

The curvature is determined from:
\[ \kappa = \frac{\varepsilon_c}{x} \]

where:
\( \varepsilon_c \) = strain at extreme compressive fibre
\( x \) = length of compressive zone

The stiffness \( EI \) is obtained from:
\[ EI = \frac{M}{x} \]
B 1. Variant A

The method of calculation according to variant A relates to a section which coincides with the transverse joint at grid lines 2 to 13, as indicated in the following diagram:

Limit state: yielding of the longitudinal connection

The distribution of forces at grid line 7 when the yield point in the connection between the beams is reached is as shown in Fig. B1. The stresses, the tensile and compressive forces and the bending moment in this limit state are calculated with the aid of the material properties in Section A1.

B1.1 Without tensile forces at the ends of the floor.

Fig. B1. Distribution of forces at grid line 7 for variant A.
The calculation is based on:

\[ \begin{align*}
A_s &= 22.6 \text{ mm}^2 \\
E_s &= 210 \times 10^3 \text{ N/mm}^2 \\
\sigma_{\text{sy}} &= 0.548 \% \\
E_c &= 1600 \text{ N/mm}^2
\end{align*} \]

From the distribution of forces as shown in Fig. B1 follows:

\[ \begin{align*}
\sigma_{s1} &= 5.48 \times 10^{-3} \\
\sigma_{s2} &= 5.48 \times 10^{-3} \frac{1710-X}{2910-X} \\
\sigma_{s3} &= 5.48 \times 10^{-3} \frac{1230-X}{2910-X} \\
\sigma_{c2} &= 5.48 \times 10^{-3} \frac{X-60}{2910-X} \\
\sigma_{c1} &= 5.48 \times 10^{-3} \frac{X}{2910-X}
\end{align*} \]

\[ \begin{align*}
N_{s1} &= 22.6 \times 5.48 \times 10^{-3} \times 210 \times 10^3 = 26000 \\
N_{s2} &= 26000 \frac{1710-X}{2910-X} \\
N_{s3} &= 26000 \frac{1230-X}{2910-X} \\
N_{c1} &= 40 \times 60 \times 5.48 \times 10^{-3} \frac{X-30}{2910-X} \times 1600 = 21043 \frac{X-30}{2910-X} \\
N_{c2} &= 1/2 \times 40 \times (X-60) \times 5.48 \times 10^{-3} \frac{X-60}{2910-X} \times 1600 = 175.36 \frac{(X-60)^2}{2910-X}
\end{align*} \]

\[ \Sigma H = 0: N_{s1} + N_{s2} + N_{s3} - N_{c1} - N_{c2} = 0 \]

\[ 26000 \left( 1 + \frac{1710-X}{2910-X} + \frac{1230-X}{2910-X} \right) - 21043 \cdot \frac{X-30}{2910-X} - 175.36 \frac{(X-60)^2}{2910-X} = 0 \]

Hence: \( X = 735.1 \text{ mm} \)
Check:
\[ \begin{align*} 
N_a &= 26000 \\
N_a &= 11655 \\
N_a &= 5916 \\
N_a &= 6822 \\
N_a &= 36747 \\
N_a &= 43571 \\
N_a &= 43569 \\
N_c &= 6822 \\
N_c &= 36747 \\
N_c &= 43569 \\
E_c &= 1.85 \times 10^{-3} \\
E_c &= 1.70 \times 10^{-3} \\
\sigma_c &= 2.96 \text{ N/mm}^2 < 24 (= f_c \text{ mortar joint}) \\
\sigma_c &= 2.72 \text{ N/mm}^2 < 9.5 (= f_c \text{ floor unit}) \\
\end{align*} \]

\[ \begin{align*} 
\Sigma M = 0 : M &= N_a \times 2880 + N_a \times 1680 + N_a \times 1200 - N_c \times (X-60)^5 + 30 \\
&= 74.88 \times 10^{-6} + 19.58 \times 10^6 + 26.68 \times 10^6 - 9.37 \times 10^6 \\
&= 92.19 \times 10^6 \text{ Nmm} \\
&= 92.19 \text{ kNm} \\
Q &= \frac{M}{1.26} = \frac{92.19}{1.26} = 73.2 \text{ kN} \\
X &= \frac{E_c}{1/26} = \frac{1.85 \times 10^{-3}}{1/26} = 2517 \times 10^{-9} \text{ 1/mm} \\
E_l &= \frac{M}{X} = 36600 \text{ kNm} \\
\end{align*} \]

Bl.2 With tensile forces (\(F_{tot}=3.04 \text{ kN}\)) at the ends of the floor.

\[ \begin{align*} 
\Sigma H = 0 : N_a + N_a + N_a - N_c - N_c &= 3040 \\
26000 (1+ \frac{1710-X}{2910-X} + \frac{1230-X}{2910-X} - 21043 \frac{X-30}{2910-X} - 175.36 \frac{(X-60)^2}{2910-X}) &= 3040 \\
\end{align*} \]

Hence: \(X=715.0 \text{ mm}\)

Check:
\[ \begin{align*} 
N_a &= 26000 \\
N_a &= 11786 \\
N_a &= 6100 \\
N_a &= 6567 \\
N_a &= 34275 \\
\Sigma N &= 3044 \\
E_c &= 1.785 \times 10^{-3} \Rightarrow \sigma_c = 2.86 \text{ N/mm}^2 \\
\end{align*} \]
\[ \sum M = 0 : M = N_{s1}x1440+N_{s2}x240-N_{s3}x240+N_{c1}x1440+N_{c2}(1410-\frac{X-60}{3}) \]
\[ = 37.44x10^6 + 2.83x10^6 - 1.46x10^6 + 9.46x10^6 + 40.84x10^6 \]
\[ = 89.11x10^6 \text{ Nmm} \]
\[ = 89.11 \text{ kNm} \]

\[ Q = \frac{M}{1.26} = \frac{89.11}{1.26} = 70.7 \text{ kN} \]

\[ X = \frac{\varepsilon c_1}{X} = \frac{1.785x10^{-3}}{715} = 2497x10^{-9} \text{ l/mm} \]

\[ EI = \frac{M}{X} = 35687 \text{ kNm}^2 \]

B2. Variant B

The method of calculation according to variant B relates to a section which coincides with the transverse joint at grid lines 2 to 13, as indicated in the following diagram:

Limit state: yielding of the longitudinal connection

The distribution of forces at grid line 7 when the yield point in the connection between the longitudinal beams is reached is as shown in Fig. B2. The stresses, the tensile and compressive forces and the bending moment in this limit state are calculated with the aid of the material properties in Section A2.
Fig. B2 Distribution of forces at grid line 7 for variant B.

The calculation is based on:

\[ A_s = 22.6 \text{ mm}^2 \]
\[ E_s = 210 \times 10^3 \text{ N/mm}^2 \]
\[ \varepsilon_{sy} = 0.548\% \]
\[ E_c = 3600 \text{ N/mm}^2 \]

From the distribution of forces as shown in Fig. B2 follows:

\[ \varepsilon_{s1} = 5.48 \times 10^{-3} \]
\[ \varepsilon_{s2} = 5.48 \times 10^{-3} \quad 1710-X \]
\[ 2910-X \]
\[ \varepsilon_{s3} = 5.48 \times 10^{-3} \quad 1230-X \]
\[ 2910-X \]
\[ \varepsilon_{c2} = 5.48 \times 10^{-3} \quad \frac{X-60}{2910-X} \]
\[ \varepsilon_{c1} = 5.48 \times 10^{-3} \quad \frac{X}{2910-X} \]

\[ N_{s1} = 22.6 \times 5.48 \times 10^{-3} \times 210 \times 10^3 = 26000 \]
\[ N_{s2} = 26000 \quad \frac{1710-X}{2910-X} \]
\[ N_{s3} = 26000 \quad \frac{1230-X}{2910-X} \]
\[ \begin{align*}
N_{c1} &= 40 \times 60 \times 5.48 \times 10^{-3} \frac{x-30}{2910-x} \times 3600 = 47347 \frac{x-30}{2910-x} \\
N_{c2} &= 1/2 \times 40 \times (x-60) \times 5.48 \times 10^{-3} \frac{x-60}{2910-x} \times 3600 = 394.56 \frac{(x-60)^2}{2910-x} \\
\Sigma H &= 0: N_{s1} + N_{s2} + N_{s3} - N_{c1} - N_{c2} = 0 \\
26000(1+\frac{1710-x}{2910-x}) + \frac{1230-x}{2910-x} - 47347 \frac{x-30}{2910-x} - 394.56 \frac{(x-60)^2}{2910-x} &= 0
\end{align*} \]

Hence: \( x = 529.8 \text{ mm} \)

Check:
\[
\begin{align*}
N_{s1} &= 26000 \\
N_{s2} &= 12892 \\
N_{s3} &= 7648 \\
N_{c1} &= 9942 \\
N_{c2} &= 36587
\end{align*}
\]

\[ \begin{align*}
\epsilon_{c1} &= 1.22 \times 10^{-3} \Rightarrow \sigma_{c1} = 4.39 \text{ N/mm}^2 < 24 (= \text{fc mortar joint}) \\
\epsilon_{c2} &= 1.08 \times 10^{-3} \Rightarrow \sigma_{c2} = 3.89 \text{ N/mm}^2 < 8.8 (= \text{fc floor unit}) \\
\epsilon &= \frac{\epsilon_{c1}}{x} = \frac{1.22 \times 10^{-3}}{529.8} = 2303 \times 10^{-9} \text{ 1/mm} \\
EI &= \frac{M}{x} = 42,900 \text{ kNm}^2
\end{align*} \]
B3. Variant C

Limit state: yielding of the longitudinal connection

The distribution of forces at grid line 7 on attainment of the yield point in the connection between the longitudinal beams in this variant is determined from the average deformations of a 120 mm wide strip. The forces in this limit state are calculated with the aid of the material properties in Section A1. The average moduli of elasticity are determined on the basis of the dimensions indicated in Fig. B3.

Calculation of average moduli of elasticity ($E_{\text{average}}$).

**Portion I at the longitudinal beams:**

The shortening due to a normal stress $\sigma$ is:

$$\Delta l = \frac{\sigma}{E_1} l_1 + \frac{\sigma}{E_2} l_2 = \frac{\sigma}{E_{\text{average}}} l_{\text{tot}}$$

where:

- $E_1 =$ modulus of elasticity of normalweight concrete = 23300 N/mm²
- $E_2 =$ modulus of elasticity of mortar joint = 1600 N/mm²
- $l_1 =$ overall length of beams and column = 114 mm
- $l_2 =$ overall width of mortar joint = 6 mm

Fig. B3 Plan of floor at grid line 7.
It follows that:

\[
\Delta l = \frac{0.114 \times 23300 + 0.6 \times 1600}{23300 + 1600} = \frac{0.120}{13900} \text{ N/mm}^2
\]

\[E_{\text{average}} = 13900 \text{ N/mm}^2\]

**Portion II** at the floor units:

\[
\Delta l = \frac{E_3}{l_3} \cdot 0.13 + \frac{E_2}{l_2} = \frac{E_{\text{average}}}{l_{\text{total}}}
\]

where: \(E_3 = \text{modulus of elasticity of floor units} = 14300 \text{ N/mm}^2\)

\(l_3 = \text{overall width of floor units} = 114 \text{ mm}\)

It follows that:

\[
\Delta l = \frac{0.114 \times 14300 + 0.6 \times 1600}{14300 + 1600} = \frac{0.120}{10200} \text{ N/mm}^2
\]

\[E_{\text{average}} = 10200 \text{ N/mm}^2\]

The distribution of forces at grid line 7 when the yield point in the longitudinal connection is reached is shown in Fig. B4. The stresses, the tensile and compressive forces and the bending moment can be calculated with the aid of the average material properties as calculated above for the portions I and II.

Fig. B4 Distribution of forces at grid line 7 for variant C.
The calculation is based on:

\[ A_s = 22.6 \text{ mm}^2 \]
\[ E_s = 210 \times 10^3 \text{ N/mm}^2 \]
\[ A_c(\text{beam}) = 40 \times 60 = 2400 \text{ mm}^2 \]
\[ E_c(\text{beam+joint}) = 13900 \text{ N/mm}^2 \]
\[ E_c(\text{floor+joint}) = 10200 \text{ N/mm}^2 \]
\[ \varepsilon_{sy} = 0.548\% \]

From the distribution of forces as shown in Fig. B4 follows:

\[ \varepsilon_s = 5.48 \times 10^{-3} \]
\[ \varepsilon_1 = 5.48 \times 10^{-3} \frac{1710-X}{2910-X} \]
\[ \varepsilon_2 = 5.48 \times 10^{-3} \frac{1230-X}{2910-X} \]
\[ \varepsilon_3 = 5.48 \times 10^{-3} \frac{X-60}{2910-X} \]
\[ \varepsilon_c = 5.48 \times 10^{-3} \frac{X}{2910-X} \]

\[ N_{s1} = 22.6 \times 5.48 \times 10^{-3} \times 210 \times 10^3 = 26000 \]
\[ N_{s2} = 26000 \frac{1710-X}{2910-X} \]
\[ N_{s3} = 26000 \frac{1230-X}{2910-X} \]
\[ N_c1 = 40 \times 60 \times 5.48 \times 10^{-3} \frac{x-30}{2910-X} \times 13900 = 182.81 \times 10^3 \frac{x-30}{2910-X} \]
\[ N_c2 = \frac{1}{2} \times 40 \times (X-60) \times 5.48 \times 10^{-3} \frac{x-60}{2910-X} \times 10200 = 1117.9 \frac{(X-60)^2}{2910-X} \]

\[ \sum H = 0: N_{s1} + N_{s2} + N_{s3} - N_c1 - N_c2 = 0 \]

\[ 26000 \left(1+\frac{1710-X}{2910-X} + \frac{1230-X}{2910-X}\right) - 182.81 \times 10^3 \frac{x-30}{2910-X} - 1117.9 \frac{(X-60)^2}{2910-X} = 0 \]

Hence: \( X = 318.3 \text{ mm} \)
Check:

\[ N_{a1} = 26000 \]
\[ N_{a2} = 13961 \] \[ \sum N_a = 49107 \]
\[ N_{a3} = 9146 \]
\[ N_{c1} = 20336 \] \[ \sum N_c = 49114 \]
\[ N_{c2} = 28778 \]

\[ \varepsilon_{c1} = 0.673 \times 10^{-3} \Rightarrow \sigma_{c1} = 0.673 \times 10^{-3} \times 13900 = 9.36 \text{ N/mm}^2 < 24 \text{ (=fc mortar joint)} \]

\[ \varepsilon_{c2} = 0.546 \times 10^{-3} \Rightarrow \sigma_{c2} = 0.546 \times 10^{-3} \times 10200 = 5.57 \text{ N/mm}^2 < 9.5 \text{ (=fc floor unit)} \]

\[ \sum M = 0 : M = N_{a1} \times 2880 + N_{a2} \times 1680 + N_{a3} \times 1200 - N_{c2} \left( \frac{X-60}{3} + 30 \right) = \]
\[ = 74.88 \times 10^6 + 23.45 \times 10^6 + 10.98 \times 10^6 - 3.34 \times 10^6 = \]
\[ = 105.97 \times 10^6 \text{ Nmm} \]
\[ = 105.97 \text{ kNm} \]

\[ Q = \frac{M}{1.26} = \frac{105.97}{1.26} = 84.1 \text{ kN} \]

\[ X = \frac{\varepsilon_{c1}}{X} = \frac{0.673 \times 10^{-3}}{318.3} = 2114 \times 10^{-9} \text{ 1/mm} \]

\[ EI = \frac{M}{X} = 50100 \text{ kNm}^2 \]
B4. Overview of variants, A, B and C

The calculated results from Sections B1, B2 and B3 are assembled in Table B1.

<table>
<thead>
<tr>
<th></th>
<th>variant A</th>
<th>variant B</th>
<th>variant C</th>
</tr>
</thead>
<tbody>
<tr>
<td>M (kNm)</td>
<td>92.19</td>
<td>98.89</td>
<td>105.97</td>
</tr>
<tr>
<td>Q (kN)</td>
<td>73.2</td>
<td>78.5</td>
<td>84.1</td>
</tr>
<tr>
<td>Ns1 (kN)</td>
<td>26.00</td>
<td>26.00</td>
<td>26.00</td>
</tr>
<tr>
<td>Ns2 (kN)</td>
<td>11.65</td>
<td>12.89</td>
<td>13.96</td>
</tr>
<tr>
<td>Ns3 (kN)</td>
<td>5.92</td>
<td>7.65</td>
<td>9.15</td>
</tr>
<tr>
<td>Nc1 (kN)</td>
<td>6.81</td>
<td>9.94</td>
<td>20.34</td>
</tr>
<tr>
<td>Nc2 (kN)</td>
<td>36.75</td>
<td>36.59</td>
<td>28.78</td>
</tr>
<tr>
<td>(\Delta) (l/mm)</td>
<td>2517x10^{-9}</td>
<td>2303x10^{-9}</td>
<td>2114x10^{-9}</td>
</tr>
<tr>
<td>EI (kNm²)</td>
<td>36600</td>
<td>42900</td>
<td>50100</td>
</tr>
</tbody>
</table>

Table B1 Overview of results of calculations.

These values relate only to the attainment of the yield point in the connection between the longitudinal beams.

For the sake of mutual comparability of the variants A, B and C the calculated results for a total horizontal load Q=6 kN are assembled in Table B2.

<table>
<thead>
<tr>
<th></th>
<th>variant A</th>
<th>variant B</th>
<th>variant C</th>
</tr>
</thead>
<tbody>
<tr>
<td>M (kNm)</td>
<td>7.56</td>
<td>7.56</td>
<td>7.56</td>
</tr>
<tr>
<td>Q (kN)</td>
<td>6.0</td>
<td>6.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Ns1 (kN)</td>
<td>2.13</td>
<td>1.99</td>
<td>1.86</td>
</tr>
<tr>
<td>Ns2 (kN)</td>
<td>0.96</td>
<td>0.99</td>
<td>1.00</td>
</tr>
<tr>
<td>Ns3 (kN)</td>
<td>0.49</td>
<td>0.58</td>
<td>0.65</td>
</tr>
<tr>
<td>Nc1 (kN)</td>
<td>0.56</td>
<td>0.76</td>
<td>1.45</td>
</tr>
<tr>
<td>Nc2 (kN)</td>
<td>3.01</td>
<td>2.80</td>
<td>2.05</td>
</tr>
<tr>
<td>(\Delta) (l/mm)</td>
<td>206x10^{-9}</td>
<td>176x10^{-9}</td>
<td>151x10^{-9}</td>
</tr>
<tr>
<td>EI (kNm²)</td>
<td>36600</td>
<td>42900</td>
<td>50100</td>
</tr>
</tbody>
</table>

Table B2 Overview of results of calculations for Q=6 kN.
The relation between the calculated results for the variants A, B and C is indicated in Fig. B5, where the tensile forces in the longitudinal connection at grid line 7 have been plotted for $Q=6$ kN.

![Graph](image)

**Fig. B5** Calculated tensile forces at grid line 7 for $Q = 6$ kN.

The relations between the calculated bending moments and the curvatures ($\kappa$) up to attainment of the yield point in the connection between the longitudinal beams for the three variants which have been analysed are presented in Fig. B6. Note that this limit state does not necessarily correspond to failure of the floor bay, because failure may occur much earlier in consequence of transverse shear.

A choice from among these three variants will be made for analysing the measured results obtained in the tests. The reasons for making this choice are given in Sections B7 and D4.
Fig. B6 Calculated M-X diagram at grid line 7.
B5. Results of measurements in series C

For comparing the calculated values with the measured values of the tensile forces in the longitudinal connections at grid line 7 a total horizontal load $Q = 6 \text{kN}$ is chosen as a reference value. This value corresponds to the characteristic wind load.

The principal results of the measurements in test series C are presented in [1]. From Fig. B7, adopted from that report, it appears that all the tests in which there is no tensile force at the ends exhibit identical behaviour with regard to the longitudinal forces at grid line 7. That is why it is justifiable to take as the basis of comparison the average of all the tests without tensile force at the ends.

Fig. B7 Relation between load and longitudinal force at grid line 7 for test series C.
On the basis of the structural analysis a linear relation can be expected to exist between the horizontal load and the tensile forces in the longitudinal connections. From Fig. B7 it is apparent that the linear relation in most cases occur only after a load of between 2 and 3 kN has been reached. The reason for this behaviour may be that at first some friction has to be overcome at the bearings on the column heads. In order to eliminate this effect, a correction will be applied to the measured values. For this purpose, in determining the corrected longitudinal forces, only the slope of the diagram in Fig. B7a after a load Q=4 kN has been reached will be considered.

The tensile forces at grid line 7 - the average measured values as well as the values corrected in the manner indicated - are assembled in Table B3, which also gives the relation $\alpha$ between the increase in the tensile forces and the increase in the load from Q=4 kN onwards. This relation can be expressed as follows:

$$\alpha = \frac{\Delta N_s}{\Delta Q}$$

where:

$\Delta N_s$ = increase in tensile force

$\Delta Q$ = increase in load

The corrected values of the tensile forces are then calculated (for Q=6 kN) from:

$$N_s = \alpha . Q$$

<table>
<thead>
<tr>
<th>connection at grid point</th>
<th>tensile force $N_s$ for $Q = 6$ kN (kN)</th>
<th>relation $\alpha$: $\alpha = \frac{\Delta N_s}{\Delta Q}$ (-)</th>
<th>corrected tensile force $N_s = \alpha . Q$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D7</td>
<td>1.76</td>
<td>0.313</td>
<td>1.88</td>
</tr>
<tr>
<td>C7</td>
<td>0.73</td>
<td>0.135</td>
<td>0.81</td>
</tr>
<tr>
<td>B7</td>
<td>0.35</td>
<td>0.060</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Table B3 Measured and corrected tensile forces in longitudinal connection at grid line 7.
The data given in Table B3 have been plotted in Fig. B8, which also includes the tensile forces in the longitudinal connections at grid line 7 for a total horizontal load Q=6 kN.

![Graph showing tensile forces at grid line 7 for Q=6 kN in test series C.](image)

**Fig. B8** Measured tensile forces at grid line 7 for Q=6 kN in test series C.
B6. Results obtained with ZEFE computer program

The principal data obtained from the calculations performed with the aid of the ZEFE computer program are assembled in [6]. The relation between load and tensile force at point D7, obtained from those calculations, is represented in Fig. B9.

![Graph showing relation between load and tensile force at D7 according to ZEFE.]

Fig. B9 Relation between load and tensile force at D7 according to ZEFE.

The calculated tensile forces at grid line 7 for a total horizontal load of 13.4 kN are assembled in Table B4.

<table>
<thead>
<tr>
<th>load</th>
<th>longitudinal connection at</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D7</td>
</tr>
<tr>
<td>Q=13.4 kN</td>
<td>4.72</td>
</tr>
</tbody>
</table>

Table B4 Tensile forces at grid line 7 for Q=13.4 kN according to ZEFE
It is to be noted that the basis adopted for the ZEFE calculations does not entirely correspond to that adopted for the simplified calculations according to the variants A, B and C.

The differences are as follows:
(a) with ZEFE the presence of a joint between the longitudinal beams and the column heads has not been taken into account;
(b) from further tests it has emerged that the actual modulus of elasticity of the floor units at right angels to the cavities in them is about twice the value adopted in the ZEFE calculations.

These differences must be given due consideration in the interpretation of the calculated values.

For mutual comparability of the measured and the calculated values a total horizontal load $Q = 6 \text{kN}$ has been chosen as the reference load. From Fig. B9, however, it appears that for this load crack had yet been formed at grid line 7, so that the tensile forces in the longitudinal connections were not able to develop.

<table>
<thead>
<tr>
<th>load</th>
<th>longitudinal connection at</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D7</td>
</tr>
<tr>
<td></td>
<td>(kN)</td>
</tr>
<tr>
<td>$Q=6 \text{kN}$</td>
<td>2.11</td>
</tr>
</tbody>
</table>

Table B5 Tensile forces at grid line 7 for $Q=6 \text{kN}$ according to ZEFE.

To make possible a comparison between the calculated and the measured values it can, according to Fig. B9, reasonably be assumed that at a cracked section there exists a linear relation between the load and the tensile force in the longitudinal connection. The tensile forces at grid line 7 calculated in this way are assembled in Table B5. The results given in this table have been plotted in Fig. B10.
Fig. B10 Calculated tensile forces at grid line 7 for Q=6 kN with ZEFE computerprogram.
B7. Comparison between calculated and measured values

The calculated and the measured values of the tensile forces in the longitudinal connection at grid line 7 have been plotted for a horizontal load \( Q = 6 \text{ kN} \) in Fig. B11. This diagram is a combination of Figs. B5, B8 and B10.

Fig. B11 Overview of tensile forces at grid line 7, comprising measured and calculated results.

It shows that the measured values as well as the corrected values are all smaller than the calculated values. It also shows that there is good agreement in the slope of the connecting lines between variant A, ZEFE and the measured and corrected values.

It is not clear why the measured values are lagging behind the calculated values. Conceivably, friction may have caused this, though it should have been eliminated in the corrected values. Another possible cause could be the
influence that tensile stresses in the transverse joint may have on the pattern of forces in the connection. The shear deformation in the longitudinal joints could also have played a part in causing the difference, for this aspect is indeed comprised in the computer program, but not in the simplified method of analysis.

Despite the difference in the method of calculation between the ZEFE computer program and the simplified analysis according to variant A there is good agreement in the slope of the connecting lines. The reason for this agreement could be that the distribution of forces at grid line 7 is due entirely to the properties of the mortar joint. This causal factor is the same for both methods of calculation.

From Fig. B11 it is also apparent that variant B is a reasonable alternative in a case where only the strength properties of the materials are known. The other properties deducible therefrom with the aid of the VB 1974/1984 rules give reliable results.

Adopting a slab strip instead of a section for calculating the distribution of forces is clearly not a very good basis. Evidently it does not allow a sufficiently accurate description of the actual distribution of forces at a transverse joint.

For the above-mentioned reasons only the method according to variant A will be used for analysing the measured results.
APPENDIX C

C. ANALYSIS OF THE ACTION OF FORCES BETWEEN GRID LINES 7 AND 8

The action of forces between the grid lines 7 and 8 will be investigated in two ways, as described below with reference to the variants D and E.

Cl. Variant D

The method of calculation according to variant D relates to a section at the mortar joint between the floor units, as shown in the diagram below. The beam is assumed to be uncracked between grid lines 7 and 8, and the mortar joint is assumed to be unable to transmit tensile stress. For the material properties the data given in Section A1 will be adopted.

Limit state: yielding of the longitudinal connection.

From Section B1 it appears that the yield point of the connection between the longitudinal beams is reached at a bending moment $M=92.19$ kNm. This moment is
taken as the limiting value for the calculation of the distribution of forces between grid lines 7 and 8, which is shown in Fig. Cl.

Fig. Cl Distribution of forces between grid lines 7 and 8, variant D.

The calculation is based on:

\[ Ac \text{ (beam)} = 6668 \text{ mm}^2 \]
\[ Ec \text{ (beam)} = 23300 \text{ N/mm}^2 \]
\[ Ec \text{ (joint)} = 23.5 \text{ N/mm}^2 \]
\[ Ec \text{ (joint)} = 1600 \text{ N/mm}^2 \]
\[ Ec \text{ (joint)} = 24 \text{ N/mm}^2 \]

From the distribution of forces as shown in Fig. Cl follows:

\[ \varepsilon_{c2} = \frac{X-60}{X} \cdot \varepsilon_{c1} \]
\[ \varepsilon_{c3} = \frac{X-1200}{X} \cdot \varepsilon_{c1} \]
\[ \varepsilon_{c4} = \frac{X-1260}{X} \cdot \varepsilon_{c1} \]
\[ \varepsilon_{ct5} = \frac{1680-X}{X} \cdot \varepsilon_{c1} \]
\[ \varepsilon_{ct6} = \frac{1740-X}{X} \cdot \varepsilon_{c1} \]
\[
\begin{align*}
\varepsilon_{c17} &= \frac{2880-X}{X} \cdot \varepsilon_{c1} \\
\varepsilon_{c18} &= \frac{2940-X}{X} \cdot \varepsilon_{c1}
\end{align*}
\]

Hence:

\[
\begin{align*}
N_{c1} &= \frac{X-30}{X} \cdot \varepsilon_{c1} \times 23300 \times 6668 = 155.36 \times 10^6 \frac{X-30}{X} \cdot \varepsilon_{c1} \\
N_{c2} &= \frac{X-60}{X} \cdot \varepsilon_{c1} \times 1600 \times 1/2 \times 1440 \times 40 = 36.48 \times 10^6 \frac{X-60}{X} \cdot \varepsilon_{c1} \\
N_{c3} &= \frac{X-1200}{X} \cdot \varepsilon_{c1} \times 1600 \times 1/2 \times 1140 \times 40 = 36.48 \times 10^6 \frac{X-1200}{X} \cdot \varepsilon_{c1} \\
N_{c4} &= \frac{X-1230}{X} \cdot \varepsilon_{c1} \times 23300 \times 6668 = 155.36 \times 10^6 \frac{X-1230}{X} \cdot \varepsilon_{c1} \\
N_{c5} &= \frac{X-1260}{X} \cdot \varepsilon_{c1} \times 1600 \times 1/2 \times 40 \times (X-1260) = 32000 \frac{(X-1260)^2}{X} \cdot \varepsilon_{c1} \\
N_{c6} &= \frac{1710-X}{X} \cdot \varepsilon_{c1} \times 23300 \times 6668 = 155.36 \times 10^6 \frac{1710-X}{X} \cdot \varepsilon_{c1} \\
N_{c7} &= \frac{2910-X}{X} \cdot \varepsilon_{c1} \times 23300 \times 6668 = 155.36 \times 10^6 \frac{2910-X}{X} \cdot \varepsilon_{c1}
\end{align*}
\]

\[
\Sigma H = 0 : N_{c1} + N_{c2} + N_{c3} + N_{c4} + N_{c5} - N_{c6} - N_{c7} = 0
\]

\[
\frac{(X-30+X-1230+1710+X-2910+X)}{155.36 \times 10^6 \varepsilon_{c1}} \cdot \frac{X}{X} + \\
+ \varepsilon_{c1} \left\{ \frac{36.48 \times 10^6}{X} \frac{X-60}{X} + \frac{36.48 \times 10^6}{X} \frac{X-1200}{X} + 32000 \frac{(X-1260)^2}{X} \right\} = 0
\]

Hence: \( x = 1381.1 \text{ mm} \)

Check:

\[
\begin{align*}
N_{c1} &= 151.99 \times 10^6 \varepsilon_{c1} \\
N_{c2} &= 34.90 \times 10^6 \varepsilon_{c1} \\
N_{c3} &= 4.78 \times 10^6 \varepsilon_{c1} \\
N_{c4} &= 17.0 \times 10^6 \varepsilon_{c1} \\
N_{c5} &= 0.34 \times 10^6 \varepsilon_{c1}
\end{align*}
\]

\[\Sigma N_c = 209.0 \times 10^6 \times \varepsilon_{c1}\]
The longitudinal forces at grid line 7 for a total horizontal load \(Q=73.2\) kN and \(Q=6\) kN are assembled in Table C1.

<table>
<thead>
<tr>
<th></th>
<th>Longitudinal forces (in N) for Q=73.2 kN</th>
<th>Longitudinal forces (in N) for Q=6 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>(N_c)</td>
<td>27054</td>
<td>2218</td>
</tr>
<tr>
<td>(N_c2)</td>
<td>6212</td>
<td>509</td>
</tr>
<tr>
<td>(N_c3)</td>
<td>851</td>
<td>70</td>
</tr>
<tr>
<td>(N_c4)</td>
<td>3026</td>
<td>248</td>
</tr>
<tr>
<td>(N_c5)</td>
<td>61</td>
<td>5</td>
</tr>
<tr>
<td>(N_c6)</td>
<td>6586</td>
<td>540</td>
</tr>
<tr>
<td>(N_c7)</td>
<td>30614</td>
<td>2511</td>
</tr>
</tbody>
</table>

Table C1 Longitudinal forces at grid line 7.
C2. Variant E

The method of calculation according to variant E relates to a 240 mm wide strip, taking account of the difference in the moduli elasticity of the floor unit and the mortar joint. See Fig. C2.

Fig. C2 Plan of floor between grid lines 7 and 8.

The beam is assumed to be uncracked between grid lines 7 and 8, and the mortar joint is assumed to be unable to transmit tensile stress. For the material properties the data given in Section A1 will be adopted.

Limit state: yielding of the longitudinal connection

The yield point of the connection between the longitudinal beams is, as has been calculated in Section B3, reached at a bending moment $M=105.97 \text{ kNm}$. 
Calculation of the average moduli of elasticity:

**Portion I** at the longitudinal beams:

\[ E_{c_{\text{beam}}} = 23300 \text{ N/mm}^2 \]

**Portion II** at the floor units:

The shortening due to a normal stress is:

\[
\Delta l = \frac{\sigma}{E_1} \cdot l_1 + \frac{\sigma}{E_2} \cdot l_2 = \frac{\sigma}{E_{\text{average}}} \cdot l_{\text{tot}}
\]

where:

- \( E_1 = \) modulus of elasticity of floor units = 14300 N/mm\(^2\)
- \( l_1 = \) overall width of floor units = 234 mm
- \( E_2 = \) modulus of elasticity of mortar joint = 1600 N/mm\(^2\)
- \( l_2 = \) width of mortar joint

Hence:

\[
\Delta l = \frac{\sigma \times 234}{14300} + \frac{\sigma \times 6}{1600} = \frac{\sigma \times 240}{E_{\text{average}}}
\]

\[ E_{\text{average}} = 11900 \text{ N/mm}^2 \]

The distribution of forces in a strip located between grid lines 7 and 8 is shown in Fig. C3.

---

Fig. C3 Distribution of forces between grid lines 7 and 8, variant E.
The calculation is based on:
\[ A_c (\text{beam}) = 6668 \text{ mm}^2 \]
\[ E_c (\text{beam}) = 23300 \text{ N/mm}^2 \]
\[ \sigma_c (\text{beam})_{\text{max}} = 23.5 \text{ N/mm}^2 \]
\[ E_c (\text{joint+slab}) = 11900 \text{ N/mm}^2 \]
\[ \sigma_c (\text{joint+slab}) = 24 \text{ N/mm}^2 \]

From the distribution of forces as shown in Fig. C3 follows:

\[ \xi_c_2 = \frac{X-60}{X} \cdot \xi_c_1 \]
\[ \xi_c_3 = \frac{1200-X}{X} \cdot \xi_c_1 \]
\[ \xi_c_4 = \frac{1260-X}{X} \cdot \xi_c_1 \]
\[ \xi_c_5 = \frac{1680-X}{X} \cdot \xi_c_1 \]
\[ \xi_c_6 = \frac{1740-X}{X} \cdot \xi_c_1 \]
\[ \xi_c_7 = \frac{2880-X}{X} \cdot \xi_c_1 \]
\[ \xi_c_8 = \frac{2940-X}{X} \cdot \xi_c_1 \]

Hence:

\[ N_{c1} = \frac{X-30}{X} \cdot \xi_c_1 \times 23300 \times 6668 = 155.36 \times 10^6 \cdot \frac{X-30}{X} \cdot \xi_c_1 \]
\[ N_{c2} = \frac{X-60}{X} \cdot \xi_c_1 \times 11900 \times 1/2 \times 40 (X-60) = 238.0 \times 10^3 \cdot \frac{(X-60)^2}{X} \cdot \xi_c_1 \]
\[ N_{c3} = \frac{1230-X}{X} \cdot \xi_c_1 \times 23300 \times 6668 = 155.36 \times 10^6 \cdot \frac{1230-X}{X} \cdot \xi_c_1 \]
\[ N_{c4} = \frac{1710-X}{X} \cdot \xi_c_1 \times 23300 \times 6668 = 155.36 \times 10^6 \cdot \frac{1710-X}{X} \cdot \xi_c_1 \]
\[ N_{c5} = \frac{2910-X}{X} \cdot \xi_c_1 \times 23300 \times 6668 = 155.36 \times 10^6 \cdot \frac{2910-X}{X} \cdot \xi_c_1 \]
\[ \Sigma H = 0 : N_{c1} + N_{c2} - N_{ct3} - N_{ct4} - N_{cts} = 0 \]

\[
155.36 \times 10^6 \epsilon_{c1} \frac{(X-30-1230+X-1710+X-2910+X)}{X} + 238 \times 10^3 \epsilon_{c1} \frac{(X-60)^2}{X} = 0
\]

Hence: \( X = 1075.2 \text{ mm} \)

Check:

\[
N_{c1} = 151.03 \times 10^6 \epsilon_{c1}
\]

\[
N_{c2} = 228.15 \times 10^6 \epsilon_{c1}
\]

\[
N_{ct3} = 22.36 \times 10^6 \epsilon_{c1}
\]

\[
N_{ct4} = 91.71 \times 10^6 \epsilon_{c1}
\]

\[
N_{cts} = 265.1 \times 10^6 \epsilon_{c1}
\]

\[ \Sigma M = 0 : M = 92.19 \text{ kNm} \]

\[
M = (N_{c1} + N_{cts}) \times 1440 + (N_{ct4} - N_{ct3}) \times 240 + N_{c2} (1440 - 30 - \frac{X-60}{3}) =
\]

\[
= (599.23 \times 10^9 + 16.64 \times 10^9 + 244.48 \times 10^9) \times \epsilon_{c1} = 860.35 \times 10^9 \epsilon_{c1} = 105.97 \times 10^6
\]

\[
\epsilon_{c1} = 0.123 \times 10^{-3} \Rightarrow \sigma_{c1} = 0.123 \times 10^{-3} \times 23 \times 300 = 2.87 \text{ N/mm}^2 < 23.5
\]

\[
\epsilon_{c2} = \frac{X-60}{X} \cdot \epsilon_{c1} = 0.116 \times 10^{-3} \Rightarrow
\]

\[
\sigma_{c2}(\text{floor+joint}) = 0.116 \times 10^{-3} \times 11 \times 900 = 1.38 \text{ N/mm}^2 < 24
\]

\[
x = \frac{\epsilon_{c1}}{\epsilon_{c1}} = \frac{0.123 \times 10^{-3}}{1075.2} = 115 \times 10^{-9} \text{ l/mm}
\]

\[
EI = \frac{M}{x} = 925 \times 10^3 \text{ kNm}^2
\]

\[
Q = \frac{M}{1.26} = \frac{105.97}{1.26} = 84.1 \text{ kN}
\]
The longitudinal forces at grid line 7 for a total horizontal load \( Q = 84.1 \) kN and \( Q = 6 \) kN are assembled in Table C2.

<table>
<thead>
<tr>
<th></th>
<th>( N_{c1} )</th>
<th>( N_{c2} )</th>
<th>( N_{c3} )</th>
<th>( N_{c4} )</th>
<th>( N_{c5} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Q = 84.1 ) kN</td>
<td>18602</td>
<td>28101</td>
<td>2754</td>
<td>11296</td>
<td>32653</td>
</tr>
<tr>
<td>( Q = 6 ) kN</td>
<td>1327</td>
<td>2005</td>
<td>196</td>
<td>806</td>
<td>2330</td>
</tr>
</tbody>
</table>

Table C2 Longitudinal forces at grid line 7.

From Table C1 it appears that for variant D the tensile stress in the longitudinal beam between grid lines 7 and 8 for a total horizontal force \( Q = 73.2 \) kN is:

\[
\sigma_{ct} = \frac{N_{c7}}{A_c} = \frac{30614}{6668} = 4.6 \text{ N/mm}^2
\]

From Table C2 it appears that for variant B the tensile stress in the longitudinal beam between grid lines 7 and 8 for a total horizontal force \( Q = 84.1 \) kN is:

\[
\sigma_{ct} = \frac{N_{c5}}{A_c} = \frac{32653}{6668} = 4.9 \text{ N/mm}^2
\]

According to Table 2.3 of Stevin Report 5-84-4 [1] the average splitting tensile strength for series C is:

\[ f_{ct} = 2.4 \text{ N/mm}^2 \]

On the assumption that the flexural (tensile) strength under short-term loading is equal to the splitting tensile strength, cracking in the longitudinal beam for variant D will occur for a total load equal to:
\[
Q_r = \frac{2.4}{4.6} \times 73.2 = 38.2 \text{ kN}
\]

and for variant E:
\[
Q_r = \frac{2.4}{4.9} \times 84.1 = 41.2 \text{ kN}
\]

The calculation for the limit state of yielding of the beam reinforcement gives the values stated in Table C3.

<table>
<thead>
<tr>
<th></th>
<th>variant D</th>
<th>variant E</th>
</tr>
</thead>
<tbody>
<tr>
<td>M (kNm)</td>
<td>220.4</td>
<td>222.2</td>
</tr>
<tr>
<td>Q (kN)</td>
<td>174.9</td>
<td>176.4</td>
</tr>
<tr>
<td>(x ) (1/mm)</td>
<td>852x10^-9</td>
<td>796x10^-9</td>
</tr>
</tbody>
</table>

Table C3 Overview of results of calculation for yielding of beam reinforcement

This calculation for the yielding of the beam reinforcement is not included in the present report.

C3. Comparison of results from variants D and E.

The results of the calculations for the curvatures (\(x\)) and the stiffnesses EI for the variants D and E are assembled in Table C4. These values relate to a bending moment at which the yield point in the connection between the longitudinal beams at grid line 7 is reached. Besides, to illustrate the relation with the variants A, B and C, the values of the curvature and stiffness for a total horizontal load \(Q=6 \text{ kN}\) are included in Table C5.

Fig. C4 represents the moment-curvature (M-X) diagrams for the variants A, B, C, D and E. The variants C and E relate to a structural analysis comprising a strip with a modulus of elasticity averaged over the width of the strip. The variants A, B and D relate to a structural analysis comprising a section at the mortar joint between the floor units.
<table>
<thead>
<tr>
<th>at cracking moment in longitudinal beam</th>
<th>at yield of beam reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>variant D</td>
<td>variant E</td>
</tr>
<tr>
<td>$M$ (kNm)</td>
<td>48.1</td>
</tr>
<tr>
<td>$Q$ (kN)</td>
<td>38.2</td>
</tr>
<tr>
<td>$x$ (1/mm)</td>
<td>$67 \times 10^{-9}$</td>
</tr>
<tr>
<td>$EI$ (kNm²)</td>
<td>715000</td>
</tr>
</tbody>
</table>

Table C4 Overview of results of calculation, variants D and E.

<table>
<thead>
<tr>
<th>variant D</th>
<th>variant E</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M$ (kNm)</td>
<td>7.56</td>
</tr>
<tr>
<td>$Q$ (kN)</td>
<td>6.0</td>
</tr>
<tr>
<td>$x$ (1/mm)</td>
<td>$10.6 \times 10^{-9}$</td>
</tr>
<tr>
<td>$EI$ (kNm²)</td>
<td>715000</td>
</tr>
</tbody>
</table>

Table C5 Overview of results of calculation for $Q=6$ kN.

<table>
<thead>
<tr>
<th>at yielding of longitudinal connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>variant D</td>
</tr>
<tr>
<td>$M$ (kNm)</td>
</tr>
<tr>
<td>$Q$ (kN)</td>
</tr>
<tr>
<td>$x$ (1/mm)</td>
</tr>
</tbody>
</table>

Table C6 Overview of results of calculation for yielding of longitudinal connection.
As appears from Fig. C4, the difference between the variants D and E is negligible. For the analysis of the deflection of the floor bay the choice was arbitrarily made in favour of variant D.

The values for the curvature corresponding to the yielding moment were calculated with the data of Table C4 for the variants A, B and C. For variant D these data are assembled in Table C6.
APPENDIX D

D. ANALYSIS OF THE DEFLECTION OF THE LONGITUDINAL EDGE

In this Appendix the calculation will be performed in accordance with a simplified model. The calculations presented in Appendices B and C will be used for the purpose. The several variants that have been considered will provide the basis for the present analysis. In order to arrive at a choice from among the variants the results obtained will be compared with measured values found in the tests. The results obtained with the ZEFE computer program will also be included in this comparison.

D.1. Calculated deflection based on simplified model

For calculating the deflection of the longitudinal edge the floor bay is schematized as shown in Fig. D1. It is conceived as an assembly of 600mm wide rigid segments between the grid lines and 120mm wide flexible segments at the grid lines. The width of 120mm has been chosen because this dimension corresponds to the length of the connector bars between the longitudinal beams.

Fig. D1 Schematization of the floor bay.
The deflection is calculated from the moment produced by the curvature diagram, in which $\xi_1$ is the curvature of the flexible segments at grid lines 7 and 8, while $\xi_2$ is the curvature of the rigid segment between these two grid lines. See Fig. D2.

The relation between the moments and curvatures at grid lines 2 to 7 is presented in Table D1.

![Curvature diagram for the floor bay.](image)

<table>
<thead>
<tr>
<th>moment</th>
<th>curvature at:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>flexible</td>
</tr>
<tr>
<td></td>
<td>segment</td>
</tr>
<tr>
<td>$M_2$ = 0.286 M</td>
<td>$0.286 \xi_1$</td>
</tr>
<tr>
<td>$M_3$ = 0.524 M</td>
<td>$0.524 \xi_1$</td>
</tr>
<tr>
<td>$M_4$ = 0.714 M</td>
<td>$0.714 \xi_1$</td>
</tr>
<tr>
<td>$M_5$ = 0.857 M</td>
<td>$0.857 \xi_1$</td>
</tr>
<tr>
<td>$M_6$ = 0.952 M</td>
<td>$0.952 \xi_1$</td>
</tr>
<tr>
<td>$M_7$ = 1.0 M</td>
<td>$1.0 \xi_1$</td>
</tr>
</tbody>
</table>

Table D1 Relation between $M$, $\xi_1$ and $\xi_2$ at grid lines.
The deflection is:

$$\delta = \frac{5}{48} x x_2 x 9360^2 = 9.126 \times 10^6 x_2$$

$$+0.286(x_1-x_2) x 120 x 720 = 0.025 \times 10^6 (x_1-x_2)$$

$$+0.524(x_1-x_2) x 120 x 2 x 720 = 0.091 \times 10^6 (x_1-x_2)$$

$$+0.714(x_1-x_2) x 120 x 3 x 720 = 0.185 \times 10^6 (x_1-x_2)$$

$$+0.857(x_1-x_2) x 120 x 4 x 720 = 0.296 \times 10^6 (x_1-x_2)$$

$$+0.952(x_1-x_2) x 120 x 5 x 720 = 0.411 \times 10^6 (x_1-x_2)$$

$$+1.0 (x_1-x_2) x 120 x 6 x 720 = 0.518 \times 10^6 (x_1-x_2)$$

$$\delta = 1.526 \times 10^6 x_1 + 7.60 \times 10^6 x_2$$

The results of the calculations of the curvatures for the variants A to E - calculated in Appendix B, Sections B1, B2 and B3 and in Appendix C, Sections C1 and C2 - are given in Table D2. These results relate to a total horizontal load $Q=6 \text{ kN}$.

<table>
<thead>
<tr>
<th>curvature</th>
<th>variants</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>$x_1$ (1/mm)</td>
<td>206$x10^{-9}$</td>
<td>176$x10^{-9}$</td>
<td>151$x10^{-9}$</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>$x_2$ (1/mm)</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>10.6$x10^{-9}$</td>
<td>8.2$x10^{-9}$</td>
<td></td>
</tr>
</tbody>
</table>

Table D2 Overview of curvatures between grid lines 7 and 8 for $Q=6 \text{ kN}$.

Variant A + D

For a combination of the variants A and D the calculated deflection is:

$$\delta = 1.526 \times 10^6 x 206 x 10^{-9} + 7.60 \times 10^6 x 10.6 x 10^{-9} = 0.39 \text{ mm}$$
Variant B + D

For a combination of the variants B and D the calculated deflection is:
\[ \delta = 1.526 \times 10^6 \times 176 \times 10^{-9} + 7.60 \times 10^6 \times 10.6 \times 10^{-9} = 0.35 \text{ mm} \]

Variant C + D

For a combination of the variants C and D the calculated deflection is:
\[ \delta = 1.526 \times 10^6 \times 151 \times 10^{-9} + 7.60 \times 10^6 \times 10.6 \times 10^{-9} = 0.31 \text{ mm} \]

The deflections can similarly be calculated for the cracking moment between grid lines 7 and 8 and for the yielding of the longitudinal connection at grid line 7. The results of this calculation are given in Tables D3 and D4. The data in these tables are represented in a Q-δ diagram in Fig. D3.

<table>
<thead>
<tr>
<th>moment (kNm)</th>
<th>deflection δ for:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A+D (mm)</td>
</tr>
<tr>
<td>48.1</td>
<td>2.51</td>
</tr>
</tbody>
</table>

Table D3 Deflection of floor bay at cracking moment.
Table D4 Deflection of floor bay at yielding of longitudinal connection.

<table>
<thead>
<tr>
<th>Moment (kNm)</th>
<th>Deflection δ for:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A+D (mm)</td>
</tr>
<tr>
<td>92.19</td>
<td>5.88</td>
</tr>
<tr>
<td>98.89</td>
<td>-</td>
</tr>
<tr>
<td>105.97</td>
<td>-</td>
</tr>
</tbody>
</table>

Fig. D3 Q-δ diagram for calculated values at D7.
D2. Results of measurements in series C

Only the tests C401 and C601 are suitable for comparing the measured results with the calculated values for the deflections of the longitudinal edge. Test C401 relates to a floor bay without a preformed crack, and test C601 to a floor bay with a preformed crack at grid line 2. The results obtained in these tests are reported in [1]. Two diagrams from that report are reproduced here as Figs. D4 and D5.

In Fig. D4 the total load has been plotted against the deflection at D7, and in Fig. D5 the deflection curve has been plotted for a total horizontal load Q=6 kN.

---

Fig. D4 Q-δ diagram for tests C401 and C601.
Fig. D5 Deflection diagrams for tests C401 and C601 at Q=6 kN.

D3. Results obtained with ZEFE computer program

Report [6] contains the principal data obtained from the calculation performed with the aid of the ZEFE computer program. The Q-δ diagram in Fig. D6, based on those data, represents the relation between the total horizontal load and the deflection of the longitudinal edge at D7.

Fig. D6 Q-δ diagram calculated with ZEFE.
D4. Comparison between calculated and measured values

In Fig. D7 the calculated deflection at D7 obtained from a simplified model, from the ZEFE computer program and from the measurements performed in the tests C401 and C601 have been plotted in one Q-δ diagram. The following conclusions can be drawn from it:

a) There is reasonably good agreement between the measured values of test C601 and the calculated values for variant A+D.

b) After the total load exceeds about Q=10 kN there is good agreement between the measured values of test C401 and the calculated values for variant B+D and those obtained from the ZEFE computer program.

c) The calculated values for variant C+E show less good agreement with the measured values.

Fig. D7 Q-δ diagram with calculated and measured values.
In view of these conclusions only the calculation procedure according to variant A+D will be employed for analysing the results of the measurements, since this procedure gives the most conservative results and is moreover in agreement with the conclusion in Appendix B, Section B7, where variant A was found to yield the most serviceable results.
APPENDIX E

E. SHEAR FORCES IN THE LONGITUDINAL JOINTS

EL. Longitudinal joint at grid line A

For calculating the shear forces in the longitudinal joints the floor bay is schematized as shown in Fig. E1. For this purpose it is divided into 240 mm wide strips, this being the width of the floor units. Forces as indicated in Figs. E2 and E3 act on each strip. In the strip shown in Fig. E2 the forces acting on the left-hand side are those calculated in Appendix B, while those acting on the right-hand side have been calculated in Appendix C. In the strip shown in Fig. E3 the forces acting on both sides are those calculated in Appendix C.

Fig. E1 Schematization of the floor bay.

The shear forces can now be calculated from the internal equilibrium, e.g.:

\[ V_{AB} = N_{c1}(r) - N_{c1}(1) \]

The shear is assumed to be uniformly distributed over the available area.
For the strips beside the grid lines in the transverse direction the average shear stress is:

$$\tau_{AB} = \frac{V_{AB}}{40 \times 193}$$
For the strips between the grid lines in the transverse direction the average shear stress is:
\[ \tau_{AB} = \frac{V_{AB}}{40 \times 240} \]

In order to obtain insight into the influence that the design methods presented in Appendices B and C has upon the magnitude of the longitudinal shear stresses, in this Appendix these stresses are calculated for the combination of the variants A+D and C+E.

E2. Longitudinal joints at grid lines B, C and D

The shear forces and the average shear stresses at grid lines B, C and D can be calculated in the same way. These data, for a total horizontal load \( Q = 6 \) kN, are assembled in Tables E1 and E2, where \( V_{AB} \) and \( V_{BA} \) denote the shear forces near the grid lines A and B respectively in the portion AB.

Figs. E4 and E5 are graphical representations of the shear stresses in the longitudinal joints for a total horizontal load \( Q = 6 \) kN.

From these figures it is apparent that the choice of the calculation variants is of major influence on the values obtained for the shear stresses in the longitudinal joints. This aspect will be further considered in Chapter 6.
<table>
<thead>
<tr>
<th>section</th>
<th>moment</th>
<th>$V_{AB}$</th>
<th>$\tau_{AB}$</th>
<th>$V_{BA}$</th>
<th>$\tau_{BA}$</th>
<th>$V_{BC}=V_{CB}$</th>
<th>$\tau_{BC}=\tau_{CB}$</th>
<th>$V_{CD}=V_{DC}$</th>
<th>$\tau_{CD}=\tau_{DC}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(kNm)</td>
<td>(N)</td>
<td>(N/mm²)</td>
<td>(N)</td>
<td>(N/mm²)</td>
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<td>(N/mm²)</td>
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Table E1 Overview of shear forces and shear stresses in longitudinal joints for Q=6 kN, variants A+D.
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Table E2 Overview of shear forces and shear stresses in longitudinal joints for \( Q=6 \) kN, variants C + E.
Fig. E4 Graphical representation of shear stresses in longitudinal joints for $Q=6kN$, variants A+D.
Fig. E5 Graphical representation of shear stresses in longitudinal joints for \( Q = 6 \) kN, variants C + E.
APPENDIX F

F. SHEAR FORCES AT GRID LINES 2 AND 13

From the analysis given in Section 2.5 it appears that the magnitude of the shear force depends on the mechanism that will develop in the transverse joint. If there is no question of a shear deformation, the mechanism will occur as shown schematically in Fig. Fl. But if the associated maximum shear force has been attained, the shear friction effect may give rise to a mechanism as shown schematically in Fig. F2.

Fig. Fl Stress distribution in a transverse joint without shear deformation.

Fig. F2 Stress distribution in a transverse joint with shear deformation.
Because there are insufficient data concerning a \( \tau - \sigma - w - s \) relation for a cracked joint it is not possible to carry out a numerical analysis on the basis of the mechanism in Fig. F2. Accordingly, in this Appendix the analysis will be confined to the mechanism in Fig. Fl.

**Fl. Without tensile forces at the ends of the floor**

The shear force that can be resisted at grid lines 2 and 13 can be calculated from the distribution of the normal stress in the section (see Fig. Fl) and the associated distribution of the shear stress.

The calculation of the normal stress in the transverse joint has been performed in Appendix B, Section Bl.1. Variant A is assumed to be applicable. The calculation of the shear stress is based on the stress-strain \((\tau - \sigma)\) diagram as described in Appendix A, Section Al.c.

**Limit state: failure due to shear at grid line 2**

Suppose: \( V = \) maximum shear force that can be resisted = \( 2/3 \) b.x.\( \tau \)

For this purpose it is assumed for the sake of convenience that the point where the shear stress has its maximum (i.e., at 1/2 x) is the critical point with regard to shear failure.

The bending moment at grid line 2 is then:

\[ M_{(2)} = 0.72 \, V \, (\text{kNm}) \]

From the calculation in Appendix B, Section Bl.1:

\[ M = 92.19 \, \text{kNm} \]
\[ \sigma_c = 2.96 \, \text{N/mm}^2 \]
\[ x = 735.1 \, \text{mm} \]

Hence for: \( M_{(2)} = 0.72 \, V \) it follows that:

\[ M_{(2)} = 0.72 \, V \]
\[ \sigma_c = \frac{0.72 \, V}{92.19} \times 2.96 = 23.12 \times 10^{-3} \, V \, (\text{N/mm}^2) \]

(where \( V \) is expressed in kN)
The average normal stress is:

\[ \sigma_{c(m)} = \frac{1}{2} \times 23.12 \times 10^{-3} \times 11.56 \times 10^{-3} \text{ V} (\text{N/mm}^2) \]

The equation of the parabola of the \( \tau-\sigma \) diagram is:

\[ \tau = T_0 + T_1 \sigma + T_2 \sigma^2 \text{ (N/mm}^2) \]

where:

\[ T_0 = 0.768 \text{ N/mm}^2 \]
\[ T_1 = -0.736 \]
\[ T_2 = -0.032 \text{ mm}^2/\text{N} \]

Hence:

\[ \tau = 0.768 + 0.736 \times 11.56 \times 10^{-3} \text{ V} - 0.032 (11.56 \times 10^{-3} \text{ V})^2 = \]
\[ = 0.768 + 8.51 \times 10^{-3} \text{ V} - 4.28 \times 10^{-6} \text{ V}^2 \text{ (N/mm}^2) \] (1)

Furthermore:

\[ V = \frac{2/3 \times r \times 40 \times 735.1}{1000} = 19.6 \text{ kN} \]

or \[ \tau = \frac{V}{19.6} \text{ (N/mm}^2) \] (2)

Equation (1) and (2) gives:

\[ \frac{V}{19.6} = 0.768 + 8.51 \times 10^{-3} \text{ V} - 4.28 \times 10^{-6} \text{ V}^2 \]

from which is obtained:

\[ V = 18.03 \text{ kN} = V_u \]

The failure load is therefore: \( Q_u = 2V_u = 2 \times 18.03 = 36.06 \text{ kN} \)

Bending moment at grid line 7: \( M(7) = 1.26 \times 36.06 = 45.4 \text{ kNm} < 92.19 \text{ kNm} \)

Bending moment at grid line 2: \( M(2) = 18.03 \times 0.72 = 12.98 \text{ kNm} \)

The longitudinal forces at grid line 2 for \( V_u = 18.03 \text{ kN} \) and \( M(2) = 13.2 \text{ kN} \) are:
F2. With tensile force (F=3.04 kN) at the ends of the floor

If the floor bay is loaded at its ends with a tensile force perpendicular to the transverse joints, a tensile force will initially occur in all the connections between the beams. When the horizontal loading is increased, the tensile force at grid line A will decrease until a compressive zone capable of resisting the shear stresses develops in the joint.

Assuming that shear stresses can be resisted only in a compressive zone, failure would theoretically occur directly after loading of the structure. In the tests this did not in fact occur, the reason being that cracked joints under tensile load are still able to resist shear stresses.

In order to obtain some idea of the magnitude of the horizontal load at which a compressive zone is formed, the distribution of forces for which zero stress develops at grid line A will now first be calculated.

(a) Limit state: zero stress at grid line A

The distribution of forces at grid line 2 when there is zero stress at grid line A is indicated in Fig. F3. From this diagram it is possible, with the aid of the conditions $\Sigma H = 0$ and $\Sigma M = 0$, to calculate the longitudinal forces at the section.

From the distribution of forces in Fig. 3 follows:

$$N_{s2} = \frac{1680}{2880} N_{s1}$$

$$N_{s3} = \frac{1200}{2880} N_{s1}$$

\[
N_{s1} = \frac{12.98}{92.19} \times 26000 = 3661 \, N
\]

\[
N_{s2} = \frac{12.98}{92.19} \times 11655 = 1641 \, N
\]

\[
N_{s3} = \frac{12.98}{92.19} \times 5916 = 833 \, N
\]
Fig. F3 Distribution of forces at grid line 2

\[ \sum H = 0 : N_1 + N_2 + N_3 = F \]

\[ N_1 \cdot \left(1 + \frac{1680}{2880} + \frac{1200}{2880} \right) = 3040 \]

\[ N_1 = 1520 \text{ N} \]
\[ N_2 = 887 \text{ N} \]
\[ N_3 = 633 \text{ N} \]

\[ \sum M = 0 : M = N_1 \times 1440 + N_2 \times 240 - N_3 \times 240 = \]
\[ = 1520 \times 1440 + 887 \times 240 - 633 \times 240 = \]
\[ = 2.25 \times 10^6 \text{ Nmm} \]
\[ = 2.25 \text{ kNm} \]

\[ Q = 2 \times \frac{2.25}{0.72} = 6.25 \text{ kN} \]

On the assumption that the shear force can be resisted by the entire transverse joint, the average shear stress is:

\[ \tau = \frac{V}{b \cdot h} = \frac{1/2 \times 6250}{40 \times 2910} = 0.027 \text{ N/mm}^2 \]
(b) Limit state: failure due to shear at grid line 2

The distribution of forces at grid line 2 for a bending moment at which shear failure may occur at that grid line is shown in Fig. F4.

Fig. F4 Distribution of forces at grid line 7, variant A.

With the aid of the conditions $\Sigma H=0$ and $\Sigma M=0$ the longitudinal forces at the section can be calculated from this diagram. This calculation has to be performed iteratively until the ultimate transverse force that can be resisted is equal to the actual transverse force.

This calculation is based on the $\sigma-\varepsilon$ diagrams as given in Section Al of Appendix A, for which:

- $A_s = 22.6 \text{ mm}^2$
- $\varepsilon_{\text{max}} = 0.548\%$
- $\sigma_{\text{c,max}} = 24 \text{ N/mm}^2$
- $E_c = 1600 \text{ N/mm}^2$
From the distribution of forces in Fig. F4 it follows that:

\[
\epsilon_{s1} = \frac{2910-X}{X} \cdot \epsilon_c
\]

\[
\epsilon_{s2} = \frac{1710-X}{X} \cdot \epsilon_c
\]

\[
\epsilon_{s3} = \frac{1230-X}{X} \cdot \epsilon_c
\]

\[
N_{s1} = 22.6 \times \frac{2910-X}{X} \cdot \epsilon_c \times 10^3 = 4746 \times 10^3 \frac{2910-X}{X} \cdot \epsilon_c
\]

\[
N_{s2} = 22.6 \times \frac{1710-X}{X} \cdot \epsilon_c \times 10^3 = 4746 \times 10^3 \frac{1710-X}{X} \cdot \epsilon_c
\]

\[
N_{s3} = 22.6 \times \frac{1230-X}{X} \cdot \epsilon_c \times 10^3 = 4746 \times 10^3 \frac{1230-X}{X} \cdot \epsilon_c
\]

\[
N_c = \frac{1}{2} \times 40 \times X \times \epsilon_c \times 1600 = 32000 X \cdot \epsilon_c
\]

\[
\Sigma H = 0 : N_{s1} + N_{s2} + N_{s3} - N_c = F
\]

\[
4746 \times 10^3 \cdot \epsilon_c \left( \frac{2910-X}{X} + \frac{1710-X}{X} + \frac{1230-X}{X} \right) - 32000X \cdot \epsilon_c - 3040 = 0
\]

\[
4746 \times 10^3 \cdot \epsilon_c \frac{(5850-3X)}{X} - 32000X \cdot \epsilon_c - 3040 = 0
\]

\[
X^2 + 445X + \frac{95 \times 10^{-3}X}{\epsilon_c} - 867.5 \times 10^3 = 0
\]

\[
\Sigma M = 0 : M = N_{s1} \times 1440 + N_{s2} \times 240 - N_{s3} \times 240 + N_c \left( 1470 - \frac{X}{3} \right)
\]

For \( \epsilon_c = 0.1 \times 10^{-3} \)

\( \sigma_c = 0.16 \text{ N/mm}^2 \)

\[
\Sigma H = 0 : X^2 + 445X + \frac{95 \times 10^{-3}X}{0.1 \times 10^{-3}} - 867.5 \times 10^3 = 0
\]

\[
X^2 + 1395X - 867.5 \times 10^3 = 0
\]

\[
X = 466.1 \text{ mm}
\]
\[ N_1 = 2488 \text{ N} \]
\[ N_2 = 1267 \text{ N} \]
\[ N_3 = 778 \text{ N} \]
\[ N_c = 1491 \text{ N} \]
\[ \sum M = 0 : = 2488 \times 1440 + 1267 \times 240 - 778 \times 240 + 1491 \left(1470 - \frac{466.1}{3}\right) = \]
\[ = 5.66 \times 10^6 \text{ Nmm} \]
\[ = 5.66 \text{ kNm} \]

The actual shear force is:

\[ V = \frac{M}{0.72} = 7.86 \text{ kN} \]

The maximum permissible shear stress associated with

\[ C = - \frac{1}{2} C_c = - 0.08 \text{ N/mm}^2 \] is:

\[ \tau_u = 0.768 + 0.736 \times 0.08 - 0.032 \times 0.08^2 = 0.827 \text{ N/mm}^2 \]

The shear force that can be resisted is:

\[ V_u = 2/3 \times 0.827 \times 40 \times 466.1 = 10280 \text{ N} \] (too high)

For \( C_{c1} = 0.15 \times 10^{-3} \):

\[ C_{c1} = 0.24 \text{ N/mm}^2 \]

\[ \sum H = 0 : X^2 + 445 X + \frac{95 \times 10^{-3} X}{0.15 \times 10^{-3}} - 867.5 \times 10^3 = 0 \]

\[ X^2 + 1078 X - 867.5 \times 10^3 = 0 \]

\[ X = 537.0 \text{ mm} \]

\[ N_1 = 3146 \text{ N} \]
\[ N_2 = 1555 \text{ N} \]
\[ N_3 = 919 \text{ N} \]
\[ N_c = 2578 \text{ N} \]

\[ \sum M = 0 : M = 3146 \times 1440 + 1555 \times 240 - 919 \times 240 + 2578 \left(1470 - \frac{537.0}{3}\right) = \]
\[ = 8.01 \times 10^6 \text{ Nmm} \]
\[ = 8.01 \text{ kNm} \]

The actual shear force is:

\[ V = \frac{M}{0.72} = 11.13 \text{ kN} \]
The maximum permissible shear stress associated with 
\[ \sigma = -1/2 \sigma_c = -0.12 \text{ N/mm}^2 \text{ is:} \]
\[ \tau_u = 0.768 + 0.736 \times 0.12 - 0.032 \times 0.12^2 = 0.856 \text{ N/mm}^2 \]
The shear force that can be resisted is:
\[ V_u = 2/3 \times 0.856 \times 40 \times 537.0 = 12259 \text{ N (too high)} \]
For \( c_1 = 0.2 \times 10^{-3} \)
\[ \sigma_{c1} = 0.32 \text{ N/mm}^2 \]
\[ \sum H = 0 : X^2 + 445X + \frac{95 \times 10^{-3}X}{0.15 \times 10^{-3}} - 867.5 \times 10^{3} = 0 \]
\[ X^2 + 920X - 867.5 \times 10^{3} = 0 \]
\[ X = 578.8 \text{ mm} \]
\[ N_1 = 3823 \text{ N} \]
\[ N_2 = 1855 \text{ N} \]
\[ N_3 = 1068 \text{ N} \]
\[ N_c = 3704 \text{ N} \]
\[ \sum M = 0 : M = 3823 \times 1440 + 1855 \times 240 - 1068 \times 240 + 3704 \left(1470 - \frac{578.8}{3}\right) = 10.42 \times 10^6 \text{ Nmm} \]
\[ = 10.42 \text{ kNm} \]
The actual shear force is:
\[ V = \frac{M}{0.72} = 14.47 \text{ kN} \]
The maximum permissible shear stress associated with:
\[ \sigma = -1/2 \sigma_c = -0.16 \text{ N/mm}^2 \text{ is:} \]
\[ \tau_u = 0.768 + 0.736 \times 0.16 - 0.032 \times 0.16^2 = 0.885 \text{ N/mm}^2 \]
The shear force that can be resisted is:
\[ V_u = 2/3 \times 0.885 \times 40 \times 578.8 = 13660 \text{ N (too low)} \]
The optimum value can be determined graphically from the calculated shear force that can be resisted and the actual shear force. The calculated values of the shear force for different strains of the jointing mortar in the compressive zone are reviewed in Table Fl.
Table F1 Overview of calculated values of shear forces.

These values have been plotted graphically in Fig. F5. The optimum value of the shear force can then be determined by measurement.

![shear force (kN)](image)

**Fig. F5** Relation between shear force and compressive zone at grid line 2.

From interpolation of the curves for $V_u$ and $V$ in Fig. F5 it appears that the optimum shear force at grid line 2 is:

$V = V_u = 13.5$ kN.
The associated compressive zone is:

\[ X = 569 \text{ mm} \]

Hence:

\[ 0.095X^2 + 445X + \frac{X - 867.5 \times 10^3}{0.15\varepsilon_c} = 0 \]

from which is obtained:

\[ \varepsilon_c = 0.0186\% \]

The longitudinal forces corresponding to this are:

\[
\begin{align*}
N_1 &= 3633 \\
N_2 &= 1771 \\
N_3 &= 1026 \\
N_c &= 3388
\end{align*}
\]

\[ \Sigma N = 3042 \]

\[ \Sigma M = 0: M = 3633 \times 1440 + 1771 \times 240 - 1026 \times 240 + 3388 \left( 1470 - \frac{569}{3} \right) = \]

\[ = 9.75 \times 10^6 \text{ Nmm} \]

\[ = 9.75 \text{ kNnm} \]
G. DEFORMATION OF THE ENDS OF THE FLOOR AT GRID LINES 1 AND 14

The deformations at the ends of the floor bay comprise a rotation $\varphi$ and a displacement $\Delta l$ at grid line D (see Fig. G1). The rotation is calculated from the deflection of the bay at grid line 2, divided by the distance between grid lines 1 and 2:

$$\varphi = \frac{\Delta l}{720} \text{ rad}$$

The displacement of the points D1 and D14 is calculated from the sum of the elongations of the longitudinal connections. The elongation of the longitudinal beams is neglected, as is also the effect of the curvature of the floor bay. These two influences moreover counteract each other and are very slight in comparison with the extension of the connections between the longitudinal beams.

![Deformation of the floor bay diagram](image)

Fig. G1 Deformation of the floor bay.

The deformation of the ends of the floor bay at grid lines 1 and 14 can theoretically be determined from the calculated values of the displacements of...
the points D1 and D14 and the rotation $\varphi$.

**GI. Deformation at the ends for $Q=6$ kN and $F=0$ kN**

The deflection of the longitudinal edge at the point D2 is calculated with the aid of the data in Appendix D, Section D1, from which it follows (assuming the jointing mortar to be the governing factor):

$$
(\delta_7 - \delta_2) \times 10^9 = \frac{5}{48}(10.6 - 3.0) \times 7920^2 = 49.7 \times 10^6
$$

$$
+ \frac{1}{2} \times 3.0 (7920/2)^2 = 23.5 \times 10^6
$$

$$
+ (108.1 - 5.6) \times 120 \times 720 = 8.9 \times 10^6
$$

$$
+ (147.4 - 7.6) \times 120 \times 2 \times 720 = 24.2 \times 10^6
$$

$$
+ (176.9 - 9.1) \times 120 \times 3 \times 720 = 43.5 \times 10^6
$$

$$
+ (196.6 - 10.1) \times 120 \times 4 \times 720 = 64.4 \times 10^6
$$

$$
+ (206.4 - 10.6) \times 120 \times 5 \times 720 = 84.6 \times 10^6
$$

$$
\frac{(\delta_7 - \delta_2) \times 10^9}{22.6 \times 210 \times 10^3} = 298.8 \times 10^6
$$

$\delta_7 = 0.395$ mm (app. D, par. D1)

$\delta_2 = 0.395 - 0.299 = 0.096$ mm

Hence the rotation of grid line 1 is:

$$
\varphi = \frac{-\delta_2}{22.6 \times 210 \times 10^3} = 0.133 \times 10^{-3}
$$

Table GI gives the moments at the grid lines and the associated tensile forces and elongations in the longitudinal connections at grid line D. The elongation per connection has been calculated from:

$$
\Delta l_i = \frac{N_s \times l_i}{A_s \times E} = \frac{N_s \times 120}{22.6 \times 210 \times 10^3}
$$
Table G1 Overview of moments, longitudinal forces and elongations.

The displacement of the points D1 and D14 is therefore:

\[ \Delta l = 0.233 \text{ mm} \]

On the assumption that the sections at grid line 1 and at grid line 14, respectively, remain plane:

\[ a = \frac{\Delta l}{\phi} = \frac{0.233}{0.133 \times 10^{-3}} = 1752 \text{ mm} \]

The displacement of grid lines A, B, C and D at grid lines 1 or 14 can be calculated as follows:

\[ \Delta l_1 \text{ in as } D = 0.233 \text{ mm} \]
\[ \Delta l_2 \text{ in as } C = 0.233 - 0.133 \times 10^{-3} \times 1200 = 0.073 \text{ mm} \]
\[ \Delta l_3 \text{ in as } B = 0.233 - 0.133 \times 10^{-3} \times 1680 = 0.010 \text{ mm} \]
\[ \Delta l_4 \text{ in as } A = 0.233 - 0.133 \times 10^{-3} \times 2880 = -0.150 \text{ mm} \]
APPENDIX H

H. CRACK WIDTH IN THE TRANSVERSE JOINTS

H1. Crack width in the transverse joints at grid line D

The crack widths at the points where measurements were performed in the transverse joints close to grid line D can be calculated from the calculated elongation of the longitudinal connections and from the strain diagram of the relevant transverse joints. The crack width will now be calculated for a total horizontal load \( Q = 6 \) kN and on the assumption that the jointing mortar is the governing factor.

H1.1 Without tensile force at the ends of the floor

The longitudinal forces can be derived from Section 4.1 and the elongation of the connections from Table G1 of Appendix G. The distance from grid line D to the location of the strain gauge is 49 mm. See Fig. H1, from which it follows that the crack width at the strain gauge can be calculated from:

\[
W = \frac{2125.9}{2125.9 + 49} \times \Delta l_1
\]

Fig. H1 Strain diagram in cracked transverse joint.
The crack widths calculated in this way for the transverse joints are assembled in Table H1.

<table>
<thead>
<tr>
<th>grid line</th>
<th>elongation</th>
<th>crack width</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta l_1$ (mm)</td>
<td>$w$ (mm)</td>
</tr>
<tr>
<td>2</td>
<td>0.015</td>
<td>0.015</td>
</tr>
<tr>
<td>3</td>
<td>0.028</td>
<td>0.027</td>
</tr>
<tr>
<td>4</td>
<td>0.039</td>
<td>0.038</td>
</tr>
<tr>
<td>5</td>
<td>0.046</td>
<td>0.045</td>
</tr>
<tr>
<td>6</td>
<td>0.051</td>
<td>0.050</td>
</tr>
<tr>
<td>7</td>
<td>0.054</td>
<td>0.053</td>
</tr>
</tbody>
</table>

Table H1 Overview of crack widths at grid line D for $Q=6 \text{ kN}$.

H1.2 With tensile force ($F=3.04 \text{ kN}$) at the ends of the floor.

The elongation of the connections between the longitudinal beams and the resulting crack widths are derivable from a linear interpolation of the data from Section 4.1 for grid lines 3 to 7 and those from Section 4.7 for grid line 2. The values thus calculated are assembled in Table H2.

<table>
<thead>
<tr>
<th>grid line</th>
<th>moment</th>
<th>$N_{s1}$</th>
<th>elongation</th>
<th>crack width</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(kNm)</td>
<td>(N)</td>
<td>$\Delta l_1$ (mm)</td>
<td>$w_1$ (mm)</td>
</tr>
<tr>
<td>2</td>
<td>2.16</td>
<td>1490</td>
<td>0.038</td>
<td>0.037</td>
</tr>
<tr>
<td>3</td>
<td>3.96</td>
<td>2009</td>
<td>0.051</td>
<td>0.050</td>
</tr>
<tr>
<td>4</td>
<td>5.40</td>
<td>2414</td>
<td>0.061</td>
<td>0.060</td>
</tr>
<tr>
<td>5</td>
<td>6.48</td>
<td>2719</td>
<td>0.069</td>
<td>0.068</td>
</tr>
<tr>
<td>6</td>
<td>7.20</td>
<td>2922</td>
<td>0.074</td>
<td>0.072</td>
</tr>
<tr>
<td>7</td>
<td>7.56</td>
<td>3023</td>
<td>0.076</td>
<td>0.074</td>
</tr>
</tbody>
</table>

Table H2 Overview of moments, longitudinal forces, elongations and crack widths in transverse joints.
H2. Crack width at grid line 2

The crack widths at the points where measurements were performed in the transverse joint at grid line 2 are calculated from the calculated longitudinal forces in the longitudinal connections. The crack width is assumed to be equal to the elongation of the connector bars between the beams; these bars are taken as being 120mm in length. The crack width at a longitudinal connection is therefore:

$$\Delta = \frac{N_s x 1}{A_s E} = \frac{120}{22.6 \times 210 \times 10^3} \text{ Ns}=25.3 \times 10^{-6} \text{ Ns (N)}$$

The crack width has been calculated for the following total horizontal loads: Q=6, 12, 18, 24 and 30 kN

For calculating the crack width at the strain gauges a linear distribution as shown in Fig. H2 has been adopted.

![Diagram of forces](image)

**Fig. H2** Distribution of forces in a transverse joint (F=0 kN).

**H2.1 Without tensile force at the ends of the floor**

In Section F1 of Appendix F the longitudinal forces have been calculated for a
total horizontal load of 36.1 kN. The longitudinal force at the point D2 in that case is 3661 N. The longitudinal forces for all levels of loading can be determined from this.

For the given horizontal loading the longitudinal force at the point D2 is as follows:

for $Q=6\text{kN}$: $N_{\text{s}1}=6/36.1\times3661=608 \text{ N}$

$Q=12\text{kN}$: $N_{\text{s}1}=1217 \text{ N}$

$Q=18\text{kN}$: $N_{\text{s}1}=1825 \text{ N}$

$Q=24\text{kN}$: $N_{\text{s}1}=2434 \text{ N}$

$Q=30\text{kN}$: $N_{\text{s}1}=3042 \text{ N}$

The various measuring points are given coded designations.

For example: 2-DC denotes the measuring point at grid line 2 between the longitudinal grid lines D and C near grid line D.

The crack width can be calculated as follows:

$2-\text{DC} : \ w = \frac{2125.9}{2174.9} \times N_{\text{s}1} \times \frac{120}{22.6\times210\times10^3} = 24.7\times10^{-6} \ N_{\text{s}1}$

$2-\text{CD} : \ w = \frac{1023.9}{2174.9} \times N_{\text{s}1} \times \frac{120}{22.6\times210\times10^3} = 11.9\times10^{-6} \ N_{\text{s}1}$

$2-\text{CB} : \ w = \frac{925.9}{2174.9} \times N_{\text{s}1} \times \frac{120}{22.6\times210\times10^3} = 10.8\times10^{-6} \ N_{\text{s}1}$

$2-\text{BC} : \ w = \frac{543.9}{2174.9} \times N_{\text{s}1} \times \frac{120}{22.6\times210\times10^3} = 6.3\times10^{-6} \ N_{\text{s}1}$

$2-\text{BA} : \ w = \frac{445.9}{2174.9} \times N_{\text{s}1} \times \frac{120}{22.6\times210\times10^3} = 5.2\times10^{-6} \ N_{\text{s}1}$

The values thus calculated are assembled in Table H3.

H2.2 With tensile force ($F=3.04 \text{ kN}$) at the ends of the floor

In Section F2 of Appendix F the longitudinal forces at grid line 2 have been calculated for various loadings and a total horizontal tensile force $F=3.04\text{kN}$ at the ends of the floor bay. The values calculated there are assembled in Table H4.
Table H3 Overview of crack widths at grid line 2 for $F=0$ kN.

<table>
<thead>
<tr>
<th>meas. point</th>
<th>6</th>
<th>12</th>
<th>18</th>
<th>24</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-DC</td>
<td>0.015</td>
<td>0.030</td>
<td>0.045</td>
<td>0.060</td>
<td>0.075</td>
</tr>
<tr>
<td>2-CD</td>
<td>0.007</td>
<td>0.015</td>
<td>0.022</td>
<td>0.029</td>
<td>0.036</td>
</tr>
<tr>
<td>2-CB</td>
<td>0.006</td>
<td>0.013</td>
<td>0.020</td>
<td>0.026</td>
<td>0.033</td>
</tr>
<tr>
<td>2-BC</td>
<td>0.004</td>
<td>0.008</td>
<td>0.012</td>
<td>0.015</td>
<td>0.019</td>
</tr>
<tr>
<td>2-BA</td>
<td>0.003</td>
<td>0.006</td>
<td>0.009</td>
<td>0.013</td>
<td>0.016</td>
</tr>
</tbody>
</table>

Table H4 Overview of calculated longitudinal forces.

<table>
<thead>
<tr>
<th>$Q$ (kN)</th>
<th>$M_2$ (kNm)</th>
<th>$N_{s1}$ (N)</th>
<th>$N_{s2}$ (N)</th>
<th>$N_{s3}$ (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.25</td>
<td>2.25</td>
<td>1520</td>
<td>887</td>
<td>633</td>
</tr>
<tr>
<td>27.00</td>
<td>9.75</td>
<td>3633</td>
<td>1771</td>
<td>1026</td>
</tr>
<tr>
<td>15.72</td>
<td>5.66</td>
<td>2488</td>
<td>1267</td>
<td>778</td>
</tr>
<tr>
<td>22.25</td>
<td>8.01</td>
<td>3146</td>
<td>1555</td>
<td>919</td>
</tr>
<tr>
<td>28.94</td>
<td>10.42</td>
<td>3823</td>
<td>1855</td>
<td>1068</td>
</tr>
</tbody>
</table>

With linear interpolation from these data the longitudinal forces have been calculated for a total horizontal load of respectively: $Q=6.25$, 12, 18, 24 and 30 kN. These values are assembled in Table H5.
Calculation of the crack widths has been based on linearity of the strain distribution diagram as shown in Fig. H3.

The crack width can be calculated as follows:

\[
\begin{align*}
\text{2-DC : } w &= \left[ \left( N_{s1} - N_{s2} \right) \times \frac{1200-49}{1200} + N_{s2} \right] \times \frac{120}{22.6 \times 210 \times 10^3} \\
\text{2-CD : } w &= \left[ \left( N_{s1} - N_{s2} \right) \times \frac{49}{1200} + N_{s2} \right] \times \frac{120}{22.6 \times 210 \times 10^3}
\end{align*}
\]
The values thus calculated are assembled in Table H6.

<table>
<thead>
<tr>
<th>meas. point</th>
<th>crack width (mm) for Q (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>point</td>
<td>6.25</td>
</tr>
<tr>
<td>2-DC</td>
<td>0.037</td>
</tr>
<tr>
<td>2-CD</td>
<td>0.023</td>
</tr>
<tr>
<td>2-CB</td>
<td>0.022</td>
</tr>
<tr>
<td>2-BC</td>
<td>0.017</td>
</tr>
<tr>
<td>2-BA</td>
<td>0.015</td>
</tr>
</tbody>
</table>

Table H6  Overview of crack widths at grid line 2 for F=3.04 kN.

H3. Crack width at grid line 5

The crack widths at the measuring points in the transverse joint at grid line 5 have been calculated in the same way as for grid line 2 in Appendix F.

H3.1 Without tensile force at the ends of the floor

The calculation of the longitudinal force \( N_{s1} \) is based on the calculation given in Section A1.2.1, where \( M=92.19 \text{ kNm} \) and \( N_{s1}=26000 \text{ N} \). A linear relation between moment and longitudinal force is assumed. The transverse force associated with this is:

\[ V_s = 1/2 \ Q - 3.5/12 \ Q = 0.208 \ Q \]
The values thus calculated for \( M \), \( V \) and \( N_s1 \) are assembled in Table H7.

<table>
<thead>
<tr>
<th>load</th>
<th>moment</th>
<th>transverse force</th>
<th>longitudinal force</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q (kN)</td>
<td>( M_s ) (kNm)</td>
<td>( V_s ) (kN)</td>
<td>( N_{s1} ) (N)</td>
</tr>
<tr>
<td>6</td>
<td>6.48</td>
<td>1.25</td>
<td>1828</td>
</tr>
<tr>
<td>12</td>
<td>12.96</td>
<td>2.50</td>
<td>3655</td>
</tr>
<tr>
<td>18</td>
<td>19.44</td>
<td>3.75</td>
<td>5482</td>
</tr>
<tr>
<td>24</td>
<td>25.92</td>
<td>5.00</td>
<td>7310</td>
</tr>
<tr>
<td>30</td>
<td>32.40</td>
<td>6.25</td>
<td>9138</td>
</tr>
</tbody>
</table>

Table H7 Overview of moments, transverse forces and longitudinal forces at grid line 5 for \( F=0 \).

The crack widths can be calculated as described in Section H2.1 and are assembled in Table H8.

<table>
<thead>
<tr>
<th>meas. point</th>
<th>crack width (mm) for Q (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6</td>
</tr>
<tr>
<td>5-DC</td>
<td>0.045</td>
</tr>
<tr>
<td>5-CD</td>
<td>0.022</td>
</tr>
<tr>
<td>5-CD</td>
<td>0.020</td>
</tr>
<tr>
<td>5-BC</td>
<td>0.012</td>
</tr>
<tr>
<td>5-BA</td>
<td>0.009</td>
</tr>
</tbody>
</table>

Table H8 Overview of crack widths at grid line 5 for \( F=0 \) kN.
H3.2 With tensile force \((F=3.04 \text{ kN})\) at the ends of the floor

The calculation of the longitudinal forces at grid line 5 has been based on the calculated values assembled in Table H4. With the aid of linear interpolation from these data the longitudinal forces have been calculated for a total horizontal load of respectively: \(Q=6, 12, 18, 24\) and \(30 \text{ kN}\). These values are assembled in Table H9.

<table>
<thead>
<tr>
<th>(Q) (kN)</th>
<th>(M_s) (kNm)</th>
<th>(N_{s1}) (N)</th>
<th>(N_{s2}) (N)</th>
<th>(N_{s3}) (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>6.48</td>
<td>2717</td>
<td>1368</td>
<td>828</td>
</tr>
<tr>
<td>12</td>
<td>12.96</td>
<td>4532</td>
<td>2167</td>
<td>1221</td>
</tr>
<tr>
<td>18</td>
<td>19.44</td>
<td>6346</td>
<td>2965</td>
<td>1614</td>
</tr>
<tr>
<td>24</td>
<td>25.92</td>
<td>8160</td>
<td>3764</td>
<td>2006</td>
</tr>
<tr>
<td>30</td>
<td>32.40</td>
<td>9970</td>
<td>4562</td>
<td>2399</td>
</tr>
</tbody>
</table>

Table H9 Overview of calculated longitudinal forces at grid line 5 for \(F=3.04 \text{ kN}\).

The crack widths can be calculated as described in Section H2.2 and are assembled in Table H10.

| meas. point | crack width (mm) for \(Q\) (kN) |
|---|---|---|---|---|---|
| 5-DC | 0.067 | 0.112 | 0.157 | 0.202 | 0.247 |
| 5-CD | 0.036 | 0.057 | 0.078 | 0.100 | 0.121 |
| 5-CB | 0.033 | 0.052 | 0.071 | 0.091 | 0.110 |
| 5-BC | 0.022 | 0.033 | 0.044 | 0.055 | 0.066 |
| 5-BA | 0.020 | 0.028 | 0.037 | 0.046 | 0.055 |

Table H10 Overview of crack widths at grid line 5 for \(F=3.04 \text{ kN}\).
APPENDIX I

I. TEST LOADING ON FLOOR UNITS

In order to determine the material properties of the floor units (hollow core slabs) loaded within the plane of the floor and at right angles to the cavities, a number of these units were tested to failure. The failure load and deformation of the specimens were measured.

II. Shape and dimensions of the test specimens.

In all, three specimens were investigated. One of these consisted of a 200mmx240mmx40mm unit as shown in Fig. II. The two others each consisted of two units, with these same dimensions, bonded together by glueing, as shown in Fig. II.

![Fig. II. Shape and dimensions of test specimens.](image)

I2. Testing procedure

The specimens were loaded in a compression testing machine. The deformation in the direction of the force was measured with a mechanical strain gauge with 135mm gauge length. The load and deformation values obtained in the test were plotted with the aid of an X-Y recorder. The failure load reading was provided by the testing machine.
I3. Results of measurements

The results of the load and deformation measurements are represented in Fig. 12 (F-Δl diagrams).

![Diagram showing load F(kN) vs. deformation Δl(mm) for test 1, test 2, and test 3.]

Fig. 12. Relation between load and deformation.

The normal stress for test 1 is: \( \sigma_c = F/40 \times 200 \) (N/mm²)

The normal stress for tests 2 and 3 is: \( \sigma_c = F/80 \times 200 \) (N/mm²)

The modulus of elasticity has been determined from the F-Δl diagrams. More particularly, it has been calculated from a line passing through the origin and through a point at 1/3 \( F_u \) on the curve. The normal stress at failure and the modulus of elasticity calculated in this way are indicated in Table II.

<table>
<thead>
<tr>
<th></th>
<th>test 1</th>
<th>test 2</th>
<th>test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_u ) (kN)</td>
<td>79.8</td>
<td>131.0</td>
<td>163.8</td>
</tr>
<tr>
<td>( \sigma_u ) (N/mm)</td>
<td>9.98</td>
<td>8.19</td>
<td>10.24</td>
</tr>
<tr>
<td>( E_0 ) (N/mm)</td>
<td>13900</td>
<td>12840</td>
<td>16270</td>
</tr>
</tbody>
</table>

Table II Overview of results of measurements.
The average values obtained from these three tests have been adopted in the calculations:

$\sigma_u = 9.5 \ \text{N/mm}^2$

$E_o = 14300 \ \text{N/mm}^2$
APPENDIX K

K. INFLUENCE OF THE BOUNDARY CONDITIONS AT THE ENDS

For judging the calculated and the measured results for the various behaviour patterns of the floor bay it is necessary to know what effect the boundary conditions at the ends of the bay have upon them. By "boundary conditions" is here understood the manner in which the ends of the bay are connected to the simulated shear walls extending along the short sides thereof. The influence that the boundary conditions exercise is represented in four diagrams, for the following behaviour patterns:

a. the deflection of the longitudinal edge at point D7 (see Fig. K1);
b. the tensile force in the connection between the beams at point D7 (see Fig. K2);
c. the tensile force in the connection between the beams at D2 (see Fig. K3);
d. the average deformation of the short side of the floor bay at the grid lines 1 and 14 (see Fig. K4).

In these diagrams the variants of the manner of connection between floor and shear wall are indicated along the horizontal axis. In the tests reported here this connection was varied in ten ways. The measured values of, respectively, the deflection at D7, the tensile force at D7 and D2, and the average deformation at grid lines 1 and 14 are marked on the vertical axis in these diagrams. The measured values have here been plotted for a total horizontal load Q=6kN.

In Figs. K1, K2 and K3 the measured values for tests which were performed under otherwise similar conditions are connected to one another. There are three sets of tests considered here:

a. tests on the floor bay without tensile force at the ends and without a preformed crack (designated as: uncracked; F=0kN);
b. tests on the floor bay without tensile force at the ends and with a preformed crack at grid line 2 (designated as: cracked; F=0kN);
c. tests on the floor bay with a total tensile force of 3.04kN at the ends and with a preformed crack at grid line 2 (designated as: cracked; F=3.04kN).

In Fig. K4 only the measured values per longitudinal grid line have been connected to one another for tests performed on the floor bay without tensile force at the ends and without a preformed crack.

![Diagram](image-url)  
Fig. K1 Influence of the boundary conditions at the ends on the deflection at D7.
tensile force at D7 (kN) for Q = 6 kN

Fig. K2 Influence of the boundary conditions at the ends on the tensile force at D7.

Fig. K3 Influence of the boundary conditions at the ends on the tensile force at D2.
deformation at the ends of the floor (mm) for $Q = 6\, \text{kN}$

Fig. K4 Influence of the boundary conditions at the ends on the average deformation at the ends of the floor.
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