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TNO report

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LEWIS structural covering floor

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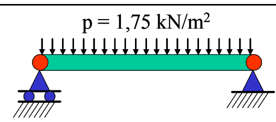
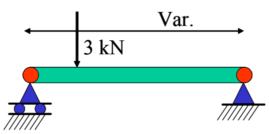
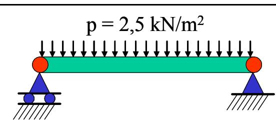
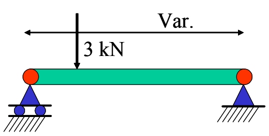
Management additional

Extensive research into the load bearing capacity (resistance) of the LEWIS structural covering floor, carried out in the Netherlands and Germany, has resulted in the load table below that, based upon conformity with the principles of the Dutch Building Decree, can be applied in the Netherlands. The table covers the majority of applications.

		Theoretical span						
Application	Uniformly distributed allowable floor load p_{rep} and concentrated floor load F_{rep} in accordance with TGB 1990 NEN 6702	Floor thick-ness	600 mm	900 mm	1200 mm	1500 mm	2000 mm	2500 mm
houses and overnight accommo-dation		50			ϕ5-150 at free edge	ϕ5-150 at free edge		
		75					ϕ5-150 whole floor	ϕ5-150 whole floor
offices, educational establish-ments and health care buildings		50			ϕ5-150 at free edge	ϕ5-150 at free edge		
		75					ϕ5-150 whole floor	ϕ5-150 whole floor

Higher variable loads are possible. Concentrated loads can vary from a maximum of 4 kN near the support to 10 kN at mid span. Additional advice can be provided, aided by the accompanying TNO report, for floors to which reinforcement must be introduced or where higher loads are demanded than shown in the table above. As well as information about the load bearing capacity of the floor, the report also includes practical recommendations for the serviceability limit stage (deflections, cracking formation).
See further:

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Contents

1 INTRODUCTION

LEWIS profiled steel sheets are hot dipped galvanised re-entrant thin walled steel sheets. The sheets are self-bearing and are used for shuttering and reinforcement of relatively thin concrete floors on a light, usually wooden support structure. A cross-section view of the LEWIS steel sheet is shown in Figure 1 [1].

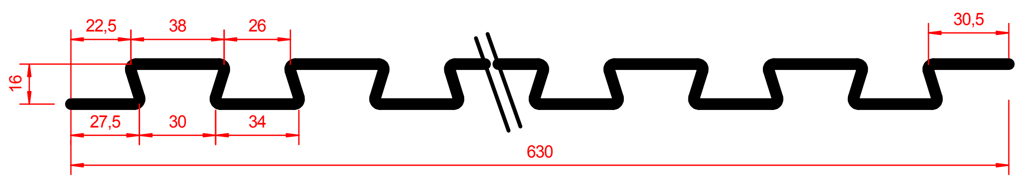


Figure 1: Cross-section of the LEWIS steel sheet.

The LEWIS profiled steel sheet has been applied in building structures for many years. At first the sheet was used as shuttering for covering floors on wooden joists. The span was limited, usually around 600 mm.
The load bearing capacity of the original profiled steel sheet was investigated for the construction stage [3] respectively the final stage [4] in 1990 and in 1991. Evaluation of the results of the investigation is reported in [5]. During the construction stage, the concrete does not reach sufficient strength to be included in the calculations of the load bearing capacity. This is why only the resistance of the steel sheet is included in the calculations in the construction stage. The profiled steel sheet and the covering concrete layer work together in the final stage. The steel-concrete floor can be considered, in regard to its structural behaviour, as an extremely low steel-concrete composite slab interaction between steel sheet and concrete floor is realised by containment of the concrete in the re-entrant profiled steel sheet. Because of this the floor falls within the scope of NEN-EN 1994-1-1:2004 (see Eurocode 4 art. 9.1.1 [15]), even though the floor does not comply with the requirements for minimal floor thickness and minimal steel core thickness (see Eurocode 4 art. 9.2.1 [15]).
The LEWIS profiled steel sheet was improved in 1995, with the aim of optimizing the interaction between steel sheet and the covering concrete layer in the final stage. Experimental research [7] has shown that the geometry of the re-entrant profiled sheet has great influence on the ultimate failure mode of the floor in the final stage. With regard to the safety of the floor, a ductile failure mode is preferable to a brittle failure mode. The new sheet geometry is such that this ductile failure mode is realised for the steel-concrete composite floor structure.
Since 1995 tests have been carried out on the improved LEWIS floor to determine the load bearing capacity in the final stage. Experimental research has been carried out for the greatest part by the University of Kaiserslautern [5]. The results of this extensive research have led to an ‘Allgemeine Bauaufsichtliche Zulassung’ (General Building Inspection Approval [2]. The Zulassung (Approval) has been set up by the Deutsche Institut für Bautechnik (German Institute for Building Technology).

The research carried out and the design model used in the various reports (see [8] to [13]) have been evaluated in a previously published TNO report [20]. The reason for the evaluation being that the results of the research were transcribed to a table that could be used in the Dutch construction practice. This means that the floor must comply with the requirements dictated by the Dutch Building Decree in regard to safety and usability. The floor must comply with specific requirements during the construction stage as well as during the final stage.

The report at hand is an update of report [20].
In the new version of NEN-EN 1994-1-1:2004 [15] various alterations have been made as a result of which, amongst other things, the vertical shear capacity must be evaluated differently than previously. The specific load situation, where a concentrated load is placed at a random position on the floor, is also included. A point load near a support especially influences the evaluation in regard to the vertical shear load.
Attention is also paid in this report to the serviceability of the floor, with particular attention paid to the bending stiffness and crack formation in the floor.

Based upon the European pre-standard of Eurocode 4 and the accompanying National Application Document (NAD), conformity with the legal requirements in the Dutch Building Decree may be proven for steel-concrete composite structures. TGB 1990 Loads and deformations (NEN 6702) is referred to for the loads.

In future, European standards will be replaced and reference will be made to EN 1991 in regard to the loads.

In this document conformity to the legal requirements in the Dutch Building Decree are proven based upon the European pre-standard and the accompanying application document. The articles in the standard text of Eurocode 4 (NEN-EN 1994-1-1:2004) will be referred to. Mention will be made when the specifications deviate from the pre-standard and reference will be made to the articles concerned in the pre-standard (ENV 1994-1-1:1994).

2 LEWIS FLOOR IN THE CONSTRUCTION STAGE

2.1 Loads

Loads and load combinations according to Eurocode 1 [14] have been used in the investigation carried out (see Chapter 2 of [20]). For the Dutch market loads and load combinations in accordance with NEN 6702 [16] are applicable. These will be dealt with later.

2.1.1 Permanent load

2.1.1.1 Self weight of the floor (NEN 6702, art. 7.1)

The self weight is determined per square metre of floor surface. The standard width of the individual LEWIS sheet is 630 mm; the working width is 610 mm.

Profiled steel sheet	: $p_{eg;rep}$	= 0.06 kN/m^2
Composite floor (incl. steel sheet)	: $p_{eg;rep}$	= 1.06 kN/m^2 (total height 50 mm) :
	: $p_{eg;rep}$	= 1.66 kN/m^2 (total height 75 mm)

2.1.1.2 Ponding (NEN-EN 1994-1-1:2004, art. 9.3.2)

If the LEWIS sheet bends substantially during the construction stage, the effect of an accumulation of liquid concrete must be taken into account . The effect is negligible if the steel sheet bends no more than 1/10 of the floor thickness. In Chapter 3.2.2. the previously set up conditions in ENV 1994-1-1 art.7.3.2.1 and the accompanying National Application Document (NAD) for deflections are checked, namely ≤ 20 mm or ≤ 0.004 times the span.

2.1.2 Variable load

2.1.2.1 Construction load (NEN-EN 1994-1-1:2004, art.9.3.2)

EN 1991-1-6, art.4.11.2 in Eurocode 4, is referred to for the construction load. This document is based upon the specifications in ENV 1994-1-1 art.7.3.2.1 and the accompanying NAD. The span of the LEWIS structural covering floor is a maximum of 2500 mm in this field of research. This means that over the whole of the surface a characteristic uniformly distributed construction load of 15 kN/m^2 must be taken account of.

2.2 Limit stages

2.2.1 Ultimate limit stage

The structural safety of the LEWIS sheet must be checked for the following possible ultimate limit stage conditions:

- positive (sagging) bending moment (at mid span),
- combination of negative (hogging) bending moment and vertical shear (at intermediate support),
- vertical shear (at end support).

The strength and stiffness of the LEWIS sheet have been experimentally researched [3]. The load capacity for the ultimate limit stage conditions mentioned can be derived from

[5], in which the results of the investigations are evaluated. The load capacity can also be calculated based upon Eurocode 3 (EN 1993-1-3).

2.2.2 *Serviceability limit stage*

During the construction stage, the serviceability limit condition does not need to be evaluated in accordance to 6.3.2 of NEN 6720. The European pre-standard and NEN-EN 1994-1-1:2004 differ in this from NEN 6702. The ENV 1994-1-1, art.7.5.2 and the accompanying NAD state that the maximum deflection of the steel sheet as a result of its self weight (the LEWIS sheet and the concrete) must amount to no more than 20 mm or L/180. L is the distance between the studs if the floor is temporary supported during construction.
The Reppel B.V. brochure [1] also states that requirements are made in regard to the use of the floor during the construction. At a span of 1200 mm or more, temporary studs are installed. The maximum deflection is therefore limited to 1/150 of the span.
In the Netherlands, no demands are made for walking on the floor during the construction. However, this is the case in Germany, see [3]. It is assumed that walking on the sheet is in order if the deflection demands of 1/150 of the span are complied with.

2.3 Floor without temporary studs

The test is carried out for a total floor thickness of 50 mm and 75 mm respectively.
For code checking the ultimate limit stage the following apply:

$p_{d,50} = 1.35 * 1.06 + 1.5 * 1.5 = 3.7\ kN/m^2$
$p_{d,75} = 1.35 * 1.66 + 1.5 * 1.5 = 4.5\ kN/m^2$

For code checking the serviceability limit stage the following apply:

$p_{d,50} = 1.06\ kN/m^2$
$p_{d,75} = 1.66\ kN/m^2$

The maximum occurring moment, the maximum vertical shear load and the maximum deflection of a floor not temporary supported during construction are shown in Table 2 and Table 3. For the stiffness of the sheet a moment of inertia is used of $I = 3.6\ cm^4/m$ (see also [5]).

Span [mm]	Bending moment [kNm/m]	Vertical shear load [kN/m]	Deflection [mm]
600	0.17	1.11	0.2
900	0.37	1.67	1.2
1200	0.67	2.22	3.8
1500	1.04	2.78	9.2
2000	1.85	3.70	29.2
2500	2.89	4.63	71.3

Table 2 Moment, vertical shear load and deflection of a floor not temporary supported during construction (total thickness of floor 50 mm).

Span [mm]	Bending moment [kNm/m]	Vertical shear load [kN/m]	Deflection [mm]
600	0.20	1.35	0.4
900	0.46	2.03	1.9
1200	0.81	2.70	5.9
1500	1.27	3.38	14.5
2000	2.25	4.50	45.7
2500	3.51	5.63	111.7

Table 3 Moment, vertical shear load and deflection of a floor not temporary supported during construction (total thickness of floor 75 mm).

According to [5] the moment capacity amounts to 1.17 kNm/m and the vertical shear load is 22 kN/m at an end support. The maximum deflection is limited to 1/150 of the span. The conclusion can be made that the floor should be temporary supported during construction for spans greater that 1500 mm (at a total floor thickness of 50 mm) respectively greater than 1200 mm (at a total floor thickness of 75 mm). One stud is still sufficient. The maximum deflection is always normative. According to the work processing advices in the Reppel b.v. brochure [1] temporary supports are required for spans of more than 900 to 1000 mm. This is in any case on the safe side.

2.4 Two span continuous floor

The check is carried out for a total floor thickness of 50 mm and 75 mm respectively. The force distribution is determined on the basis of the theory of elasticity. The same loads are applicable as stated in paragraph 2.3.
In the ultimate limit stage, the combination of negative bending (hogging) moment and support reaction at the intermediate support is normative. The maximum deflection is located at 0.42 times the span length.
The hogging moment, the support reaction and the maximum deflection in a not temporary supported floor are shown in Table 4 and Table 5. For the stiffness of the sheet, a moment of inertia is again used of $I = 3.6\ cm^4/m$ (see also [5]).

Span [mm]	Bending moment [kNm/m]	Support reaction [kN/m]	Deflection [mm]
2 x 600	0.17	2.78	0.1
2 x 900	0.37	4.16	0.5
2 x 1200	0.67	5.55	1.6
2 x 1500	1.04	6.93	3.8
2 x 2000	1.85	9.25	12.2
2 x 2500	2.89	11.56	29.7

Table 4 Moment, support reaction and deflection in a floor not temporary supported during construction (total thickness of floor 50 mm).

Span [mm]	Bending moment [kNm/m]	Support reaction [kN/m]	Deflection [mm]
2 x 600	0.20	3.38	0.2
2 x 900	0.46	5.06	0.8
2 x 1200	0.81	6.75	2.5
2 x 1500	1.27	8.44	6.0
2 x 2000	2.25	11.25	19.0
2 x 2500	3.51	14.06	46.5

Table 5 Moment, support reaction and deflection in afloor not temporary supported during construction (total thickness of floor 75 mm).

According to [5] the hogging moment capacity amounts to 1.17 kNm/m and the support capacity at an intermediate support amounts to 44 kN/m. For support reactions less than 8.3 kN/m the hogging moment capacity does not have to be reduced. For support reactions greater than 8.3 kN/m a design graph is included in [5]. The maximum deflection is limited to 1/150 of the span length.
A conclusion can be made that in this case on spans of more than 1500 mm (with a total floor thickness of 50 mm) and on spans of more than 1200 mm (with a total floor thickness of 75 mm)the floor must be temporary supported during construction. For a continuous floor the maximum deflection is no longer normative. In that case, the force distribution at the intermediate support is critical.

3 LEWIS STRUCTURAL COVERING FLOOR IN THE FINAL STAGE

3.1 Loads

In the investigation carried out (see Chapter 2 of [20]) loads and combinations are used in accordance with Eurocode 1 [14]. For the Dutch market the loads and combinations in accordance with NEN 6702 [16] are applicable. These will be dealt with later.

3.1.1 Permanent load

3.1.1.1 Floor's self weight (NEN 6702-7.1)

The self weight is determined per square metre of floor surface. The standard width of the individual LEWIS sheet is 630 mm; the cover width is 610 mm.
If temporary studs are not used during assembly, the floor's self weight does not act on the composite floor, but on the steel sheet itself. This is favourable when the structural interaction between steel and concrete is critical in the ultimate limit stage. If the steel sheet is fully supported during assembly, the floor's self weight acts entirely on the composite structure. This is unfavourable for the composite floor. The last situation is assumed for further evaluation because safe results are obtained with this in every case. In reality, temporary studs will be used in a number of cases resulting in only a small part of the self weight acting on the composite floor structure.

Profiled steel sheet	: $p_{eg;rep}$	$= 0.06$ kN/m^2
Composite floor (incl. steel sheet)	: $p_{eg;rep}$	$= 1.06$ kN/m^2 (total height 50 mm)
	: $p_{eg;rep}$	$= 1.66$ kN/m^2 (total height 75 mm)

3.1.1.2 Non-load bearing inner walls (NEN 6702-7.1.3.2)

If the floor is applied in buildings of category a (houses etc.) or category b (offices, educational establishments, health care buildings, apartments etc.), account must be taken of an uniformity distributed load as a consequence of non-load bearing inner walls. Next the load that must be taken into account for a free high wall of 2.5 metres is given. If the actual height differs, the load must be adjusted in accordance with 7.1.3.2 of NEN 6702.

Self weight inner wall ≤ 40 kg/m^2	: $p_{eg;rep} = 0.5$ kN/m^2
Self weight inner wall ≤ 80 kg/m^2	: $p_{eg;rep} = 0.8$ kN/m^2
Self weight inner wall ≤ 120 kg/m^2	: $p_{eg;rep} = 1.2$ kN/m^2

3.1.2 Variable load

3.1.2.1 Uniformly distributed floor load (NEN 6702-8.2.2.1)

For a safe approach to the load bearing capacity of the LEWIS floor, a maximum floor load of $p_{rep} = 2.5$ kN/m^2 will be considered. With this the LEWIS floor can be applied to buildings in various categories:

a) houses, residential caravans, accommodation, outside storage, garages;

b) offices, educational establishments, health care buildings, apartments, accommodation.

Higher floor loads are possible. Reppel B.V. can give advice about this. Depending on the application, provisions must be made with the following categories of buildings (see also appendix A3/3):

c) sales areas in shop buildings
d) station buildings, hotel and catering buildings, assembly buildings, sport accommodations;
e) industrial buildings (if p_{rep} no greater than 5 kN/m^2)

3.1.2.2 *Concentrated load (NEN 6702-8.2.2.1)*

The concentrated load depends on the category of the building. The load must be placed on a surface of 0.1 x 0.1m^2, except in category a, for which a surface of 0.5 x 0.5 m^2 can be used. Within the same load case, no combinations of uniformly distributed floor load and concentrated loads need be considered. The extreme values for the concentrated loads are:

- category a and b : F_{rep} = 3 kN
- category c and d : F_{rep} = 7 kN
- category e : F_{rep} = 10 kN (or higher)

3.1.2.3 *Line load (NEN 6702-8.2.2.2)*

With free edges, such as stairways and balconies, a vertical line load of q_{rep} = 5 kN/m over a length of 1 metre must be placed at a distance of a maximum of 0.1 metre from the edge. Within the same load case, no combinations of uniformly distributed floor load and concentrated loads need be considered.

3.2 Boundary conditions

In the construction stage, a distinction can be made between the ultimate limit stage and the serviceability limit stage.

3.2.1 *Ultimate limit stage*

The structural safety is tested for the following ultimate limit stages:

- bending (plastic moment),
- horizontal shear (longitudinal shear force),
- vertical shear (vertical shear load),
- punching.

For determining the extreme capacity of the floor, use is made of the European pre-standard from Eurocode 4 [15] together with the NAD for this pre-standard. This is because there is no separate standard for the calculation of steel-concrete composite structures available in the Netherlands.

Rules for the calculation of the four ultimate limit stages mentioned are given in 9.7 of NEN-EN 1994-1-1:2004. In the experimental research (see Chapter 2 of [20]) a fifth failure mode was observed, namely bending in transverse direction due to a relatively high concentrated load near the free edge of a not additionally reinforced floor. There are no design rules available with which the resistance in regard to transverse bending can be calculated simply. This brittle failure mode must therefore be avoided by using sufficient reinforcements in the floor (see also [13]).

3.2.2 *Serviceability limit stage*

The usability of the floor is checked for the following requirements:

- additional deflection (due to variable loads and long term effects),
- sagging in the final stage,
- vibrations.

The maximum additional deflection u_{bij} is 0.003 times the span l_{rep} (NEN 6701-10.2.1). If the floor bears partition walls made from stone, the maximum additional deflection u_{bij} is 0.002 times the span l_{rep} (NEN 6702-10.2.2). The maximum sagging in the final stage u_{eind} amounts to 0.004 times the span l_{rep} (NEN 6702-10.4.1).
To prevent the floor vibrating, the first natural frequency of the floor must not be lower than 3 Hz. With a floor upon which dancing or jumping will take place, the natural frequency must not be lower than 5 Hz (NEN 6702-10.5.2). The first natural frequency of the floor can be determined in accordance with appendix A.4 of NEN 6702. It appears from the following calculation that the vibration requirements of the LEWIS structural covering floor are always satisfied because the spans are relatively limited (see also Table 6).

$$f_e = \sqrt{(a/\delta)}$$

where: a = 0.315 m/s^2 (see figure 35 in NEN 6702)
and δ = largest deflection in the momentary load combinations in metres (is maximum 0.004 l_{rep})

Span	Maximum deflection	First natural frequency
600 mm	0.0024 m	11.5 Hz
900 mm	0.0036 m	9.4 Hz
1500 mm	0.0060 m	7.2 Hz
2000 mm	0.0080 m	6.3 Hz
2500 mm	0.0100 m	5.6 Hz

Table 6 Lower limit for first natural frequency with various spans.

3.3 Single span floor in the ultimate limit stage

The largest part of the experimental research carried out (see Chapter 2 of [20]) was concerned with LEWIS structural covering floors with a total thickness of 50 mm and a maximum span of 1500 mm. This will also be an important area of application for the Dutch market. The floors can be regarded as hinged, simply supported slabs.
In general the LEWIS structural covering floors are not (additionally) reinforced. Prior research has actually shown that reinforcement is needed for the resistance to a relatively high concentrated load. This is to prevent for collapse of the floor due to transverse bending moments. This failure mode has a particularly brittle character. At the moment there is no design model available with which this failure mode can be calculated.
The tests are only carried out for a total floor thickness of 50 mm. The ultimate capacity for the longitudinal shear load, as calculated in [8] and explained in paragraph 2.2, is also based on specimens with a thickness of 50 mm. Previous research [5] has shown that the load bearing capacity of a LEWIS structural covering floor with a thickness of 75 mm is slightly higher than one with a thickness of 50 mm. The capacity of the 75

mm thick floor will also be higher than the 50 mm thick floor in other failure modes (bending, shearing and punching).
The load combinations are formulated for floors in the building categories a and b (in which an uniformly distributed load, as a consequence of non-load bearing inner walls, must be installed) and floors in building categories c, d and e (in which the concentrated load is higher than 3 kN and application is possible with advice from Reppel B.V.).

3.3.1.1 Floors in building categories a and b

The following load combinations are valid for code checking the ultimate limit stages:

1) $p_d = 1.2 * (1.06 + 1.2) + 1.5 * 3.5 = 8.0$ kN/m^2
   $q_d = 0$
   $F_d = 0$
2) $p_d = 1.2 * (1.06 + 1.2) = 2.7$ kN/m^2
   $q_d = 1.5 * 5.0 = 7.5$ kN/m (only at free edges)
   $F_d = 0$
3) $p_d = 1.2 * (1.06 + 1.2) = 2.7$ kN/m^2
   $q_d = 0$
   $F_d = 1.5 * 3 = 4.5$ kN

For code checking the serviceability limit stage the following are valid:

4) $p_d = 1.06 + 1.2 + 3.5 = 5.8$ kN/m^2
   $q_d = 0$
   $F_d = 0$
5) $p_d = 1.06 + 1.2 = 2.3$ kN/m^2
   $q_d = 5.0$ kN/m (only at free edges)
   $F_d = 0$
6) $p_d = 1.06 + 1.2 = 2.3$ kN/m^2
   $q_d = 0$
   $F_d = 3$ kN

3.3.1.2 Floors in building categories c and d (application with advice from Reppel B.V.)

The following load combinations are valid for code checking in ultimate limit stages:

7) $p_d = 1.2 * 1.06 + 1.5 * 5.0 = 8.8$ kN/m^2
   $q_d = 0$
   $F_d = 0$
8) $p_d = 1.2 * 1.06 = 1.3$ kN/m^2
   $q_d = 1.5 * 5.0 = 7.5$ kN/m (only at free edges)
   $F_d = 0$
9) $p_d = 1.2 * 1.06 = 1.3$ kN/m^2
   $q_d = 0$
   $F_d = 1.5 * 7 = 10.5$ kN

For code checking the serviceability limit stage the following are valid:

10) $p_d = 1.06 + 5.0 = 6.1$ kN/m^2
    $q_d = 0$
    $F_d = 0$
11) $p_d = 1.1$ kN/m^2
    $q_d = 5.0$ kN/m (only at free edges)
    $F_d = 0$
12) $p_d = 1.1$ kN/m^2
    $q_d = 0$

F_d = 7 kN

3.3.1.3 *Floors in building category e (application with advice from Reppel B.V.)*

The following load combinations are only valid for industrial floors with a floor load of a maximum of 5.0 kN/m^2 and a concentrated load of a maximum of 10 kN. If one of these loads should be higher then the floor falls outside the application area of this report.

The following load combinations are valid for code checking in ultimate limit stages:

13) p_d = 1.2 * 1.06 + 1.5 * 5.0 = 8.8 kN/m^2
q_d = 0
F_d = 0

14) p_d = 1.2 * 1.06 = 1.3 kN/m^2
q_d = 1.5 * 5.0 = 7.5 kN/m (only at free edges)
F_d = 0

15) p_d = 1.2 * 1.06 = 1.3 kN/m^2
q_d = 0
F_d = 1.5 * 10 = 15 kN

For code checking the serviceability limit stage the following are valid:

16) p_d = 1.06 + 5.0 = 6.1 kN/m^2
q_d = 0
F_d = 0

17) p_d = 1.1 kN/m^2
q_d = 5.0 kN/m (only at free edges)
F_d = 0

18) p_d = 1.1 kN/m^2
q_d = 0
F_d = 10 kN

3.3.2 *Plastic moment*

The positive bending moment capacity can be determined in accordance with 9.7.2 of NEN-EN 1994-1-1:2004. The capacity is determined per metre of floor width. The capacity is also determined per effective width, i.e. the width over which line loads and point loads are distributed. The effective width of a not additionally reinforced floor is, according to 9.4.3 of NEN-EN 1994-1-1:2004, only 168 mm at a load width of 100 mm. This low value is the consequence of the limited floor thickness and the absence of transverse reinforcement in the floor. Strain measurements in the tests (see [10]) have shown that the effective width for both point loads and line loads are in fact significantly higher. The continuous cracks at the ends also showed that the effective width was a minimum of 475 mm at a load width of 50 mm and a span of 1500 mm. A load width of 100mm may be used with loads, in accordance with NEN 6702. The effective width is therefore determined at 500 mm. For smaller spans the actual effective width will be slightly less and for larger spans the effective width will be greater than 500 mm.

Applicable:
A_p = 753 mm^2/m
f_{yp} = 320 N/mm^2
γ_{ap} = 1.0 (see NAD in ENV1994-1-1:1994)
N_{cf} = 241kN

f_{ck} = 18 N/mm^2 (characteristic compressive strength for concrete grade B25)
0.8 f_{ck} = 14.4 N/mm^2 (stress block with composite floor)
γ_c = 1.2 (zie NEN 6720)
b = 1000 mm
x = 20.1 mm
d_p = 41.7 mm
$M_{p,Rd}$ = 7.6 kNm/m
= 3.8 kNm per 500 mm width

The maximum moment appears at load combination 15, where a very high point load of 15 kN must be taken into account (see Table 7). For the uniformly distributed load, the whole width may be taken into account; for the point load only the effective width of 500 mm. The floor does not conform to spans above 1200 mm. LEWIS structural floor coverings with a thickness of 50 mm and without any additional reinforcement can only be applied in industrial buildings (category e) for spans up to 900 mm.
Floors without additional reinforcement conform for all spans for the load combinations for category a and b. In principle, it is not combination 3 (with a concentrated distributed load of 4.5 kN) that is indicative, but combination 1 (with an uniformly distributed load of 8 kN/m^2).
With the load combination for category c and d (with a concentrated load of 10.5 kN) the floor conforms for spans up to 1200 mm. For greater spans, the floor in this application area will have to be additionally reinforced. It is also possible to increase the height of the floor.

	Bending moment [kNm/m]		
Span [mm]	Combination 3	Combination 9	Combination 15
600	0.1 + 0.7	0.1 + 1.6	0.1 + 2.3
900	0.3 + 1.0	0.1 + 2.4	0.1 + 3.4
1200	0.5 + 1.4	0.2 + 3.2	0.2 + 4.5
1500	0.8 + 1.7	0.4 + 3.9	0.4 + 5.6
2000	1.4 + 2.3	0.7 + 5.3	0.7 + 7.5
2500	2.1 + 2.8	1.0 + 6.6	1.0 + 9.4

Table 7 Maximum bending moments occurring.

If sufficient transverse reinforcement has been installed, the effective width, according to 9.4.3 of Eurocode, is equal to 168 mm plus ½ times the span for a concentrated load at mid span. The effective width varies in this from 468 mm (for a span of 600 mm) to 1418 mm (for a span of 2500 mm), see Table 8. The floor conforms in that case for all combinations.
If the concentrated load can be found near a free edge (i.e. a non-supporting edge), the effective width must be adapted accordingly. The effective width varies from 284 mm (for a span of 600 mm) to 759 mm (for a span of 2500 mm), see also Table 8. The floor in that case can only be applied in:
- building categories a and b : span 600 mm up to 2500 mm

And with advice from Reppel B.V. in:
- building categories c and d : span 600 mm up to 1200 mm;
- building category e : no span at all.

	Floor area		Free edge	
Span [mm]	b_{em} [mm]	$M_{p,Rd}$ [kNm]	b_{em} [mm]	$M_{p,Rd}$ [kNm]
600	468	3.6	284	2.2
900	618	4.7	359	2.7
1200	768	5.8	434	3.3
1500	918	7.0	509	3.9
2000	1068	8.1	634	4.8
2500	1418	10.8	759	5.8

Table 8 Effective width and moment capacity of LEWIS structural covering floor without additional reinforcement.

3.3.3 *Longitudinal shear force*

The capacity for horizontal shear can be determined in accordance with 9.7.3 of NEN-EN 1994-1-1:2004. Use is made of the m & k method in this report. In the research reports [8] up to [13] use is made of the 'partial connection' method in accordance with Annex E from ENV 1994-1-1:1994. Both methods are permitted in principle. In NEN-EN 1994-1-1:2004 this method is actually adapted and a friction coefficient of μ=0,5 introduced. The shear strength accompanying this design model can be calculated with the available test results.

A_p = 753 mm^2/m
m_k = 66.5 N/mm^2
k_k = 0.255 N/mm^2
γ_{vs} = 1.25
b = 1000 mm
d_p = 41.7 mm
L_s = shear length (depending on span and load situation)
$V_{l,Rd}$ = $1.67/L_s + 8.51$ kN/m (with L_s in metres)
= $0.84/L_s + 4.16$ kN (per 500 mm width)

For spans up to 2 metre the maximum support reaction with load combination 15 (see Table 9) comes into operation. With spans above 2 metres, the combinations 7 and 13 provide the maximum support reaction. But still the combination 15 is indicative here, because the point load must be distributed across a more limited width. The load consists of a large concentrated load in combination with a limited uniformly distributed load. This means that the shear length L_s is almost equal to half the span (see 9.4.3 of [15]).

	Support reaction [kNm/m]		
Span [mm]	Combination 3	Combination 9	Combination 15
600	0.8 + 2.25	0.4 + 5.25	0.4 + 7.5
900	1.2 + 2.25	0.6 + 5.25	0.6 + 7.5
1200	1.6 + 2.25	0.8 + 5.25	0.8 + 7.5
1500	2.0 + 2.25	1.0 + 5.25	1.0 + 7.5
2000	2.7 + 2.25	1.3 + 5.25	1.3 + 7.5

2500	3.4 + 2.25	1.6 + 5.25	1.6 + 7.5

Table 9 Maximum support reaction with a point load at mid span.

LEWIS structural covering floors without additional reinforcement cannot be applied for any span if a concentrated load of 15 kN has to be taken into account. Floors without additional reinforcement do not conform either at a point load of 10.5 kN. Only at lower loads (combinations 1 to 3) will the longitudinal shear force be sufficiently small.
The effective width is now equal to 168 mm plus ½ times the span if sufficient transverse reinforcement is applied. With this the effective width again varies from 468 mm (for a span of 600 mm) to 1418 mm (for a span of 2500 mm). The longitudinal shear capacity is not only dependent on the effective width, but also on the shear length. The shear length is dependent on the span and on the load combination (relationship between point load F and uniformly distributed load p).
In Table 10 the longitudinal shear capacity is shown for load combination 15. With smaller spans, the reinforced floor still does not conform because the load must still be distributed over a relatively smaller width.

When the concentrated load is to be found near a free edge of the floor, the effective width must be adapted accordingly. The effective width then varies between 284 mm (for a span of 600 mm) and 759 mm (for a span of 2500 mm). The longitudinal shear capacity is also shown for this case in Table 10. The floor can be applied in building categories a and b for spans of 600 mm up to 2500 mm.
The floor cannot be applied for any span in building categories c, d, and e. That is why advice must be requested from Reppel B.V. for these building categories.

	Floor area			Free edge		
Span [mm]	b_{em} [mm]	L_s [mm]	$V_{l,Rd}$ [kN]	b_{em} [mm]	L_s [mm]	$V_{l,Rd}$ [kN]
600	468	293	6.7	284	293	4.0
900	618	434	7.6	359	434	4.4
1200	768	572	8.8	434	572	5.0
1500	918	707	10.0	509	707	5.5
2000	1068	926	11.0	634	926	6.5
2500	1418	1139	14.1	759	1139	7.6

Table 10 Effective width, shear length and longitudinal shear capacity of a LEWIS structural covering floor without additional reinforcement for load combination 15.

3.3.4 *Vertical shear load*

The vertical shear load capacity can be determined in accordance with 9.7.5 of NEN-EN 1994-1-1:2004. The design rules in the current version of NEN-EN 1994-1-1:2004 [15] differ from the previous version that was used for the calculations in [19] and [20]. In the current version of NEN-EN 1994-1-1:2004 you are immediately directed to 6.2.2. of Eurocode 2.
The vertical shear load capacity of the not additionally reinforced covering floor with a thickness of 50 mm and concrete grade C20/25 (corresponding to B25) can be determined as follows.

A_p = 753 mm^2/m = 18,825 mm^2 per 25 mm
b_0 = 25 mm
d_p = 41.7 mm

ρ = 0.018
$V_{v,Rd}$ = 0.4 * 41.7 = 16.68 kN/m
= 8.34 kN (per 500 mm width)

3.3.4.1 Point load at mid span

The maximum support reaction occurs at load combination 15 for spans of up to 2 metres, (see Table 9). The combinations 7 and 13 provide the maximum support reaction with spans greater than 2 metres and the combination 15 is still indicative here because the point load must once again be distributed over a more limited width. LEWIS structural covering floors without additional reinforcement cannot be applied for spans greater that 1200 mm if a concentrated load of 15 kN must be taken into account. The not additionally reinforced floor does conform for all spans at a point load of 10.5 kN or 4.5 kN if an effective width of 500 mm used.

If sufficient transverse reinforcement is installed, the effective width in accordance with 9.4.3 of NEN-EN 1994-1-1:2004 is equal to 168 mm plus ¼ of the span. The effective width therefore varies between 318 mm (for a span of 600 mm) and 793 mm (for a span of 2500 mm). The reinforced floor does not conform with smaller spans under 1500 mm because the higher point load of 15 kN still has to be distributed over a too limited width. This is also the case in load combination 9. The floor can in that case only be applied in:

- building categories a and b : span 600 mm up to 2500 mm;

And with the advice of Reppel B.V. in:

- building categories c and d : span 900 mm up to and including 2500 mm;
- building category e : span 1500 mm up to and including 2500 mm.

	Floor area		Free edge	
Span [mm]	b_{em} [mm]	$V_{v,Rd}$ [kN]	b_{em} [mm]	$V_{v,Rd}$ [kN]
600	318	5.5	209	3.5
900	393	6.6	246.5	4.1
1200	468	7.8	284	4.7
1500	543	9.1	321.5	5.4
2000	643	10.7	384	6.4
2500	793	13.2	446.5	7.4

Table 11 Effective width and vertical shear load capacity of a LEWIS structural covering floor without additional reinforcement with a point load at mid span.

3.3.4.2 Point load near the end support of the floor.

If the vertical shear load moves to the end support, the load situation in regard to the vertical shear load becomes unfavourable. A greater vertical shear load occurs that also has to be distributed over a limited effective width.

The most extreme situation is checked in this paragraph, namely the vertical shear load at 120 mm from the theoretical end support. According to Eurocode 2 the vertical shear load does not have to be checked up to a length 2d (=83.4 mm) from the furthest edge of the end support. Through another failure mechanism, the capacity over the length is significantly raised and no longer relevant. This rule corresponds to the well-known ‘concrete mechanica rule’ that a raised vertical shear load capacity may be taken into account for a point load that moves within 3h (is approximately 120 mm) of the theoretical support. The maximum vertical shear load for the situation when the point load is to be found 120 mm from the end support is shown in Table 12.

	Support reaction [kNm/m]		
Span [mm]	Combination 3	Combination 9	Combination 15
600	0.8 + 3.6	0.4 + 8.4	0.4 + 12.0
900	1.2 + 3.9	0.6 + 9.1	0.6 + 13.0
1200	1.6 + 4.1	0.8 + 9.5	0.8 + 13.5
1500	2.0 + 4.1	1.0 + 9.7	1.0 + 13.8
2000	2.7 + 4.2	1.3 + 9.9	1.3 + 14.1
2500	3.4 + 4.3	1.6 + 10.0	1.6 + 14.3

Table 12 Maximum support reaction with a point load at 120 mm from the end support.

The effective width is determined on the basis of 9.4.3. of NEN-EN 1994-1-1:2004 and is significantly more limited because the point load can be found very close to the end support. The effective width and the accompanying vertical shear load capacity are shown in Table 13. It is obvious that the point loads of 10.5 kN and 15 kN cannot be included in any span. Also the vertical shear load capacity at greater spans (from 1500 mm) is somewhat exceeded at a point load of 4.5 kN.

	Floor area		$V_{v,Sd}$ / $V_{v,Rd}$	
Span [mm]	b_{em} [mm]	$V_{v,Rd}$ [kN]	Combination 3	Comb. 9 and 15
600	264	4.4	0.87	>> 1
900	272	4.5	0.94	>> 1
1200	276	4.6	0.99	>> 1
1500	278	4.6	1.01	>> 1
2000	281	4.7	1.06	>> 1
2500	282	4.7	1.11	>> 1

Table 13 Effective width and vertical shear load capacity of the LEWIS structural covering floor with a point load at 120 mm from the end support.

The limited theoretical capacity ensures a considerable restriction in the application area of the LEWIS floor. The actual capacity will be higher; however it is difficult to indicate how conservative the approach is in accordance to Eurocode 2. To have some sort of reference, the vertical shear load capacity was once again approached however not assuming the concrete calculation rules this time but based upon consecutively;

a. the vertical shear load capacity of the profiled steel sheet;
b. test with a point load at 150 mm (from [10]);
c. test with a line load at 150 mm (from [10]).

Ad. a.
The vertical shear load capacity at the end support of the LEWIS sheet is 22.0 kN/m (from [5]). At an effective width of a minimum of 264 mm the vertical shear load capacity amounts to at least 5.8 kN. This is in any case significantly more than in Table 13, which demonstrates how conservative the calculations are according to Eurocode 2.

Ad b.
The only test where a load with a force is near the support can be found in [10]. Specimen L16512 is loaded with a weight of 14.56 kN at a distance of 150 mm from the end support. The length of the span was 1500 mm; the total floor thickness was 50 mm. The maximum vertical shear load occurring was therefore 0.9 * 14.56 = 13.1 kN.

The specimen was not loaded to the point of failure because it had to be used later for other experiments. Beside the point load was installed on an area of 50 x 50 mm^2, which is more unfavourable than the 100 x 100 mm^2 required by Dutch standards. Suppose that the one test result has the highest value from a series of three good tests. Then the vertical shear load belonging to the lowest test result is minimal:

$V_{R,min}$ = 0.9/1.1 * 13.1 = 10.7 kN

The characteristic value of the vertical shear load is then:

$V_{R,k}$ = 0.9 * 10.7 = 9.6 kN

Assuming that the concrete is indicative (material factor 1.5) and assuming a brittle failure behaviour, the following can be used as the design value for the strength

$V_{R,d}$ = 9.6 / (1.5 * 1.25) = 5.1 kN

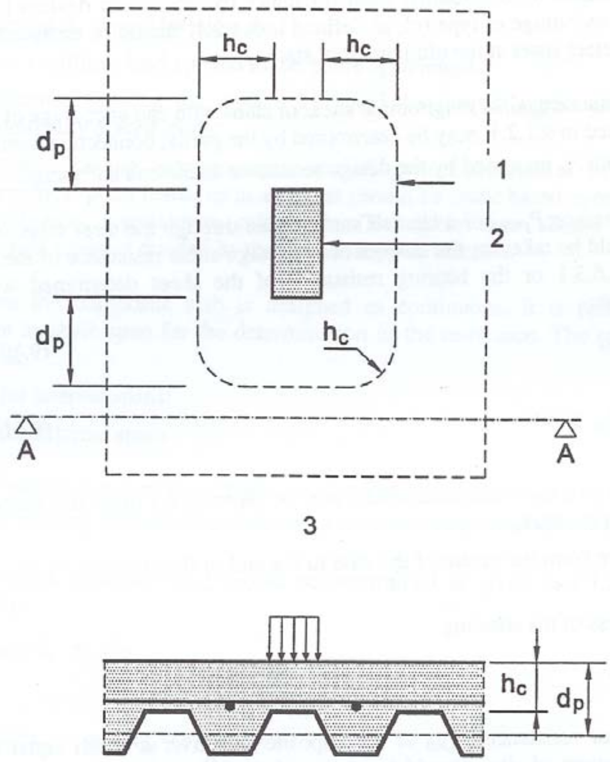
This test result provides a higher capacity than the conservative approach of Eurocode 2 (the calculated 5.1 kN can be compared to 4.6 kN).

Ad c.
The only test in which a line load near the end support was loaded can be found in [10]. Specimen L16512a is loaded with a line load at a distance of 150 mm from the end support. The width of the specimen was 1.22 m. The length of the span was 1500 mm; the total floor thickness was 50 mm. The failure load was 48.76 kN but it must be mentioned that the specimen collapsed from horizontal shear and not from vertical shear load.
The maximum vertical shear load occurring was therefore 0.9 * (48.76/1.22) = 36.0 kN/m.
At an effective width of 278 mm (see Table 13) the maximum vertical shear load is 10.0 kN. This is lower than the 13.1 kN in the other experiment; however a good comparison is not possible because the specimen did not fail because of vertical shear load.

From previous observations, the conclusion can be drawn that a concentrated load of 10.5 kN respectively 15.0 kN cannot be applied for any span if the load must be able to rest near the end support. The vertical shear load capacity is sufficient for a point load of 4.5 kN.

3.3.5 Punching

With floors without additional reinforcement, in principle 9.7.6 of NEN-EN 1994-1-1:2004 can be used for punching. The calculation rule is similar to the one for determining the vertical shear load capacity, however now the shear stresses must be distributed on a surface around the concentrated load (see Figure 14). The indicated perimeter is only valid if there is transverse reinforcement present that ensures sufficient distribution of the load. The surface of the load (100 x 100 mm^2 in accordance with NEN 6702) will be used as a minimum value for the perimeter.



Key
1 Critical perimeter c_p
2 Loaded area
3 Section A-A

Figure 9.8 - Critical perimeter for punching shear

Figure 14. Critical perimeter for punching (from NEN-EN 1994-1-1:2004)

Where:

A_p	=	753 mm^2/m = 18.825 mm^2 per 25 mm
b_0	=	25 mm
d_p	=	41.7 mm
ρ	=	0.018
h_c	=	34 mm
k_v	=	1.6 – d_p (in metres) = 1.56
τ_{Rd}	=	(0.25*1.5)/1.5 = 0.25 N/mm^2 (concrete grade C20/25, value on the careful side)
τ_{Rd}	=	(0.25*1.15) = 0.29 N/mm^2 (concrete grade B25, these values will be used)
C_p	=	644 mm (in the floor area according to [15])
C_p	=	422 mm (on the free edges according to [15])
$V_{p,Rd}$	=	19.0 kN (11.8 kN at a minimum perimeter of 400 mm) in the floor area
$V_{p,Rd}$	=	12.5 kN (8.9 kN at a minimum perimeter of 300 mm) on a free edge

It has been recognised from various tests that in floors without additional reinforcement no failures take place at punchings, but the failures are a result of bending in transverse direction. This failure mode is not included in NEN-EN 1994-1-1:2004 because a floor with transverse reinforcement is used, which ensures that line loads and point loads are distributed over the effective width of the floor.

3.3.5.1 Concentrated load near the free edge

In [12] three tests are described in which the point load was placed on the free edge of a floor without additional reinforcement and spanning 1500 mm. The resistance in this series of experiments was 5.03 kN (see paragraph 2.6 in [20]) with the concrete grade present. When this resistance is converted to concrete grade B25, the resistance amounts to 3.0 kN at a span of 1500 mm. Table 15 shows the resistance at other spans. The concentrated load of 4.5 kN can only be recorded for smaller spans up to 1000 mm. The larger point loads from 10.5 and 15 kN cannot not be recorded for any span. The conclusion can be drawn that with spans of more than 1000 mm reinforcements must always be used unless it is indicated that no free edges occur in the floor construction.

Span [mm]	Maximum load F_d [kN]
600	7.5
900	5.0
1200	3.7
1500	3.0
2000	2.2
2500	1.8

Table 15 Maximum design value for the load for the load on a LEWIS structural covering floor without additional reinforcement for a concentrated load at a free edge.

The amount of reinforcement necessary can be determined with the design model that is derived from [13], see also paragraph 2.7 in [20]. The various test results and the design model appear to be in agreement for the higher concrete grades. The model provides an underestimation for the lower concrete grades (B25, B30), at any rate the test results are significantly higher in this case than the design model indicates. This is in any case on the safe side.
In [13] the scale factor k_v is made equal to 1.0 in the design model. This scale factor takes into account that the average shear force when thicker floors fail is smaller than in floors with a more limited thickness. This factor can also be found in the calculation rules in NEN 6720. In this report the actual value of k_v is used. This is permissible because the model for the lower concrete grade has already provided a very safe value. The perimeter in the model can now definitely be used in accordance with NEN-EN 1994-1-1:2004 after all a reinforced floor is used.

Where:
A_p = 753 mm^2/m = 18.825 mm^2 per 25 mm
b_0 = 25 mm
d_p = 41.7 mm
ρ = 0.018
ρ_1 = reinforcement percentage
k_v = 1.6 – d_p (in metres) = 1.56
τ_{Rd} = (0.25*1.5)/1.5 = 0.25 N/mm^2 (concrete grade C20/25, value on the careful side)
τ_{Rd} = (0.25*1.15) = 0.29 N/mm^2 (concrete grade B25, this value will be used)

C_p = 422 mm (on the free edge in accordance with [15])
$V_{p,Rd}$ = 12.5 (1 + 20.8 ρ_1) kN

The floor conforms even without additional reinforcement for the point loads of 4.5 kN and 10.5 kN. To prevent failures as a consequence of bending in transverse direction, a minimum mesh reinforcement of Ø5-150 (Q131) is used. This reinforcement is not necessary with spans smaller than 1000 mm, (see Table 15) for the point load of 4.5 kN. Reinforcements must be used for point loads of 15 kN to prevent punching. The minimum percentage of reinforcement necessary is approximately 1 %. A mesh reinforcement of Ø8-150 (Q335) could be applied here for example.

3.3.5.2 Concentrated load on the floor area

During the experimental research [10], described in paragraph 2.4 of [20], one of the tests concerned the concentrated load in the middle of the floor area. The span was 1500 mm and the thickness of the floor 50 mm. The failure load amounted to 12.20 kN. If it is assumed that this test result is the highest failure load from a series of three tests, and that this failure load lies 10% above average, a safe approach to the design value of the resistance can be calculated (see also paragraph 2.6 in [20]).
The fictive average failure load amounts to 11.1 kN and the fictive lowest failure load is 10.0 kN. The characteristic resistance, assuming 90 % of the lowest load, is equal to 9.0 kN. With a total safety factor of 2.63 (see also paragraph 2.6 of [20]) the maximum and characteristic point load of 3.4 kN can be recorded or a design value for the load of 5.1 kN. If more tests had been carried out, the results would probably have given a slightly higher resistance.
The resistance of 5.1 kN is valid for concrete grade present for the test. A pressure intensity of 28 N/mm^2 is equal to a grade between C25/30 and C30/37. The average tensile strength according to NEN-EN 1994-1-1:2004 will amount to about 2.8 N/mm^2. At C20/25, which is the same as B25, the average tensile strength is 2.2 N/mm^2. A resistance of 4.0 kN can therefore be used with concrete grade B25. This is slightly higher than the resistance with a concentrated load on a free edge (3.0 kN at a span of 1500 mm). The resistance with other spans is shown in Table 16. The concentrated load of 4.5 kN can only be recorded for small spans up to 1350 mm. The larger point loads of 10.5 and 15 kN cannot be recorded for any span. The conclusion can be made that, in principle, reinforcement must be installed with spans greater than 1350 mm.
Previously conducted tests [6] and the results of the tests [11] have not been included in the evaluation because of the different circumstances. Based on these tests, that in every case resulted in a failure load higher than the fictive lowest failure load of 10.0 kN, the safe value may be raised from 1350 mm to 1500 mm.

Span [mm]	Maximum load F_d [kN]
600	10.0
900	6.7
1200	5.0
1500	4.0
2000	3.0
2500	2.4

Table 16 Maximum design value of the load for a LEWIS structural covering floor without additional reinforcement with concentrated loads in the floor area.

The amount of reinforcement required can also be determined with the design model deduced in [13], see also Paragraph 2.7 of [20].

Where:
A_p = 753 mm^2/m = 18.825 mm^2 per 25 mm
b_0 = 25 mm
d_p = 41.7 mm
ρ = 0.018
ρ_1 = reinforcement percentage
k_v = 1.6 – d_p (in metres) = 1.56
τ_{Rd} = (0.25*1.5)/1.5 = 0.25 N/mm^2 (concrete grade C20/25, value on the safe side)
τ_{Rd} = (0.25*1.15) = 0.29 N/mm^2 (concrete grade B25, this value is stuck to)
C_p = 644 mm (in floor area in accordance with [15])
$V_{p,Rd}$ = 19.0 (1 + 20.8 ρ_1) kN

The floor complies even without additional reinforcement. To prevent failure as a consequence of transverse bending, a minimum reinforcement mesh of Ø5-150 (Q131) is installed. No reinforcement need be installed for a load of 4.5 kN for spans up to 1500 mm.

3.4 Serviceability limit stage

Practical recommendations will be given in this paragraph for the serviceability limit stage.

3.4.1 Deformations

Deformation of the floor within the normal application will not be indicative because of the limited spans.
To calculate the deflections in actual situations, the stiffness of the floor in its final stage must be known. The automatic calculation (MATH CAD) of the bending stiffness is shown in Appendix 2. The calculation of the bending stiffness of the LEWIS floor in the bearing direction of the sheet is carried out in Table 17 for actual concrete qualities. The calculation of the bending stiffness perpendicular to this is shown in Table 18.
A factor of n = 10 for short term loads and n = 20 for long term loads can be used for the deflection calculations. A factor n = 15 is often used in practice as the average value for all loads.
EI = 7.7 kNm^2 can be used for the bending stiffness of the LEWIS sheet (in the load direction of the sheet, during the execution stage). The neutral axis lies at e = 8.3 mm from the under side of the sheet.

	n = 10		n = 15		n = 20		n = 25	
H	E	EI	E	EI	e	EI	e	EI
40	20.4	44.1	19.3	37.5	18.4	33.9	17.7	31.4
50	25.5	78.5	24.2	66.2	23.2	59.8	22.2	55.4
60	30.5	125.3	29.2	105.0	28.0	94.9	26.9	88.1
70	35.6	185.6	34.1	154.2	32.8	139.3	31.7	129.6
80	40.6	260.4	39.1	214.3	37.7	193.3	36.5	180.1

Table 17 Bending stiffness of LEWIS structural covering floor, which is not cracked (in span direction).

	n = 10		n = 15		n = 20		n = 25	
H	E	EI	E	EI	e	EI	e	EI
40	28.0	24.2	28.0	16.1	28.0	12.1	28.0	9.7
50	33.0	68.8	33.0	45.9	33.0	34.4	33.0	27.5
60	38.0	149.1	38.0	99.4	38.0	74.5	38.0	59.6
70	43.0	275.6	43.0	183.7	43.0	137.8	43.0	110.2
80	48.0	458.8	48.0	305.8	48.0	229.4	48.0	183.5

Table 18 Bending stiffness of LEWIS structural covering floor, which is not cracked (perpendicular to span direction, excl. support construction).

The following notations have been used in the tables:

h = thickness of the structural covering floor in mm
e = distance from under side of floor to neutral axis in mm
EI = bending stiffness in kNm^2

E_{steel} = 210000 kNm^2
$E_{concrete}$ = E_{steel} / n

3.4.2 *Crack formation*

3.4.2.1 *General*

The LEWIS structural covering floor is installed in the majority of the applications without additional reinforcement. The tensile forces that occur due to shrinkage, temperature or bending of the concrete can result in cracks. Mostly these cracks are restricted to no wider than 0.1 to 0.2 mm and form no practical problems. Cracks of this size are hardly noticeable.
Larger cracks do not necessarily have to lead directly to practical problems. The durability (in a non-aggressive environment) and the strength are not usually at risk. For example when cracks appear above the supports, the continuous structural covering floor, in the structural sense, is in fact reduced to a simply supported hinged slab. All the resistance tables for the LEWIS structural covering floor are determined on the basis of a similar static scheme. The strength is not at risk because of this. If a ‘dry environment’ is present above the floor, this sort of crack formation forms no problem for the durability of the profiled steel sheet.

In practice cracks can lead to problems when high requirements are demanded aesthetically or for the durability in, for example, an aggressive environment.

Aesthetics:
With a floor that is finished off immediately (ground) or to which a coating will be applied later, the aesthetics are at risk when crack formation results in one or more large cracks. This can be prevented with practical additional reinforcement (see Table 21 and here after under ‘durability’ and ‘shrinkage’).

Durability:
The durability of the steel sheet through crack formation in the concrete floor only comes into question when the top of the floor is exposed to an aggressive environment. Distributing the crack formation over more small cracks by means of additional reinforcement is also valid here.

In this chapter a number of practical instructions are given for the application of additional reinforcement to distribute crack formation. In this we must make a distinction between cracks resulting from shrinkage, temperature or bending. Also examples will be given of a number of practical situations that require special attention.

A choice can be made between the application of additional reinforcement and the introduction of a visible seam.

In regard to previous crack formation the following can be observed in practice:
- crack formation is mainly caused by shrinkage;
- the end shrinkage in relatively thin floors is reached within a relatively short period (approximately 90% of the end shrinkage a year after pouring);
- end value shrinkage approximately 0.3 to 0.5 mm/m;
- as a result of shrinkage, a floor with wooden beams will suffer additional (limited) deflection; additional tensile stress in the concrete occurs above the supporting steel beams, sometimes causing crack formation;
- the primary cracks always occur above the steel beams for these reasons;
- this effect is increased by creeping effects;
- the cracks have no effect on the structural safety once they are repaired to prevent corrosion of the steel sheet when necessary

With regard to the necessary crack reinforcement in accordance with NEN-EN 1994-1-1:2004, the following can be observed:
- The floor does not actually fall within the application area of NEN-EN 1994-1-1:2004 in regard to measurements (thickness).
- According to art. 9.2.1 of EC4 the floor must be provided with a mesh reinforcement of a minimum of 80 mm^2/m. This is the same as the Ø5-150 that is already applied in many situations to provide the necessary structural safety.
- The centre to centre distances of the reinforcement may not actually be more than 100 mm (for a 50 mm thick floor).

3.4.2.2 *Practical instructions*

Deflection:
The deflection of a floor is partially determined by the sagging of the supports in the floor. The LEWIS floor supported by a steel construction is drawn in Figure 19. The deflection in the middle of the floor is determined by the sagging of the main joist, the primary beam and lastly by the LEWIS floor itself. This last sagging is frequently

relatively limited in respect to the other two examples of sagging. In the end the total sagging must comply with the demands made.

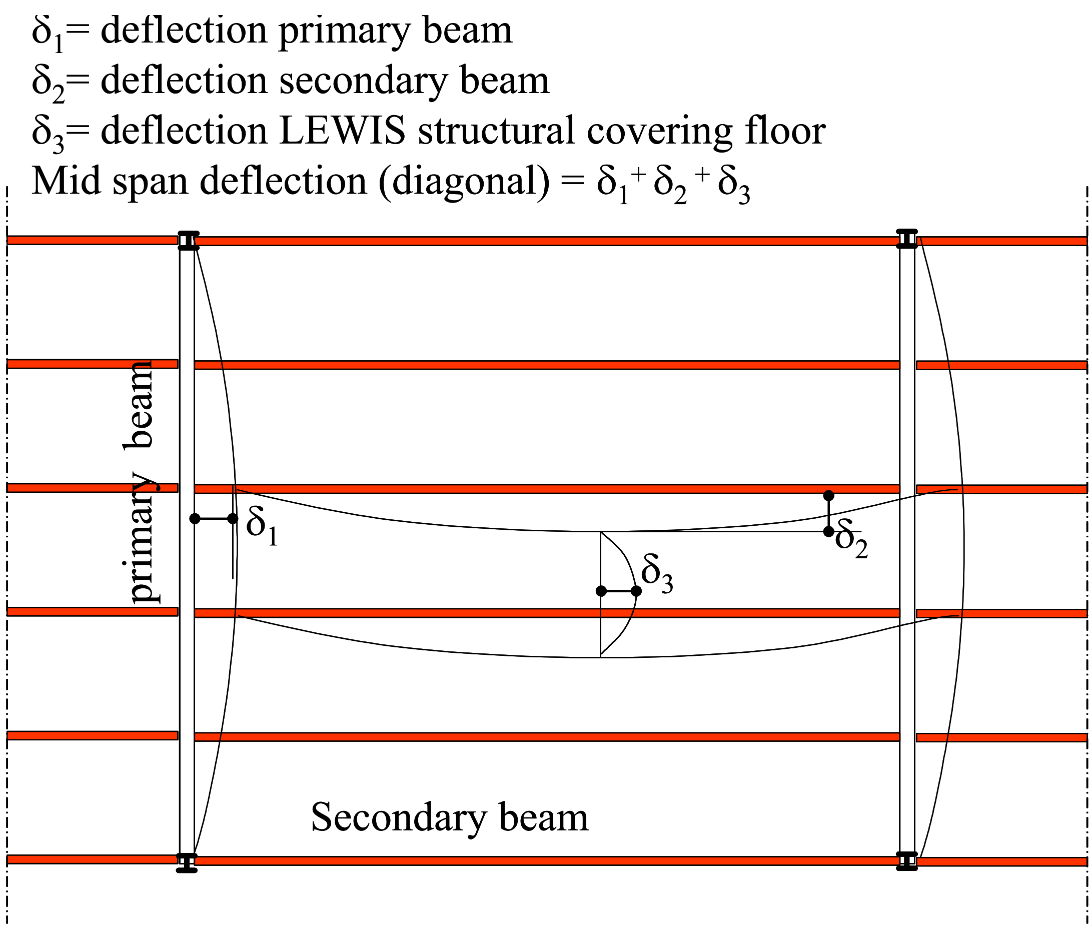


Figure 19 Deflection of the floor.

Two aspects are of importance in the deformation of a floor according to NEN 6702: the sagging of the floor and the additional deflection. In the first case, mostly a subjective aspect of the deformation is involved. The subsidence of the under side of the floor must not be such that people find it uncomfortable to walk under. A limit to the sagging of 1/250 of the span is used. This aspect is of no importance when a suspended ceiling is present. Even so the vibration of the floor is judged in NEN 6702 in relation to this requirement. In regard to the deformation of the floor, the additional deflection of the floor always remains important; this is the deformation which happens to the top layer of the floor during use. The load on the concrete floor, after hardening, is often a variable load. If the floor is sensitive to deformation following these additional loads, the consequence could be that doors and windows stick or cracks appear in partition walls. The standard indicates minimum demands on additional deflection (1/333 of the span). In Figure 19, the diagonal of the whole floor area can be seen in this case as the span. It is advisable in cases occurring such as the application of brickwork partition walls to maintain sagging demands of 1/500 of the span.

Stiffness:
The contribution of the load on the floor to the support is determined by the stiffness and strength of the floor. The LEWIS floor has a greater stiffness in the direction of the ribs than in the perpendicular direction. Besides this the load is transferred more to the shortest span when there is the same stiffness in two directions. In Table 20, in the left-hand column the steel sheet is laid in the direction of the greatest span. The floor will only transfer the load in that direction after it has tried to transfer the load in transverse direction.

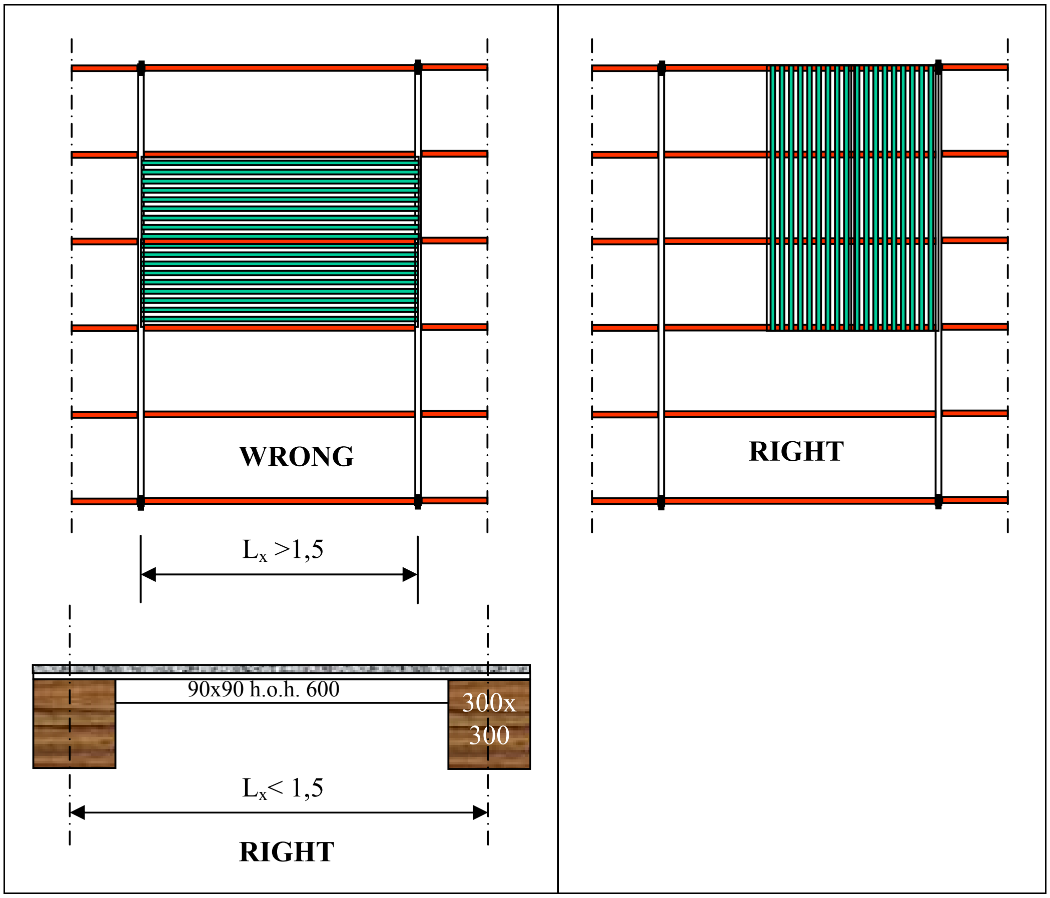


Table 10 Stiffness and span direction.

In regard to strength, it will not be successful in the transverse direction and cracks will occur, causing the stiffness proportion to decrease further in transverse direction and the load will eventually be carried after much deformation and crack formation in the longitudinal direction. The ribs of the LEWIS floor must usually be placed in the direction of the shortest span (right-hand column).
A general application rule is not possible here because a combination of spans and stiffness of the primary beam and the secondary beam is concerned. A situation that for example differs from what was generally indicated before is the application in building renovations, where often heavy wooden beams (approx. 300/300) at a centre to centre distance of approx. 1.5 m with, in between, ceiling joists (approx. 90/90) with a centre to centre distance of approx. 0.6 mm are applied (the so-called primary and secondary beams). In this case the LEWIS structural covering floor is placed in the direction of the

largest span (approx. 1.5 m) for the fire resistance aspect of this construction. That is why there is a limit of 1.5 m in the left-hand column of Figure 20.

Shrinkage:
The transverse overlapping of the LEWIS profiled steel sheet (see Table 21: left-hand column) are of course weak points where crack formation can easily occur. If tensile stresses occur in the longitudinal direction they are resisted in the first place by the steel sheet. Only at the joint to a new sheet there is no steel to transfer this tensile stress unless a hogging mesh reinforcement was installed. If the structural covering floor cannot move freely this kind of tensile stress can occur through shrinkage and temperature fluctuations. If a floating covering floor is supported on rock wool and is not connected to the beams, less tensile stress can occur than when the floor cannot move freely.

There is still a second aspect in which crack formation that can take place at the transverse overlapping of the LEWIS floor.
When the floor is laid as a floating covering floor, the load is distributed first by the 'uniformly elastic supported' covering floor. Then the bending moments occur in the structural covering floor. If no additional reinforcements have been used, crack formation can occur at the transverse edges of the sheet when the overlapping is inadequate.

Good transverse and longitudinal overlapping is imperative. Depending on the demands in regard to aesthetics and durability (see also page 26), a hogging mesh reinforcement at the overlap is desirable with large floor surfaces. In practice, it will often be a shrinkage reinforcement mesh in this case.

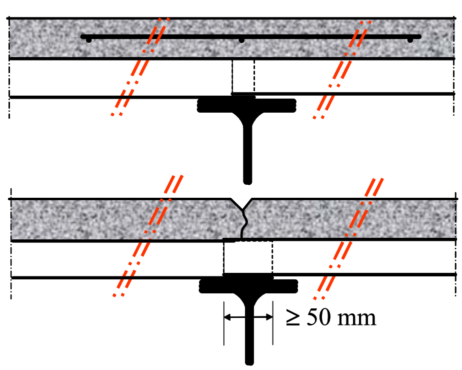
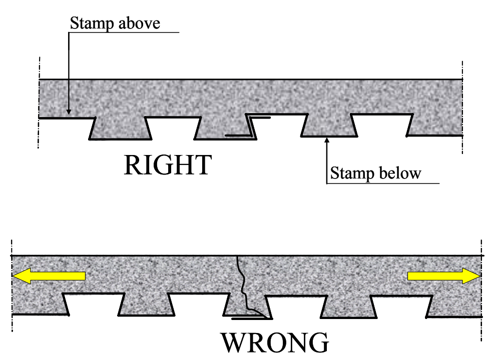
Transverse overlap	Longitudinal overlap
	

Table 21 Overlap steel sheet.

4 SUMMARY AND CONCLUSIONS

The resistance of the floor has been extensively researched in the last years. Experimental research has been carried out for the largest part by the Universität of Kaiserslautern. The results have led to an 'Allgemeine Bauaufsichtliche Zulassung' [2] (General Building Inspection Approval). The Zulassung (Approval) has been drawn up by the Deutsche Institut für Bautechnik (German Institute for Building Technology).
In a previously published TNO report, the design models used in the research carried out in Germany and in the various reports (see [8] up to and including [13]) are judged (see [20]). From this the following conclusions can be made:

- The longitudinal shear capacity of the LEWIS structural covering floor can be calculated on the basis of the m&k method as well as the 'partial connection' theory. The necessary characteristic values (m&k, respectively τ_d) are determined on the basis of the tests carried out.
- The tests with concentrated loads have shown that as well as the recognised failure phenomenon (bending, horizontal shear, vertical shear load and punching) a fifth failure mode can occur, namely bending in transverse direction. This failure mode can be indicative by a concentrated load on a not additionally reinforced LEWIS structural covering floor, especially when this concentrated load is to be found near a free edge of the floor.
- Failure through bending in transverse direction can be prevented by installing sufficient additional reinforcement to the floor. Another type of failure is indicative in an additional reinforced floor. Punching was always indicative in the tests carried out.
- In [13] a design model was set up for determining the bearing force of an additional reinforced LEWIS structural covering floor in regard to punching. This model is tested with the test results. The influence of the concrete grade is verified with numerical calculations. The design model produces safe values, especially for the lower concrete qualities (B25 for example).

Based upon experimental research and the consequent evaluation, the LEWIS structural covering floor has been checked for the Dutch building industry, for both the construction stage and the final stage. During the check the following were used:

- concrete grade B25,
- yield strength LEWIS profile steel sheet 320 N/mm^2,
- simply supported spans,
- span between 600 and 2500 mm,
- floors in building category in accordance with 8.2.2.1 of NEN 6702,
- total floor thickness minimum 50 mm.

The following characteristic loads have been used for the final stage:

- point loads
  building categories a and b: 3 kN
  building categories c and d: 7 kN
  building category e : 10 kN
- uniformly distributed loads
  building categories a and b : 3.5 kN/m^2 (variable load) +
  1.2 kN/m^2 (as a result of inner walls)
  building categories c, d and e : 5.0 kN/m^2 (variable load)

The following conclusions may be drawn from the check:

- The LEWIS structural covering floor must be provided with additional reinforcement in a number of situations for the Dutch building industry (see Table 22).
- Even if it can be proven that there are no free edges, the reinforcement necessary for the floor area must be applied in every case.
- The high load concentrations of 7 kN and 10 kN cannot be recorded near a support on an additionally reinforced LEWIS structural covering floor. The load cannot be sufficiently distributed over the width when the point load rests near the support, causing the floor to fail in places through bending, horizontal shear or vertical shear load.
- The application areas for the LEWIS floor are buildings in the category a and b. These are actually:
  a. Houses, mobile homes, overnight accommodation, outside storage, garages
  b. offices, educational establishments, health care facilities, apartments, overnight accommodation
- The allowable variable load is shown in Table 23

	Free edge	Floor area
Span	F_{rep} = 3 kN	F_{rep} = 3 kN
600 mm		
900 mm		
1200 mm	Ø5 – 150	
1500 mm	Ø5 – 150	
2000 mm	Ø5 – 150	Ø5 – 150
2500 mm	Ø5 – 150	Ø5 – 150

Table 22 Additional reinforcement for the resistance of concentrated load of 3 kN.

Span in mm	Floor thickness in mm	Allowable variable load p_{rep} in kN/m^2	Allowable concentrated load F_{rep} in kN	Remarks
600	50	30.2	3	
900	50	19.5	3	
1200	50	13.8	3	
1500	50	9.7	3	
2000	75	6.1	3	
2500	75	4.1	3	

1) Mesh reinforcement Ø5 – 150 recorded for resistance of concentrated load of 3 kN

Table 23 Resistance of LEWIS Re-entrant profiled steel sheet floors in accordance with NEN 6702 and NEN-EN 1994-1-1:2004.

5 REFERENCES

[1] ‘LEWIS re-entrant profiled steel sheets’, brochure REP 9811, Reppel B.V. Bouwspecialiteiten, Dordrecht.

[2] Deutsche Institut für Bautechnik, ‘Allgemeine Bauaufsichtlucht Zulassung. LEWIS-Böden’, Approval number Z-26.1-36, 16 November 1998.

[3] Universität Fridericiana Karlsruhe, ‘Tragfähigkeitsuntersuchung von Stahlblechprofilen LEWIS SC 16/63, Bericht Nr. 90 0076/2, 17 May 1990.

[4] Van Hove, “Experimental research into the resistance of the LEWIS re-entrant profiled steel sheet in the final stage”, TNO report B-91-0281, March 1991.

[5] Brekelmans and Van Hove, ‘Resistance of the LEWIS re-entrant profiled steel sheet in the construction stage and in the final stage’, TNO report B-91-0328, May 1991.

[6] Reppel/Kiwa, ‘Testing the resistance of LEWIS sheets finished off with gravel concrete’, 22.06.1993.

[7] Brekelmans, ‘Experimental research’, TNO Bouw 95-CON-B0613/BSJ, 8 March 1995.

[8] Bode en Minas, ‘Gutachten über das Tragverhalten der LEWIS-Fußbodenkonstruktion mit der Blechdicke t = 0.50 mm’, Universität Kaiserslautern, November 1995.

[9] Bode en Minas, ‘Ergänzung zur Gutachten über das Tragverhalten der LEWIS-Fußbodenkonstruktion mit der Blechdicke t = 0.50 mm’, Universität Kaiserslautern, February 1996.

[10] Bode en Minas, ‘Gutachten über das Tragverhalten der LEWIS-Fußbodenkonstruktion bei konzentrierter Belastung (Punkt- und Linienlasten)’, Universität Kaiserslautern, januari 1997.

[11] Bode en Minas, ‘Gutachten über das Tragverhalten der LEWIS-Fußbodenkonstruktion bei konzentrierter Punktbelastung mit künstlichen Rissen’, Universität Kaiserslautern, mai 1997.

[12] Bode en Däuwel, ‘Gutachten zum Tragverhalten von LEWIS-Fußbodenplatten mit einer Blechdicke von t = 0,50 mm bei randnahen Einzellasten’, Universität Kaiserslautern, December 1997.

[13] Bode en Däuwel, ‘Gutachten zum Tragverhalten von bewehrten LEWIS-Fußbodenplatten mit einer Blechdicke von t = 0,50 mm bei Belastung durch randnahe Einzellasten und bei Ausbildung als Durchlaufplatten’, Universität Kaiserslautern, July 1998.

[14] ENV 1991: Basis of design and actions on structures

[15] NEN-EN 1994-1-1:2004: Design and calculation of steel and concrete constructions – Part 1-1: General rules and rules for buildings.

[16] NEN 6702, TGB 1990 Loads and distortion, NNI, 1st edition 1991.

[17] NEN 6720, TGB 1990 Concrete constructions, NNI, 1st edition 1991.

[18] ENV 1992: Design of concrete structures

[19] Van Hove, “LEWIS structural covering floor. Part 2”, TNO-report 1999-CON-BIS-R5006/2, June 1999

[20] Van Hove, “LEWIS structural covering floor. Part 1”, TNO-report 1999-CON-BIS-R5006/1, June 1999

A Load table for LEWIS re-entrant profiled steel sheet floors in accordance with NEN 6702 and NEN-EN 1994-1-1:2004

Summary from TNO report 2005-BCS-R0000/1 dated July 2005

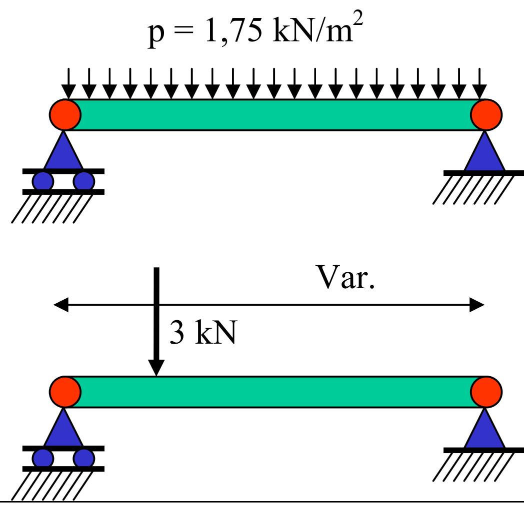
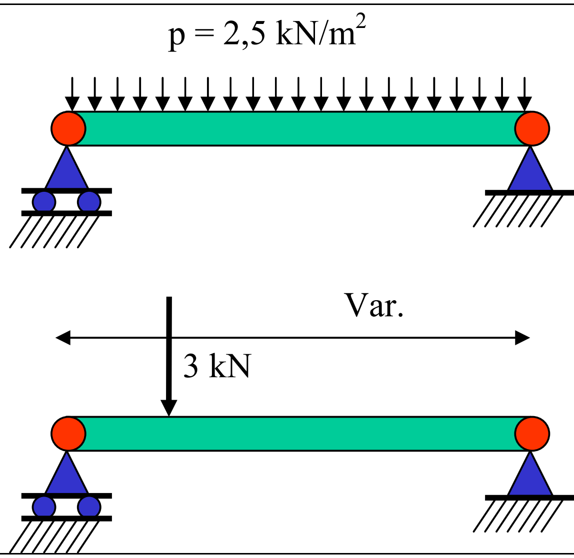
		Theoretical span						
Application	Uniformly distributed effective floor load p_{rep} and concentrated floor load F_{rep} in accordance with TGB 1990 NEN 6702	floor thickness	600 mm	900 mm	1200 mm	1500 mm	2000 mm	2500 mm
Houses and overnight accommodation		50			ϕ5-150 at the free edge	ϕ5-150 at the free edge		
		75					ϕ5-150 whole floor	ϕ5-150 whole floor

Table 24 Loads in accordance with NEN 6702, applications inclusive accompanying additional reinforcement.

		Theoretical span						
Application	Uniformly distributed effective floor load p_{rep} and concentrated floor load F_{rep} in accordance with TGB 1990 NEN 6702	floor thickness	600 mm	900 mm	1200 mm	1500 mm	2000 mm	2500 mm
Offices, educational establishments and health care facilities.		50			ϕ5-150 at the free edge	ϕ5-150 at the free edge		
		75					ϕ5-150 whole floor	ϕ5-150 whole floor

(continuing) Table 24 Loads in accordance with NEN 6702, applications inclusive of accompanying additional reinforcement.

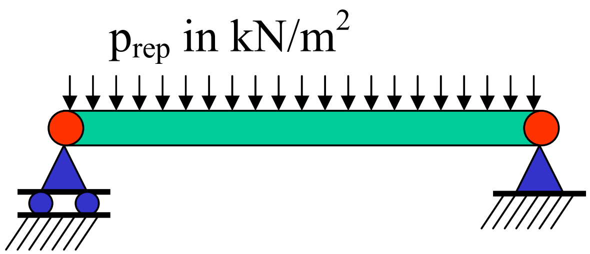
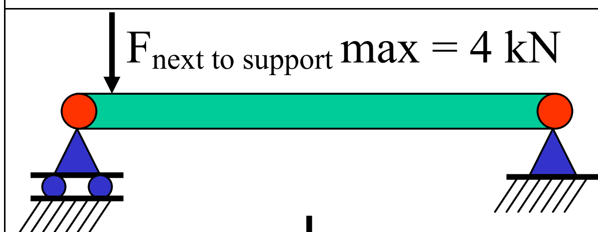
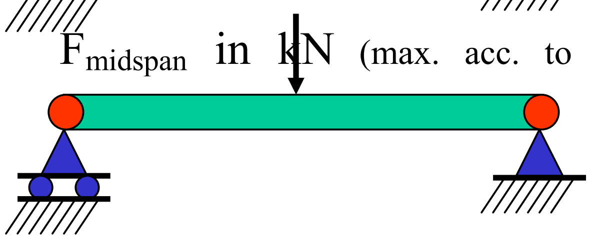
		Theoretical span						
Application	Uniformly distributed effective floor load p_{rep} and concentrated floor load F_{rep} in accordance with TGB 1990 NEN 6702	floor thickness	600 mm	900 mm	1200 mm	1500 mm	2000 mm	2500 mm
Various applications		50	p_{rep} = 30.2 kN/m^2	p_{rep} = 19.5 kN/m^2	p_{rep} = 13.8 kN/m^2	p_{rep} = 9.7 kN/m2		
		75					p_{rep} = 6.1 kN/m^2	p_{rep} = 4.1 kN/m^2
	 	50	$F_{mid\ span}$ 7 kN + B37.5 + ϕ5-150	$F_{mid\ span}$ 7 kN + B37.5 + ϕ5-150	$F_{mid\ span}$ 7 kN + B37.5 + ϕ5-150	$F_{mid\ span}$ 7 kN + B37.5 + ϕ5-150		
		75					$F_{mid\ span}$ 10 kN + ϕ5-150	$F_{mid\ span}$ 10 kN + ϕ5-150

Table 25 Maximum allowable floor loads p_{rep}.

B Bending stiffness of the LEWIS structural covering floor

$kN := 1000 \cdot N$

$ribdistance := 39 \cdot mm + 25 \cdot mm$

$width\_upperflange := 39 \cdot mm$

$width\_bottomflange := 35 \cdot mm$

$steelthickness := 0.5 \cdot mm$

$steelcorethickness := steelthickness - 0.02 \cdot mm - 0.02 \cdot mm$

$depth\_profile := 16 \cdot mm$

$profiledepth := depth\_profile - steelthickness$

$slabthickness := 50mm - steelthickness$

$$elast\_steel := 2.1 \cdot 10^5 \cdot \frac{N}{mm^2}$$

$n\_factor := 15$

$$elast\_concrete := \frac{elast\_steel}{n\_factor} \qquad elast\_concrete = 1.4 \times 10^4 \frac{N}{mm^2}$$

$$web\_length := \sqrt{profiledepth^2 + \left(\frac{ribdistance - width\_bottomflange - width\_upperflange}{2}\right)^2}$$

$web\_length = 16.3mm$

$$lijf\_thickness := \frac{web\_length}{profiledepth} \cdot steelcorethickness$$

$area\_rib := (width\_bottomflange + width\_upperflange) \cdot steelcorethickness + 2 \cdot profiledepth \cdot lijf\_thickness$

$$neutral\_axis := \frac{\left[\begin{array}{l}(width\_upperflange \cdot profiledepth) \cdot steelcorethickness \ \ldots \\ + lijf\_thickness \cdot profiledepth^2\end{array}\right]}{area\_rib}$$

$$\text{neutral\_axis} = 8\,\text{mm}$$ (from centre line steel sheet)

$$\text{traagheidsmoment\_rib} := \left[\begin{aligned}&\text{width\_bottomflange}\cdot\text{neutral\_axis}^2 \ldots\\ &+\text{width\_upperflange}\cdot(\text{profiledepth}-\text{neutral\_axis})^2\end{aligned}\right]\cdot\text{steelcorethickness}\ \ldots$$
$$+2\cdot\text{lijf\_thickness}\cdot\text{profiledepth}\cdot\left(\frac{\text{profiledepth}}{2}-\text{neutral\_axis}\right)^2 \ldots$$
$$+\frac{\text{width\_upperflange}+\text{width\_bottomflange}}{12}\cdot\text{steelcorethickness}^3+2\cdot\frac{\text{lijf\_thickness}}{12}\cdot\text{profiledepth}^3$$

$$\text{traagheidsmoment\_rib} = 2.3\times10^3\,\text{mm}^4$$

$$\text{bendingstiffness\_sheet} := \text{traagheidsmoment\_rib}\cdot\text{elast\_steel}\cdot\left(\frac{\text{m}}{\text{ribdistance}}\right)$$

$$\text{bendingstiffness\_sheet} = 7.7\,\text{kN}\cdot\text{m}^2$$

$$\text{concrete\_area\_rib} := \text{ribdistance}\cdot\text{slabthickness}\ \ldots$$
$$+(-\text{ribdistance}+\text{width\_bottomflange})\cdot\text{depth\_profile}\ \ldots$$
$$+(-\text{width\_upperflange}+\text{ribdistance}-\text{width\_bottomflange})\cdot\frac{\text{depth\_profile}}{2}$$

$$\text{concrete\_n\_axis} := \left[\frac{\begin{aligned}&\text{ribdistance}\cdot\frac{\text{slabthickness}^2}{2}\ \ldots\\ &+(-\text{ribdistance}+\text{width\_bottomflange})\cdot\frac{\text{depth\_profile}^2}{2}\ \ldots\\ &+(-\text{width\_upperflange}+\text{ribdistance}-\text{width\_bottomflange})\cdot\frac{\text{depth\_profile}^2}{3}\end{aligned}}{\text{concrete\_area\_rib}}\right]$$

$$\text{concrete\_n\_axis} = 28.1\,\text{mm}$$ (from top steel sheet)

$$\text{concrete\_traagh\_rib} := \text{ribdistance}\cdot\text{slabthickness}\cdot\left(\frac{\text{slabthickness}}{2} - \text{concrete\_n\_axis}\right)^2 \ldots$$
$$+ (-\text{ribdistance} + \text{width\_bottomflange})\cdot\text{depth\_profile}\cdot\left(\frac{\text{depth\_profile}}{2} - \text{concrete\_n\_axis}\right)^2 \ldots$$
$$+ (-\text{width\_upperflange} + \text{ribdistance} - \text{width\_bottomflange})\cdot\frac{\text{depth\_profile}}{2}\cdot\left(\frac{2\cdot\text{depth\_profile}}{3} - \text{concrete\_n\_axis}\right)^2 \ldots$$
$$+ \frac{\text{ribdistance}}{12}\cdot\text{slabthickness}^3 \ldots$$
$$+ \frac{(-\text{ribdistance} + \text{width\_bottomflange})}{12}\cdot\text{depth\_profile}^3 \ldots$$
$$+ \frac{(-\text{width\_upperflange} + \text{ribdistance} - \text{width\_bottomflange})}{3}\cdot\text{depth\_profile}^3$$

$$\text{bendingstiffness\_concrete} := \text{concrete\_traagh\_rib}\cdot\text{elast\_concrete}\cdot\left(\frac{\text{m}}{\text{ribdistance}}\right)$$

$$\text{n\_axis} := \frac{\frac{\text{concrete\_area\_rib}}{\text{n\_factor}}\cdot\left(\text{concrete\_n\_axis} + \frac{\text{steelthickness}}{2}\right) + \text{area\_rib}\cdot\text{neutral\_axis}}{\frac{\text{concrete\_area\_rib}}{\text{n\_factor}} + \text{area\_rib}}$$

$\text{n\_axis} = 23.9\text{mm}$ (from centre line steel sheet)

$$\text{bendingstiffness\_slab} := \text{bendingstiffness\_sheet} + \frac{\text{bendingstiffness\_concrete}}{\text{n\_factor}} \ldots$$
$$+ \text{elast\_steel}\cdot\text{area\_rib}\cdot\left(\frac{\text{m}}{\text{ribdistance}}\right)\cdot(\text{n\_axis} - \text{neutral\_axis})^2 \ldots$$
$$+ \text{elast\_steel}\cdot\frac{\text{concrete\_area\_rib}}{\text{n\_factor}}\cdot\left(\frac{\text{m}}{\text{ribdistance}}\right)\cdot\left[\text{n\_axis} - \left(\text{concrete\_n\_axis} + \frac{\text{steelthickness}}{2}\right)\right]^2$$

$\text{bendingstiffness\_slab} = 66.2\text{kN}\cdot\text{m}^2$