TNO-report

94-CON-R0965-2

Final report: HODY composite floor system
Shear bond optimisation of a new composite decking

TNO Building and
Construction Research

Lange Kleiweg 5, Rijswijk
P.O. Box 49
2600 AA Delft
The Netherlands

Phone +31 15 284 20 00
Fax +31 15 284 39 90
Telex 38270

Date
7 March 1996

Author(s)
Ing. J.W.P.M. Brekelmans
Dr.ir. B.J. Daniels

To:
- Dywidag-systems International B.V.
- Ingenieursbureau Bartels B.V.
- Hoogovens Staalverwerking

Projectname : Stabo-vloer fase 4+5
Projectno. : 5.21.4.7896

Pages : 48
Tables : 13
Figures : 14
Appendices : 4

Theme : Staalplaat, betonvloeren
WP-subject : 221
Keyword(s) : Steel-concrete slabs, steel sheeting

SUMMARY

This report contains a summary of a project aimed at improving the longitudinal shear bonding characteristics of a new decking. In the final phase full-scale composite slabs were tested. These tests are described and their results are presented in this report. The objective of these tests is to determine their shear bond capacity in accordance with the Eurocode 4 "Design of composite steel and concrete structures". A comparison between full-scale test results and small-scale "push-off" test results is given. A Dutch summary of the research and its conclusions is given in appendix IV of this report.

1 INTRODUCTION

This final report summarises the important findings of a collective research project in which a decking was optimised for a proprietary system consisting of cast-in-place concrete beams and composite slabs. Two interim reports were written for this project [1][2].

In the first interim report several modified decking rib geometries were developed, thought to have improved longitudinal shear-bonding characteristics. These were then prototyped and tested using a small-scale "push-off" test. End anchorage provided by concrete encasement was also tested using a small-scale "pull-out" test. From these tests a choice was made for the final decking rib geometry.

In the second interim report an estimation of the actual longitudinal shear bonding characteristics of the chosen decking geometry in a full-scale composite slab was made. Based upon this estimate, the minimum span length at which the full plastic cross-sectional resistance could be reached was calculated for a number of geometrical combinations. Preliminary load tables were made based upon the recommendations in the second interim report.

Based upon the results of the first two interim reports, a choice for the geometry of nine full-scale test specimens was made. In this report these full-scale composite slabs tests and test results are presented. The objective of these tests is to determine their shear bond capacity according to the Eurocode 4 "Design of composite steel and concrete structures" [3]. As such all specimens, tests and analyses are made in accordance with the Eurocode 4 and the Dutch NAD (National Application Document). A comparison between the results of these tests and "push-off" test results is given. Full-scale test results are analysed according to the Eurocode 4 and design values for the longitudinal shear bonding between concrete and decking are given.

2 DESCRIPTION OF FULL-SCALE TESTS

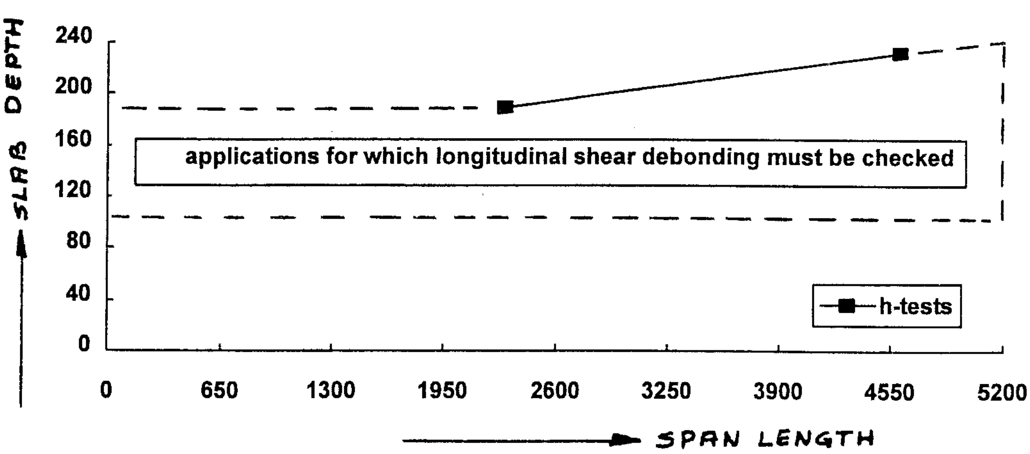
2.1 General

A total of 9 full-scale composite slabs were tested. The principle nominal cross-sectional and longitudinal dimensions of these specimens are given in Table 1. The definitions of the variables indicated in Table 1 are shown in Figure 1.

The cross-sectional dimensions of the decking are shown in Figure 2. This decking, referred to as the HODY-SB 60x202x0.75 profile, was delivered already installed in wooden formwork. Measured cross-sectional decking dimensions were in all cases within tolerances specified by the Dutch NAD to Eurocode 4 for full-scale shear bond test specimens.

The formwork supported the decking at approximately 500 mm intervals along its length. Figure 3 shows the decking installed in the formwork ready for concreting.

For this decking two applications are foreseen; composite slabs, with and without additional sagging reinforcement. Practical span lengths and slab depths for which the longitudinal shear debonding mode of failure must be checked, are indicated in the following graph.



The behaviour of the applications with and without reinforcement, were analysed using the same test results and the two longitudinal shear debonding methods contained in the Eurocode 4:

- The m&k-method:
  In this design method three short & thick and three long & slender specimens are tested without reinforcement. A linear interpolations is drawn between the results from the two test series.

- The τ–method:
  In this design method the first test series (short & thick without reinforcement) is intended to show that the composite slab remains ductile. The second test series (long & slender without reinforcment) is intended to have an partial shear interaction percentage between 70% and 100% and is used to determine longitudinal shear bonding characteristics. A third series of tests is conducted on similar specimens with reinforcement, such that the influences of the reinforcement may be analized.

To satisfy the requirements of these two design methods and applications, specimen geometries were chosen as follows:

- First a conservative choice for the thickness of the short span specimens was made. As thickness between 110 mm and 200 mm are anticipated, 190 mm was chosen.
- To fulfill the requirements of minimum shear span slenderness (three times the shear span length) this resulted in an overall span length of about 2300 mm.

From previous "push-off" test results and their evaluation [2], full shear-bond interaction was foreseen for span lengths of about 5500 mm. In practice the slab thickness of this span length would be about L/25 or 230 mm. To ensure that the tested moment capacity would remain below the plastic moment capacity a span length of 4800 mm was chosen.

Reinforcement diameter and spacing was chosen to be a lower bound for most applications.

2.2 Material properties

Nominal and measured material properties for the concrete, decking and reinforcement are given in Tables 2, 3 and 4. The nominal concrete grade was C25/30 (equivalent to a Dutch B30 notation). As may be seen in Table 2, the actual concrete grade was close to nominal. Two tensile test coupons were cut from the decking delivered separately and one tensile test coupon was cut out of a tested specimen (specimen 3 near the support). The nominal thicknesses of the decking are 0.75 mm (with zinc) and 0.71 mm (without zinc). The average measured decking thicknesses without the zinc (core steel thickness) is 0.68 mm. This is 4.6 % less than nominal. The reinforcement yield stress is 12% higher than nominal.

Table 1: Nominal cross-sectional and longitudinal dimensions.

Specimen number	Width b mm	Height h mm	h_p mm	h_c mm	Length L mm	L_s mm	L_o mm	Reinforcement	h_s mm
1	1126	190	60	130	2300	575	50	none	
2	1126	190	60	130	2300	575	50	none	
3	1126	190	60	130	2300	575	50	none	
4	1126	230	60	170	4800	1200	50	none	
5	1126	230	60	170	4800	1200	50	none	
6	1126	230	60	170	4800	1200	50	none	
7	1126	230	60	170	4800	1200	50	ϕ10 -202	70
8	1126	230	60	170	4800	1200	50	ϕ10 -202	70
9	1126	190	60	130	2300	575	50	ϕ10 -202	70

Table 2: Nominal and measured concrete material characteristics.

Specimen number	Tested on day (*)	f_{ck} (**) N/mm^2	f_c (***) N/mm^2	f_t (***) N/mm^2	Volumetric mass kg/m^3
-	7	C25/30	21.8		2378
-	14	C25/30	25.7		2371
-	21	C25/30	31.5		2340
-	28	C25/30	31.7		2380
1	21	C25/30	31.2	3.48	
2	22	C25/30	29.7	2.92	
3	26	C25/30	29.9	3.17	
4	33	C25/30	33.2	3.17	
5	34	C25/30	33.2	3.19	
6	35	C25/30	33.0	3.26	
7	40	C25/30	34.1	3.40	
8	41	C25/30	34.8	3.62	
9	43	C25/30	34.2	3.31	

* Concreting date was 27 April, 1995

** f_{ck} is specified as C25/30 by EC4 which is a B30 in accordance with the NEN 6720 (cylinder strength = 25 N/mm^2, cube strength = 30 N/mm^2)

*** f_c, f_t are measured using concrete cubes, tested according to the NEN 5968

(f_c is the compressive strength, f_t is the split-tensile strength)

Table 3: Nominal and measured decking material characteristics.

Coupon number	Thickness (w/o zinc) mm	f_y measured N/mm^2	f_y nominal N/mm^2	f_u measured N/mm^2	ε_{ult} estimated %
1	0.69	337	320	413	8.3
2	0.68	315	320	414	9.7
3	0.67	346	320	416	7.4

Table 4: Nominal and measured reinforcement material characteristics.

Coupon number	Diameter mm	f_y measured N/mm^2	f_y nominal N/mm^2	f_u measured N/mm^2	ε_{ult} measured %
1	9.8	561	500	621	4.7
2	9.8	570	500	624	3.4

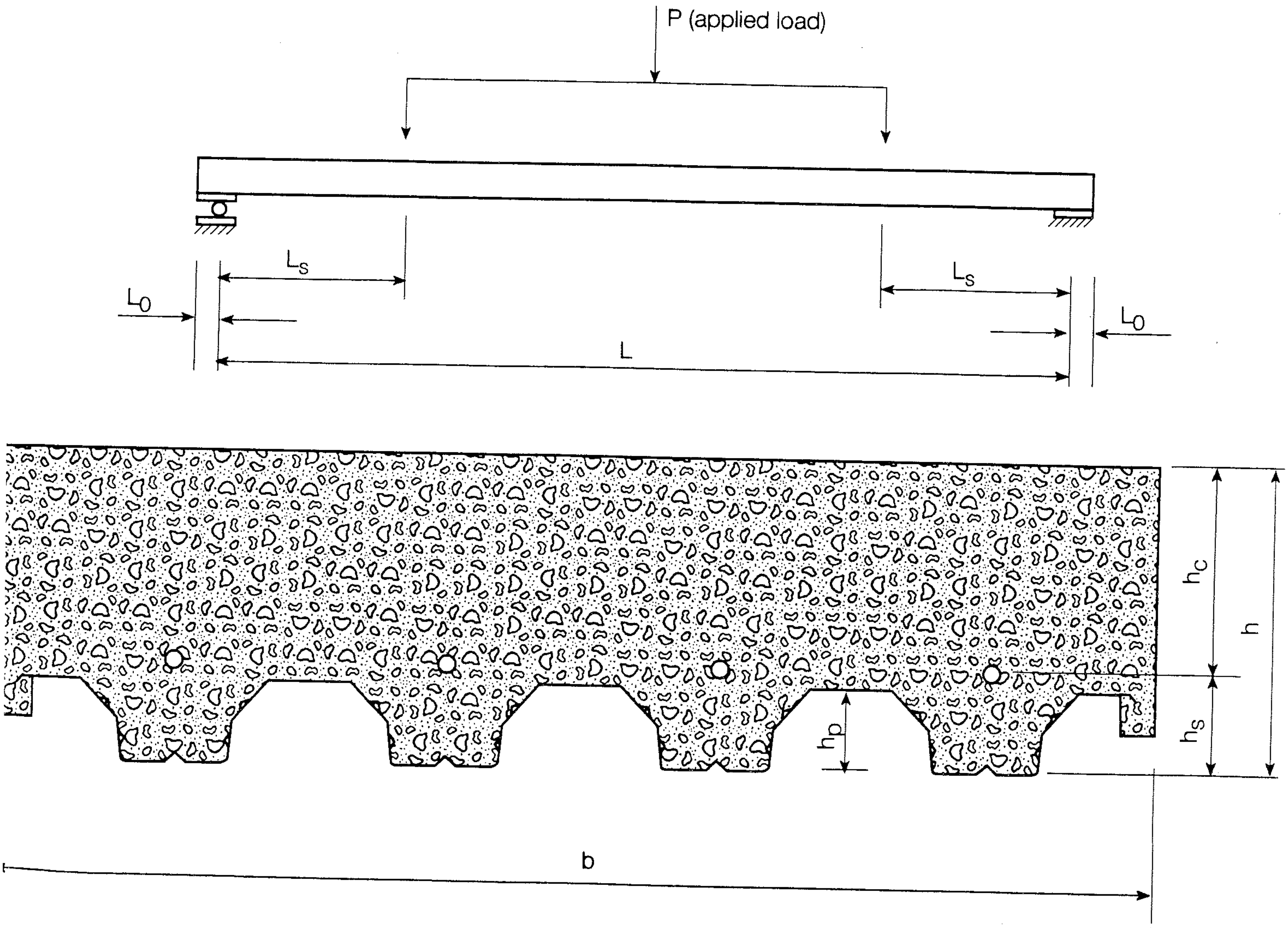


Figure 1: Notation for major specimen dimensions.

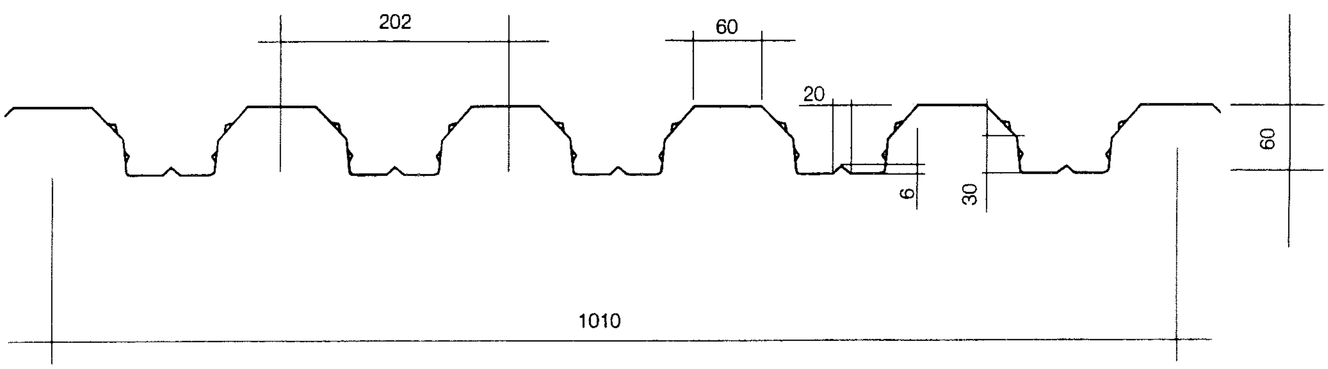


Figure 2: Nominal cross-sectional dimensions of the decking tested.

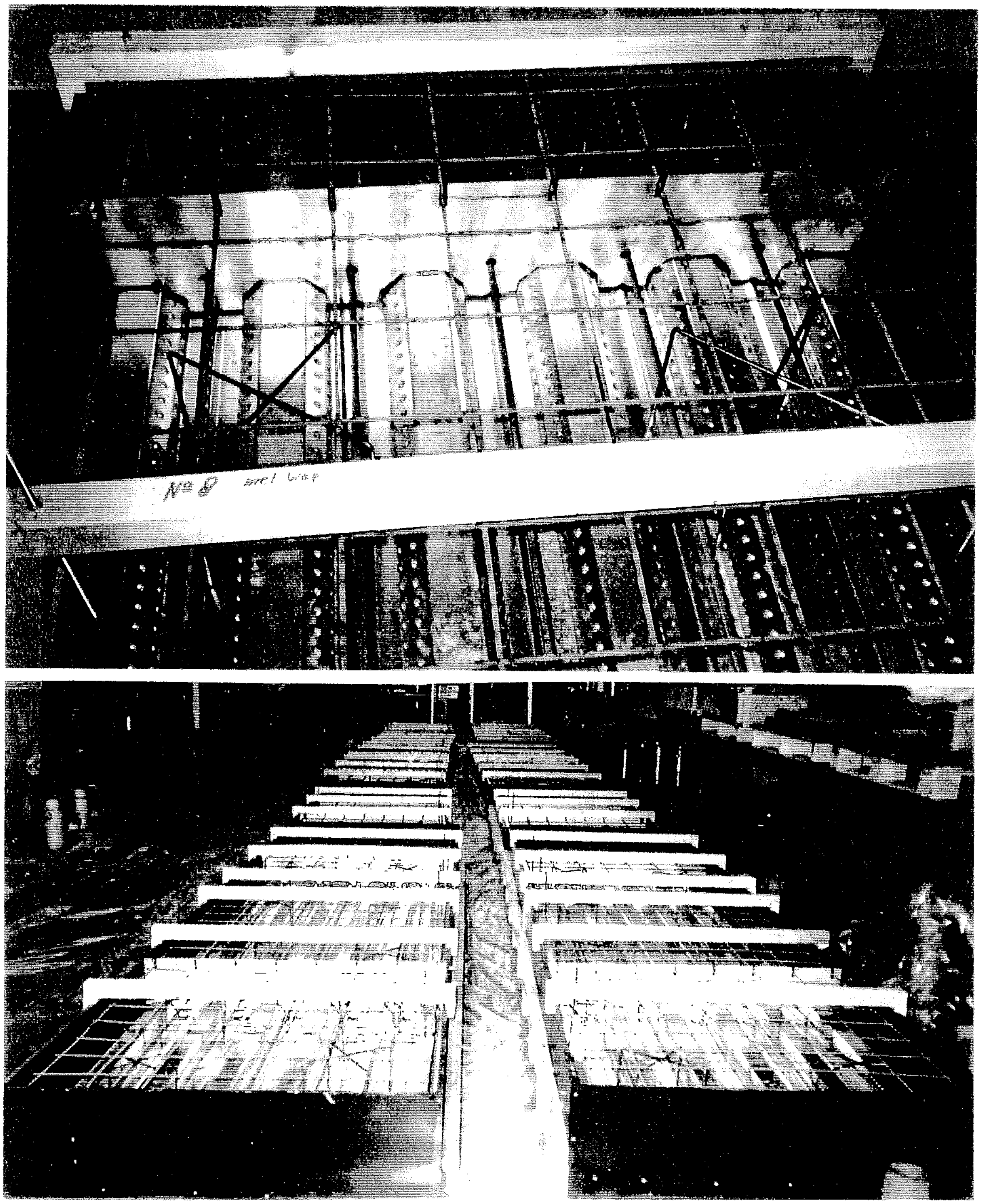


Figure 3: Photo of decking installed in formwork, as delivered to TNO, ready for concreting.

3 Testing arrangement, loading and instrumentation

3.1 Testing arrangement

The nominal dimensions of the testing frame are shown in Figures 4 and 5. A typical view of this arrangement with a test specimen installed is shown in Figure 6. Note that the specimens are simply supported with a roller at one end and a pinned support at the other. Loads are applied using a system of spreader beams (see Figure 4).

3.2 Test loading procedure

The loading sequence of each specimen is illustrated by means of Figure 7. Note in this figure that specimen weight and the corresponding midspan deflection must be added to all measured test values as the specimens were prepared in the fully supported condition (as specified in Eurocode 4).

1. **Loading procedure for specimens without cycling (Specimens 1,3,4,6 and 7)**

   The initial loading rate was set based upon the uncracked stiffness of the composite slab, such that first cracking would be observed after about 1 hour. At these low load levels the test was load controlled. After the observation of significant cracking, or first slip, the test was stopped and changed to deflection control. This allows the unloading characteristics of the specimen to be observed. Deflection rates were increased after a significant reduction in load carrying capacity was observed. Maximum imposed deformations were typically near L/150. Total testing times were about 3 hours.

2. **Loading procedure for specimens with cycling (Specimens 2,5,8 and 9)**

   The initial loading rate was set based upon the uncracked stiffness of the composite slab, such that first cracking would be observed after about 1 hour. Load was applied to 1.5 P_{av}, and then reduced to P_{av} before a final measurement prior to cycling (see Figure 7, where P_{av} is the service load to a maximum of the load at which moment failure will occur divided by 1.5). Cycling was performed using a load control between $0.5P_{av}$ and $1.5P_{av}$. A total of 5000 cycles were applied in a time exceeding 3 hours. After cycling subsequent loading was performed using deflection control from a load of P_{av}. In some specimens cycling was repeated at higher load level if no significant increase in midspan deflection or end slip was observed. Deflection rates were increased after a significant reduction in load carrying capacity was observed. Maximum imposed deformations were typically near L/150. Total testing times, with one series of cycling, was about 6 hours.

According to the standard test procedure in the Eurocode 4 to determine longitudinal shear characteristics two of three tests have to be cycled to prove that the connection is adequate after long-term loading. For the additional tests with reinforcement (Tests 7 to 9) this is not required.

In this specific case where two applications are foreseen (with and without reinforcement) it was decided to carry out one cyclic test for each group of tests without reinforcement and one cyclic test for each group of tests with reinforcement. The cyclic load level for the tests with reinforcement was increased such that the higher load level produced the same (or greater) longitudinal shear in the decking as for the corresponding specimen without reinforcement, thus fulfilling Eurocode requirements.

3.3 Instrumentation

A list of all measuring devices is given in Table 5. Their locations on the specimen are shown in Figures 4 and 5. Detailed illustrations of individual measuring devices for end slip and midspan deflections are shown in Figures 7 and 8. All measurements are automatically read using a data acquisition system linked to a computer, which records them. The time between measurement recording varies, depending upon the loading rate, between 10 and 30 seconds.

Table 5: List of measurement devices.

Measurement	Device and type	Maximum capacity	Accuracy
Applied load	Load cell (TNO made)	500 kN	+/- 0.100 %
Midspan deflection	Linear translation transducer (Schlumberger)	100 mm	+/- 0.006 %
End slip	Linear translation transducer (Schlumberger)	10 mm	+/- 0.005 %

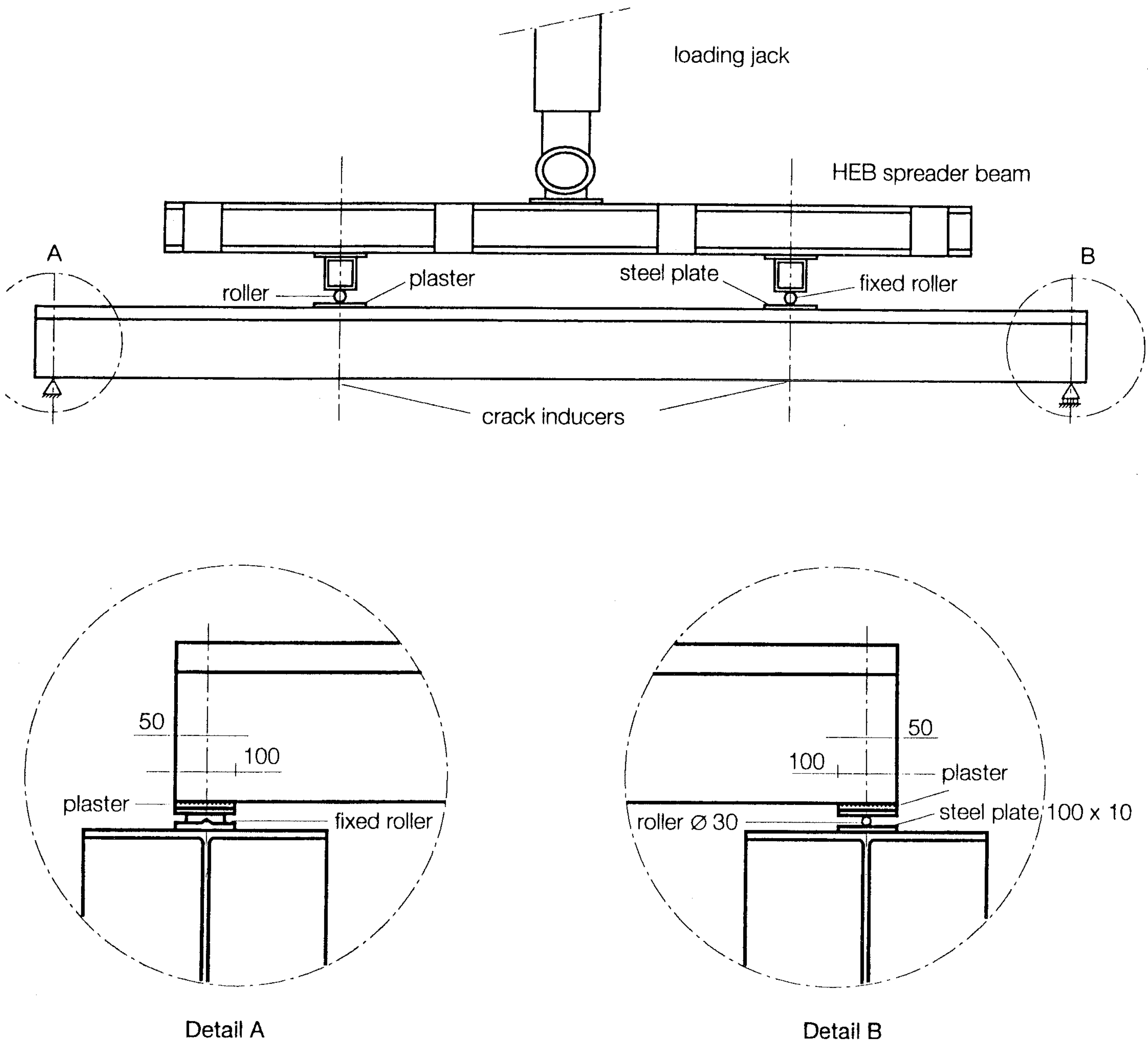


Figure 4: Testing arrangement: Longitudinal view.

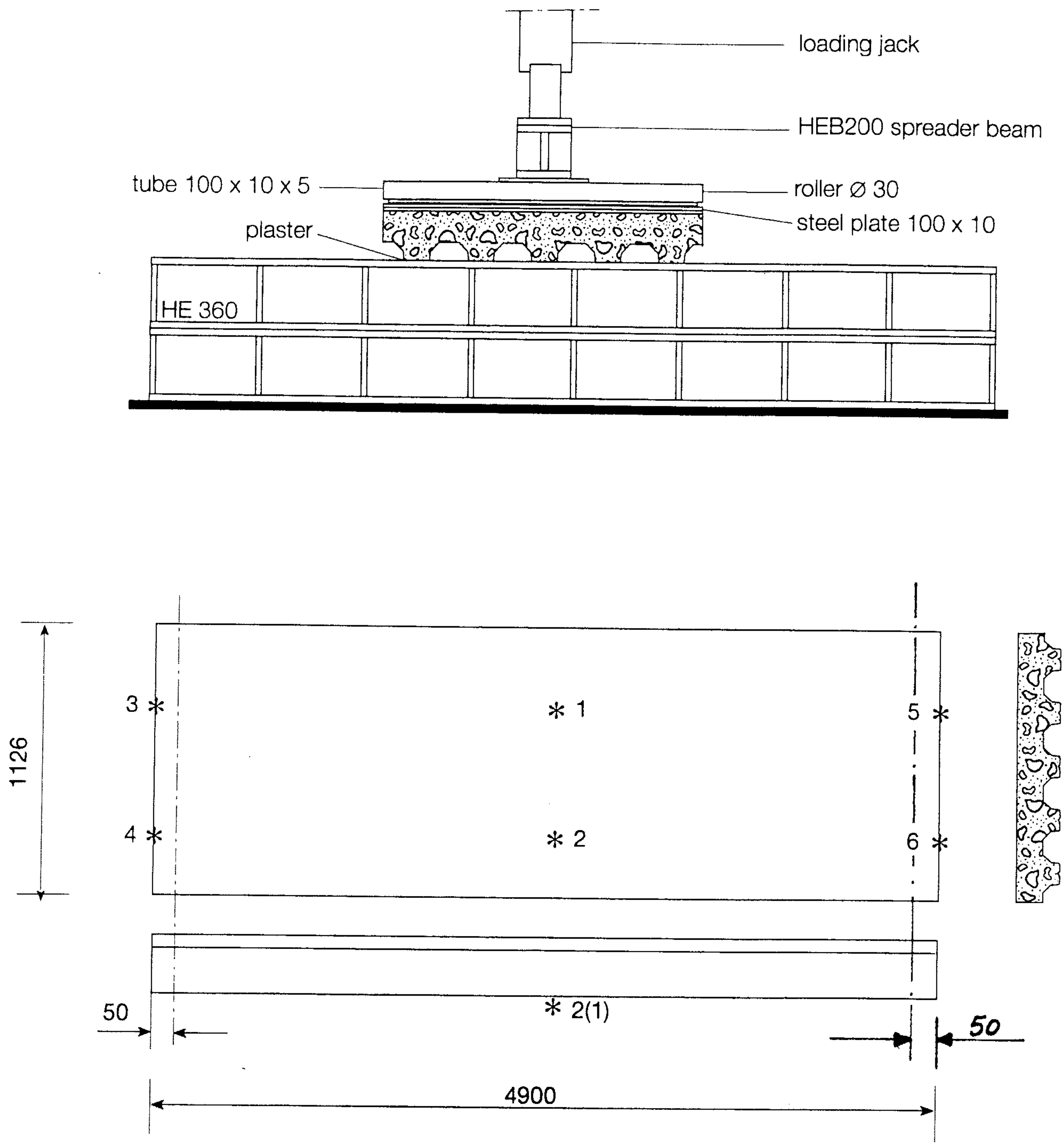


Figure 5: Testing arrangement: Transverse view.

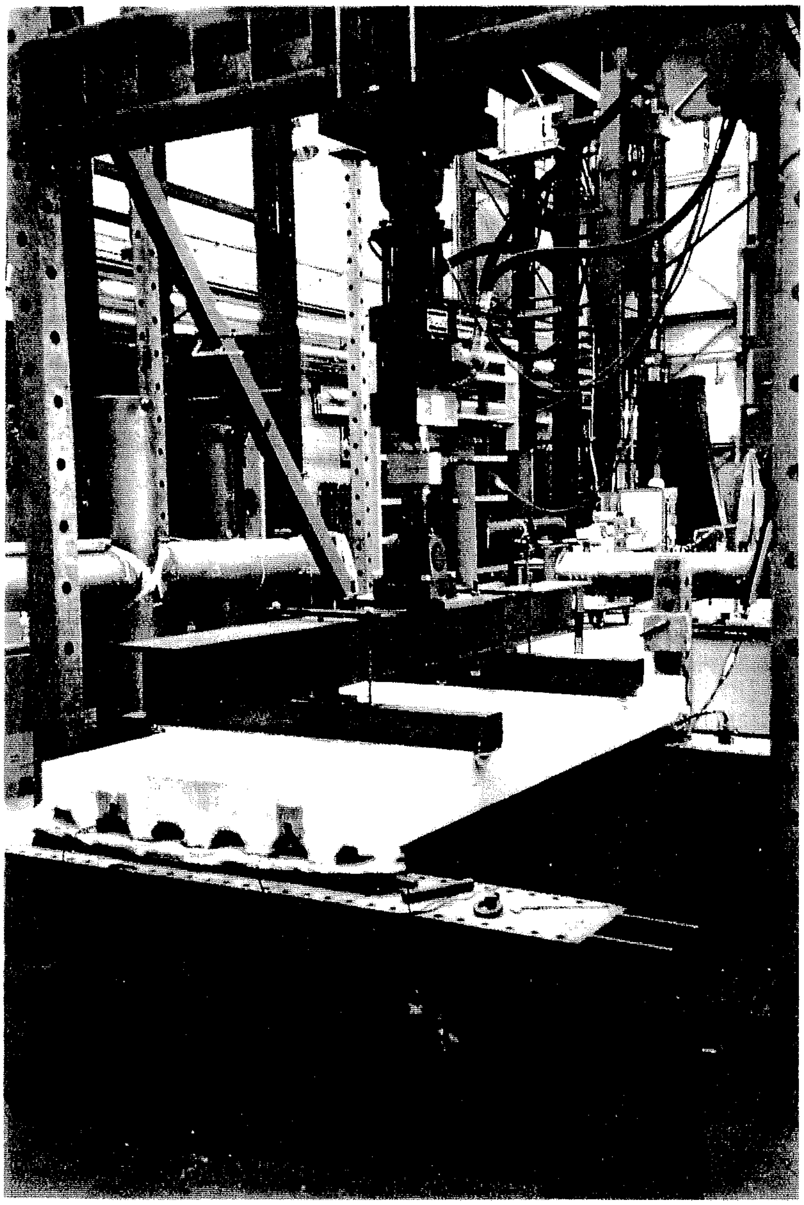


Figure 6: View of specimen in load frame before testing.

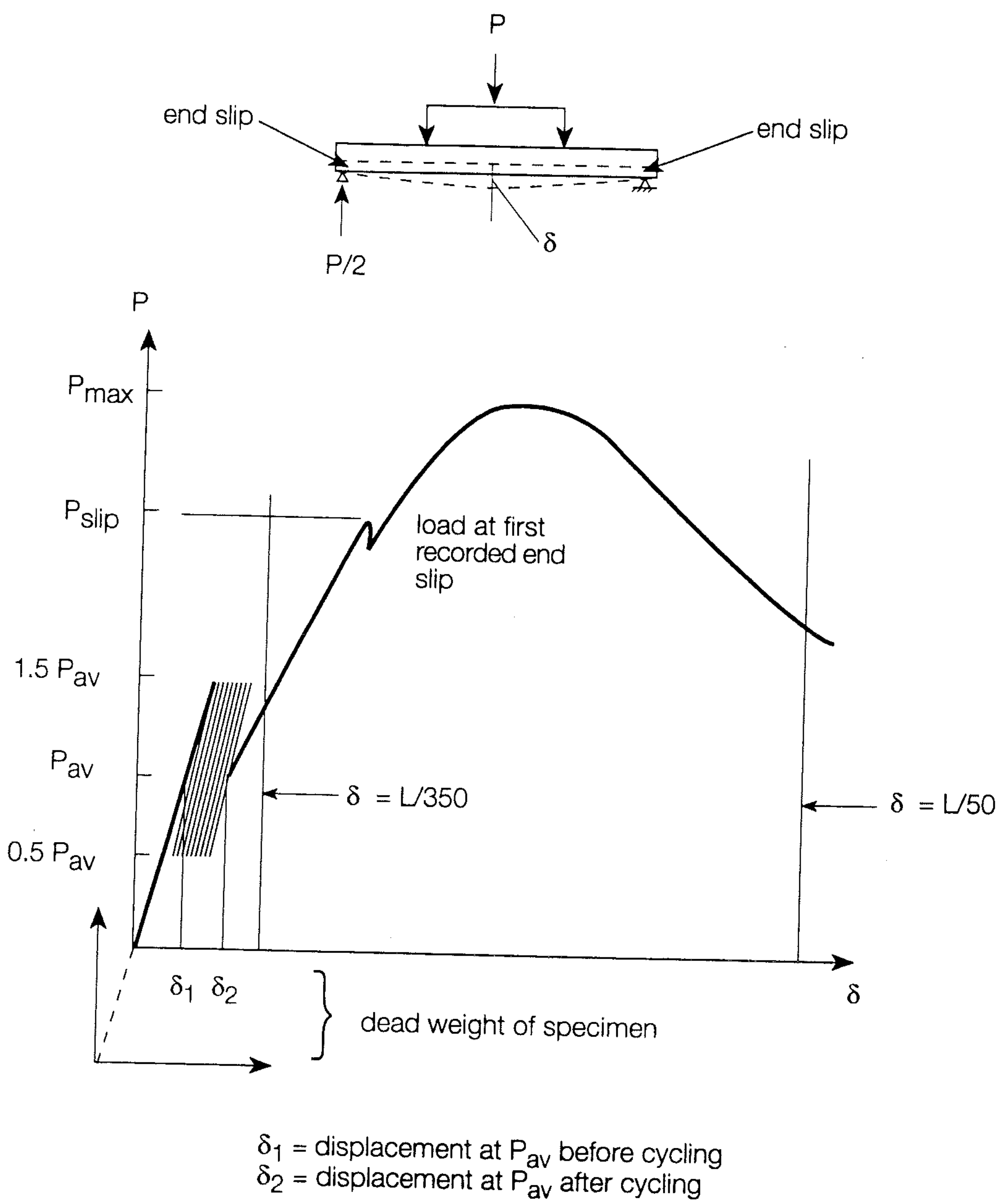


Figure 7: Schematic loading sequence with important events indicated.

4 Recorded data and test results

A summary of important test results is shown in Table 6. Note that the definition of each of the values given in this table is indicated in Figure 7. Graphs of support reaction (excluding self weight) vs. midspan displacement for the specimens with span lengths of 2300 mm and 4800 mm are shown in Figures 8 and 10. Graphs of support reaction (excluding self weight) vs. maximum end slip for the specimens with span lengths of 2300 mm and 4800 mm are shown in Figures 9 and 11. Only the self-weight of the specimens is not included in this table as the load cell was auto-balanced while the loading frame was suspended.

Table 6: Summary of important test results.

Specimen number	P_{max} (*) kN	δ at P_{max} (**) mm	P_{slip} kN	Max. slip mm	P_{av}, cycling kN	δ before cycling at P_{av} (**) mm	δ after cycling at P_{av} (**) mm	Slip during cycling mm
1	149.3	12.65	64.4	8.82	none	-	-	-
2	136.9	11.94	75.1	9.16	60.	4.23	5.26	0.13
3	139.5	11.57	62.0	9.85	none	-	-	-
4	82.3	66.62	74.4	0.10	none	-	-	-
5	84.5	91.01	81.0	0.03	29.0	10.80	12.50	0.00
6	82.0	92.45	74.1	0.08	none	-	-	-
7	135.1	88.49	87.6	0.21	none	-	-	-
8	137.7	94.53	88.3	0.14	29.0 33.6	7.66 10.31	10.00 10.91	0.00 0.00
9	211.5	18.33	102.3	9.86	40.0 82.6	1.62 5.09	2.10 6.25	0.00 0.19

* P, is the total applied load (see Figure 1) excluding the self weight.

** All displacements are due to the applied load only (exclude the displacement under self-weight)

The self-weight of specimens 1,2,3 and 9 is 3.76 kN/ m^2, the corresponding deflection is 0.2 mm. The self-weight of specimens 4,5,6,7 and 8 is 4.71 kN/m^2, the corresponding deflection is 2.7 mm. (Deflections are estimated using n = 18 and uncracked cross-sectional properties).

In Table 7 a summary of important observations noted during the tests are given. In Appendix I photo's of each specimen after failure are shown.

Table 7: Important observations during testing.

Specimen number	Concrete cracking	Other
1,2&3	Major crack under applied load corresponds to first slip. Secondary cracking occurs between applied loads prior to failure.	Applied load reduces rapidly after the observation of the maximum load.
4,5&6	Small flexural cracks occur between applied loads. At failure cracks near the applied loads predominates.	
7,8	Flexural cracks occur between applied loads. Inclined cracks occur between applied loads and supports.	
9	Major crack occurs near applied loads. Significant flexural cracking is observed between applied load before failure.	Failure occurred due to rupture of the reinforcement near an applied load.

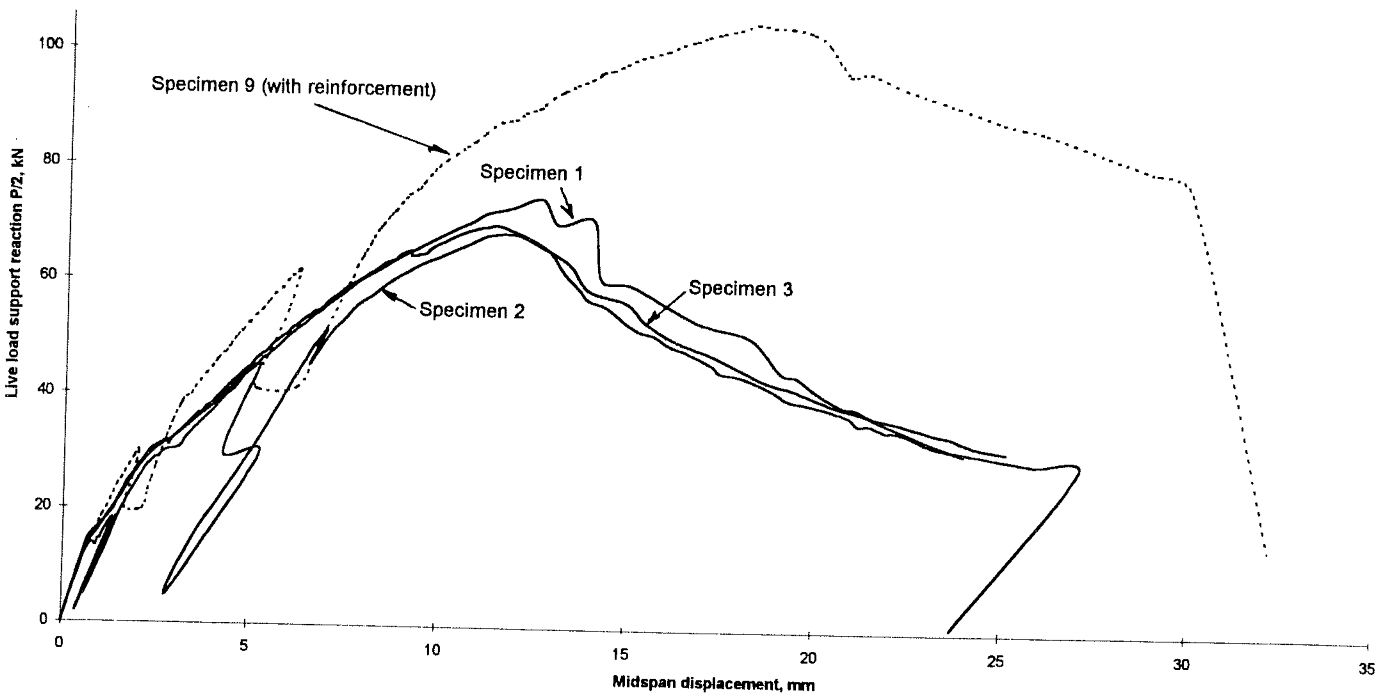


Figure 8: Summary of support reaction (excluding specimen self-weight) vs. midspan displacement for the specimens with span lengths of 2300 mm. Solid line denotes specimens without reinforcement. Dashed line denotes specimens with reinforcement.

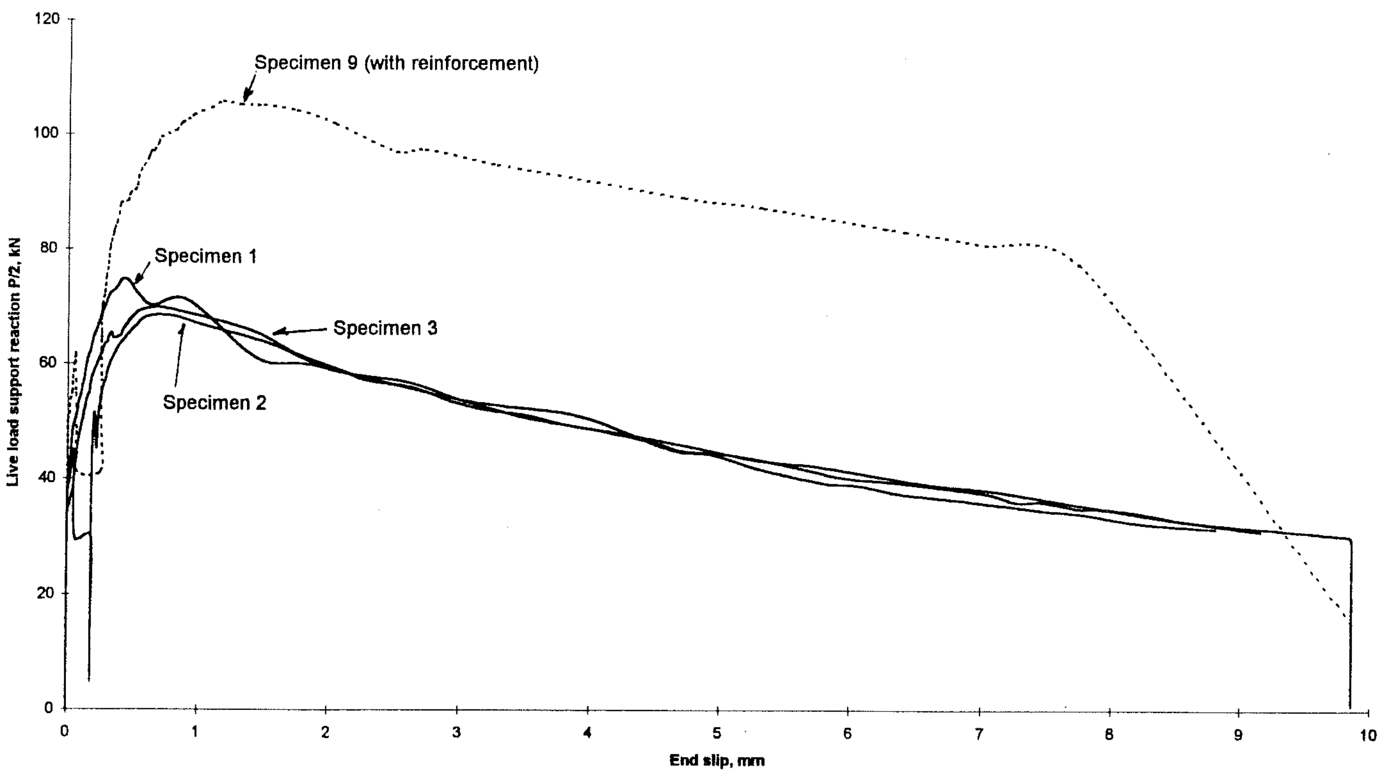


Figure 9: Summary of support reaction (excluding specimen self-weight) vs. maximum end slip for the specimens with span lengths of 2300 mm. Solid line denotes specimens without reinforcement. Dashed line denotes specimens with reinforcement.

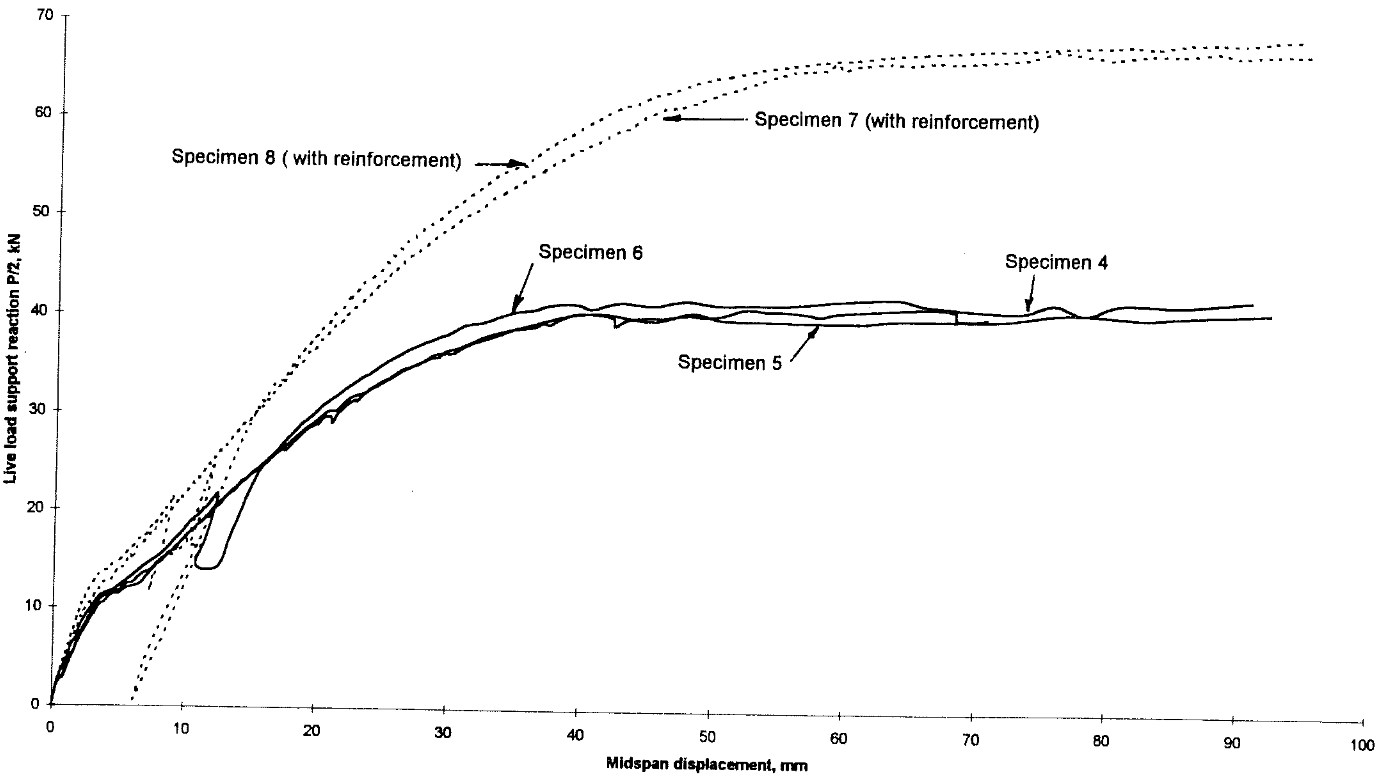


Figure 10: Summary of support reaction (excluding specimen self-weight) vs. midspan displacement for the specimens with span lengths of 4800 mm. Solid line denotes specimens without reinforcement. Dashed line denotes specimens with reinforcement.

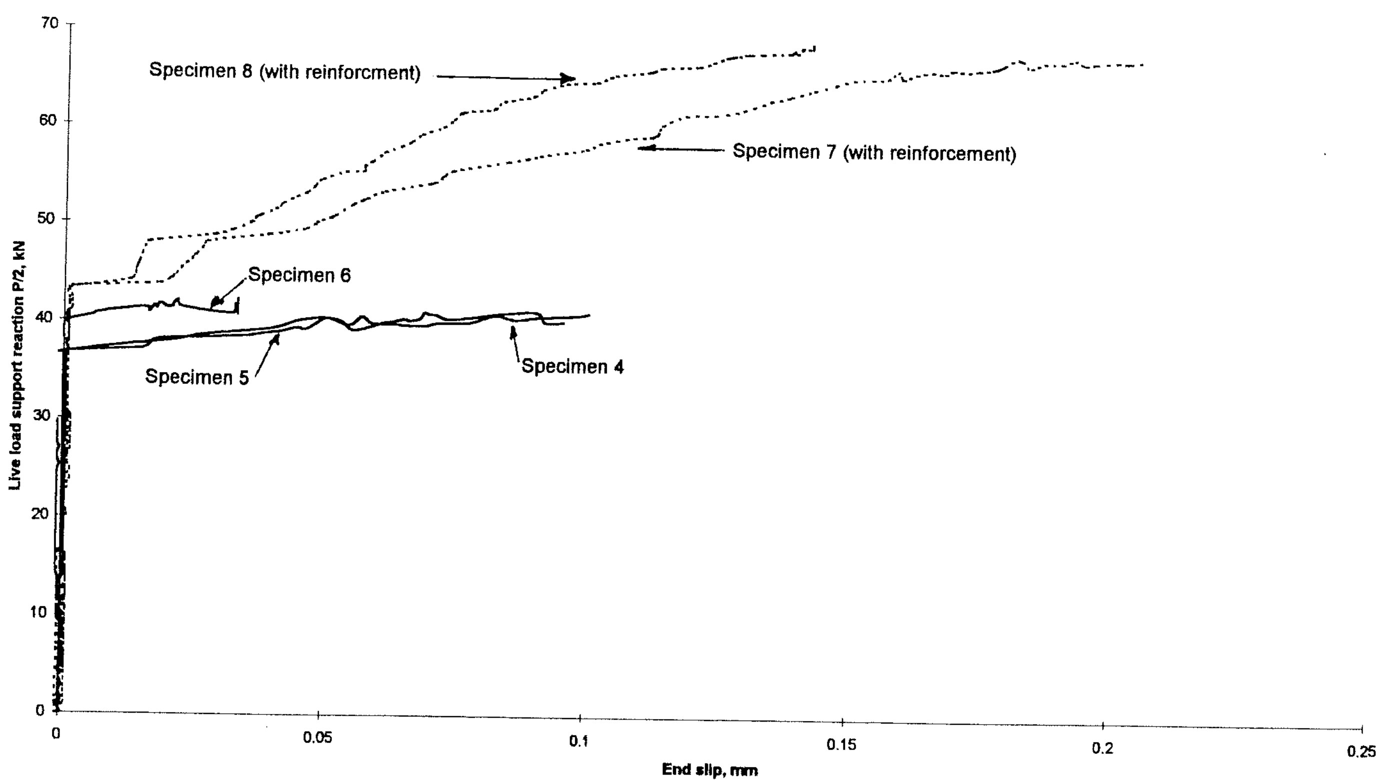


Figure 11: Summary of support reaction (excluding specimen self-weight) vs. maximum end slip for the specimens with span lengths of 4800 mm. Solid line denotes specimens without reinforcement. Dashed line denotes specimens with reinforcement.

5. EVALUATION OF TEST RESULTS

5.1 Introduction

The longitudinal shear bond capacity of the full-scale test specimens are evaluated for the two methods described in the Eurocode 4 [3]. These are referred to here as the m&k-method and τ–method. Longitudinal shear debonding is only one the checks that must be made to determine the ultimate limit state (ULS) strength of a composite slab; other ULS strengths such as vertical shear and punching shear must also be checked but are not within the scope of this report.

The maximum loads that may be applied at the serviceability limit state (SLS) are also evaluated. This evaluation is only concerned with equivalent uniformly applied predominantly static loads, as defined using Eurocode 4 rules.

5.2 ULS evaluation

5.2.1 General

The cross-sectional dimensions given in Table 8 were used to calculate the elastic decking cross-sectional properties given in Table 9. The width of the decking was assumed to by 1086 mm. This gives a measured decking cross-sectional area of (193/.202•1.086 ≈) 1038 mm^2. The nominal cross-sectional area is (193/.202•0.71/0.68 ≈) 998 mm^2. Decking positive moment resistance for the determination of τ_{Rd} was assumed to be 6.9 kNm/1086 mm. For the calculation of allowable loads a reduced value of 4.411 kNm was assumed (see appendix II).

A summary of the input values used to calculate both m&k and τ–values is given in Appendix II. These values are used to calculate allowable uniformly distributed loads for a limited number of span lengths and depths (those tested), and are given in Appendix III.

Table 8: Cross-sectional dimensions used in calculations.

Dimension	Value mm	Dimension	Value mm
Rib width	202	Core thickness	0.677
Bottom flange width	82	Height	60.0
Top flange width	60	Height bottom flange stiffener	6.0
		Width bottom flange stiffener	20.0

Table 9: Calculated cross-sectional properties (decking subjected to sagging moments).

Property	Value
Neutral axis from lower flange y_p	28.8 mm
Cross-sectional area per rib	193 mm^2
Moment of inertia x-x per rib	111'171 mm^4

5.2.2 Calculation of m&k values

A summary of input and intermediate values used to calculate m&k are given in Table 10. For specimen number 3 a coupon test was made. Therefore, for specimen number 3 only the actual value was considered while for the others the mean value was taken into account. This results in the following:

1. Unreduced mean values. The unreduced m&k values, not including partial safety factors, are:
   $m_1 = 214$ $k_1 = 0.07926$

2. Reduced characteristic values. The reduced m&k values, including a factors of 0.9 for the observed test dispersion, are:
   $m_2 = 176.78$ $k_2 = 0.0817$

These values were calculated for use with formula 7.6 of the EC4, which does not including concrete strength:

$$V_{l.Rd} = b\, d_p\, [(m_2\, A_p/\, b\, L_s) + k_2)]/\, \gamma_{vs} \qquad [1]$$

The values of m&k are shown graphically in Figure 12. Note that the reduction factor of 0.9 is placed upon the lowest test result in each group of tests.

Table 10: Input and intermediate values for the m&k analysis.

Specimen number	L_s mm	V_t kN	b mm	d_p mm	A_p mm^2	$V_t/b\ d_p$	$A_p/b\ L_s \bullet 10^{-2}$
1	575	80.060	1126	161.21	1038.5	0.441	0.1610
2	575	73.835	1126	161.21	1038.5	0.407	0.1610
3	575	75.170	1126	161.21	1017.0	0.415	0.1580
4	1200	54.855	1126	201.21	1038.5	0.242	0.0770
5	1200	55.965	1126	201.21	1038.5	0.247	0.0770
6	1200	54.730	1126	201.21	1038.5	0.242	0.0770

5.2.3 Calculation of τ values

A summary of input and intermediate values used to calculate τ are given in Table 11. This results in the following:

1. The unreduced τ–value, not including partial safety factors, is:
   $\tau = 0.241$ N/mm^2

   This value is taken as the lowest τ–value obtained from the group of long-span tests.

2. The reduced τ–value, including the partial safety factor of 1.25 and a factor of 0.9 for the observed test dispersion, is:
   $\tau_{Rd} = 0.241 \bullet 0.9 / 1.25 = 0.173$ N/mm^2

   The value of τ is shown graphically in Figure 13. Note that the reduction factor of 0.9 for tests was used since the dispersion of test results is less than 10%.

Table 11: Input and intermediate values for the τ analysis.

Specimen number	M_u kNm	$M_{test}/ M_{p.Rd}$	L_s+L_o mm	τ N/mm^2
1	41.0	0.86	625	0.408
2	38.0	0.80	625	0.369
3	38.0	0.80	625	0.376
4	59.0	0.98	1250	0.241
5	60.0	1.00	1250	0.246
6	58.0	0.98	1250	0.241

For design purposes τ_{RD} is used with nominal cross-sectional dimensions and material resistances for the calculation of load tables. This has been done and the results are tabulated in Appendix III

5.3 SLS evaluation

Recommended values for vertical deflections of floors are given in Table 4.1 of the Eurocode 3 [4]. These values are repeated here in Table 12 for completeness.

Test results of midspan deflection are used to estimate actual composite slab stiffness under short term loading (n = 8). In accordance with Eurocode 4, the following simple equation is proposed which fits the observed test behaviour:

$$EI_{av} = (EI_{cracked} + EI_{uncracked})/2 \qquad [2]$$

where:

$$EI_{cracked} = 0.5\, E_p\, A_p\, (h - y_p)^2 + 0.5\, E_r\, A_r\, (h - h_r)^2 \qquad [3]$$

A comparison between test results and Equation [2] is given in Table 13. It may be seen that this equation gives a good approximation of long span test stiffnesses both with and without reinforcement. The formula is less accurate for the shorter spans, where deflections are not the critical design criteria. To approximate long-term stiffnesses the stiffness in [2] must be reduced to take concrete creep and shrinkage into account (n = 15). Comparing Table 13 with the Appendix III, it may be seen that only for span lengths of 4800 mm (with reinforcement) small reductions in applied load may be necessary if deflections are important. The values of Appendix III are indicated in table 13 between brackets.

Table 12: Extract from Table 4.1 of the EC3 "Recommended limiting values for vertical deflections.

Condition	Limits	
	δmax (*)	δ2 (**)
Floors generally	L/250	L/300
Floors and roofs supporting plaster or other brittle finish or non-flexible partitions	L/250	L/350
Floors supporting columns (unless the deflections have been included in the global analysis for the ULS)	L/400	L/500

* Maximum deflection including precambering, self weight, creep, shrinkage and service load.

** Maximum deflection due to service loads only.

Table 13: Tested and predicted values of q_{serv}, in kN/m², at two service load deflections L/500 and L/350.

Cross-sectional type and span length	EI_{av}• 10^{11}/m	Test value at (*) :		Equation [2] at (**):	
		δ = L/500	δ = L/350	δ = L/500	δ= L/350
L= 2300 h= 190 w/o Reinf. (Average of specimens 1, 2 and 3)	2.20	32.0	41.2	66.3 (≤16.2)	94.6 (≤16.2)
L= 2300 h= 190 w/ Reinf. (Specimen 9)	2.30	38.6	48.6	69.3 (≤30.9)	99.0 (≤30.9)
L= 4800 h= 230 w/o Reinf. (Average of specimens 4, 5 and 6)	3.42	6.4	8.1	7.3 (≤5.7)	10.4 (≤5.7)
L= 4800 h= 230 w/ Reinf. (Average of specimens 7 and 8)	3.63	8.0	9.8	**7.6** (≤10.8)	10.9 (≤**10.8**)

* $q_{serv} = 2 P / (1.126 L)$, in kN and meters.

** $q_{serv} = 768\ \delta\ EI / (11\ L^4)$; for two point load at L/4, in kN and meters.

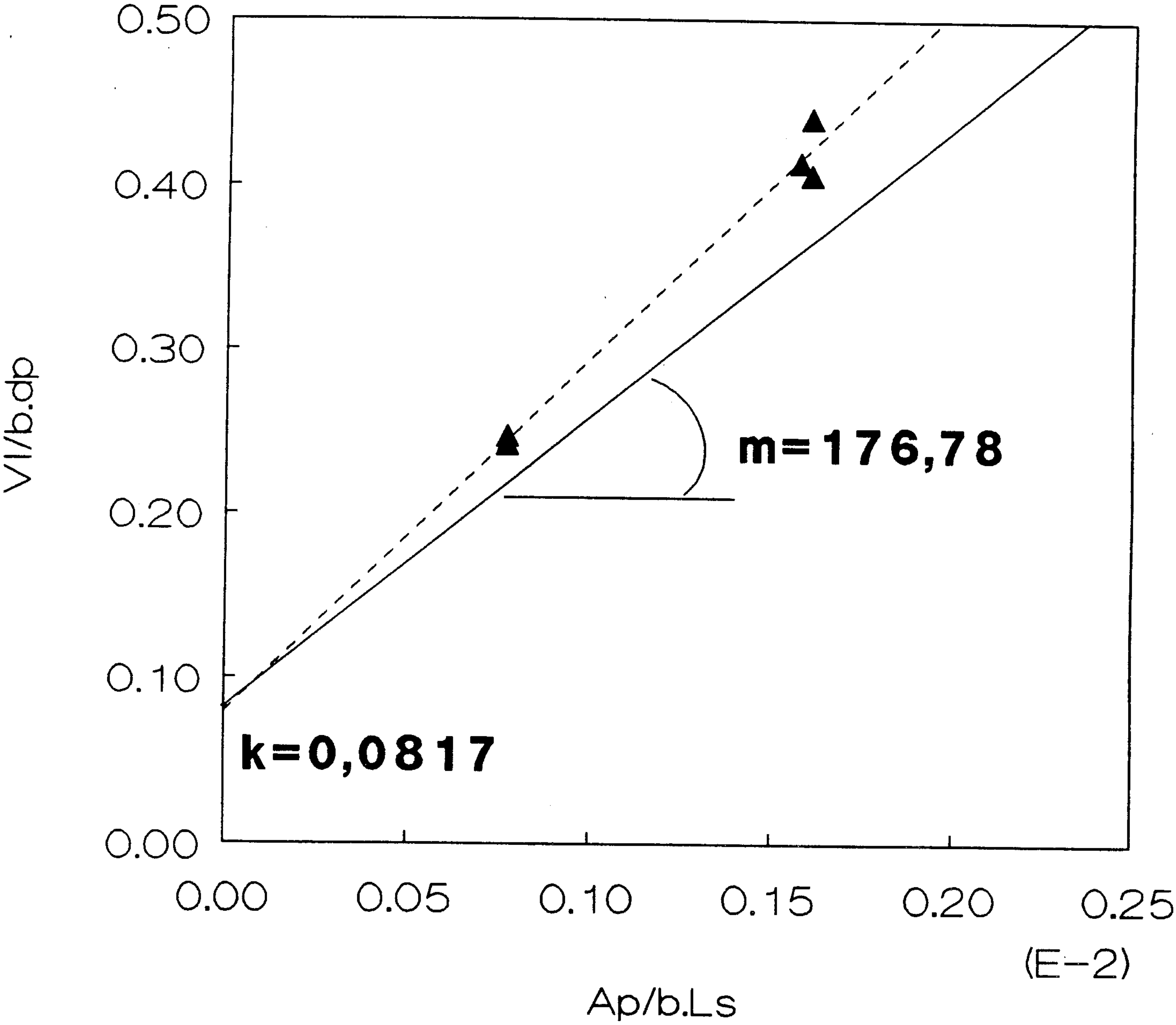


Figure 12: Graphical representation of m&k values.

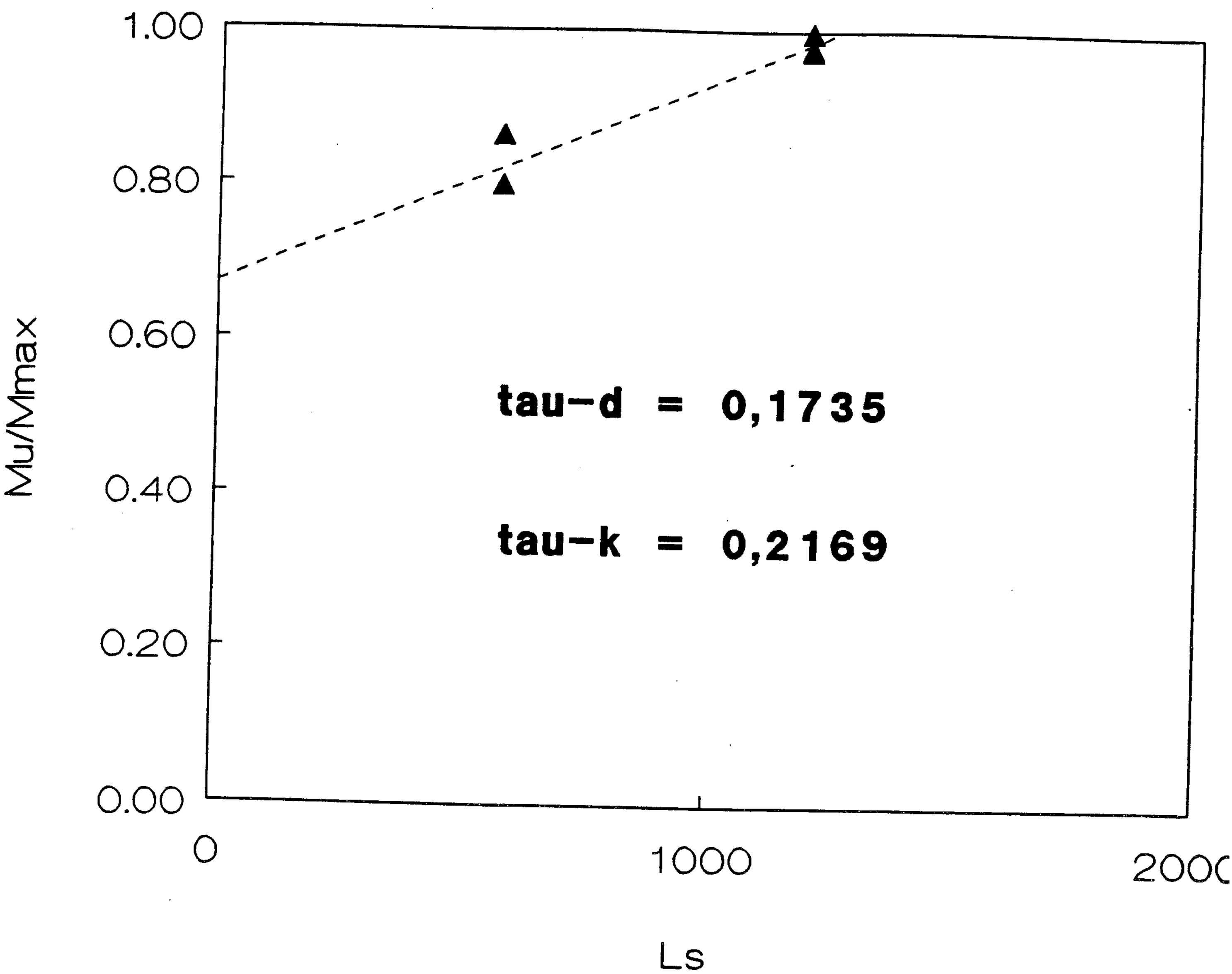


Figure 13: Graphical representation of τ values.

6. SHEAR BOND CAPACITY WITH REINFORCEMENT

Based upon the results of the tests with and without reinforcement, a decision must be made as to whether the reinforcement can be considered to work together with the reinforcement for the range of applications tested. If this can be assumed, then a load table including the reinforcement can be made according to the τ–method of Eurocode 4, and included in Appendix III.

It must be shown that the combined moment capacity from the simply supported tests is at least equal to the design plastic moment capacity:

1. L=2300 mm and h=190 mm

Determine total axial force in test if reinforcement and decking work together;

Axial load in decking, $N = \tau\ b\ (L_s+L_o) = 0.241 \bullet 1126(575 + 50) = 169'604\ N \leq 1038 \bullet 333$

(Note: τ–value is determined from lowest test result)

Axial load in reinforcement, $N_a = 5\ f_{yr}\ \pi\ r^2 = 5 \bullet 565.5\ \pi\ 4.9^2 = 213'277\ N$

Total axial force $= N + N_a = 382'881\ N$

Determine corresponding height of compressive zone in concrete;

Compressive zone height:
$x = (N + N_a)/(0.85\ b\ 0.8f_{cm}) = 382'881/(0.85 \bullet 1126 \bullet 0.8 \bullet 34.2) = 14.6\ mm$

Determine resulting plastic moment capacity of test;
$M_{pl.Rd} = N(h_t - e - x/2) + N_a(h_r - x/2) + 1.25\ M_{pa}(1 - N/(A_p\ f_p))$
$M_{pl.Rd} = 169'604(190 - 28.8 - 7.3) + 213'277(120 - 7.3) + 1.25 \bullet 6.9(1 - 169'604/(1038 \bullet 333))$
$M_{pl.Rd} = 26.1 + 24.0 + 4.5 = 54.6\ kNm/b$

Maximum moment obtained from test;

$M_{test} = M_{dead\ load} + M_{live\ load} = 1/8 \bullet 4.00 \bullet 2.3^2 + 1/8 \bullet 211.5 \bullet 2.3 = 63.5\ kNm/b$

M_{test} is greater than $M_{pl.Rd}$. Therefore the reinforcement and decking can be combined. If the test value of $\tau = 0.369\ N/mm^2$, for specimens of L=2300 mm and h=190 mm is used for this calculation, $M_{pl.Rd}$ would be equal to 65.4 kNm/b. This is approximately equal to M_{test}.

2. L=4800 mm and h=230 mm

Determine total axial force in test in case reinforcement and decking work together;

Axial load in decking, $N = \tau\, b\, (L_s+L_o) = 0.241 \bullet 1126(1200 + 50) = 339'207\ N \leq 1038 \bullet 333$

Axial load in reinforcement, $N_a = 5\, f_{yr}\, \pi\, r^2 = 5 \bullet 565.5\, \pi\, 4.9^2 = 213'277\ N$

Total axial force $= N + N_a = 552'484\ N$

Determine corresponding height of compressive zone in the concrete:

$$x = (N + N_a)/(0.85\, b\, 0.8 f_{cm}) = 552'484/(0.85 \bullet 1126 \bullet 0.8 \bullet 34.1) = 21.2\ mm$$

Determine resulting plastic moment capacity of test;

$M_{pl.Rd} = N(h_t - e - x/2) + N_a(h_r - x/2) + 1.25\, M_{pa}(1 - N/(A_p\, f_p))$

$M_{pl.Rd} = 339'207(230 - 28.8 - 10.6) + 213'277(160 - 10.6) + 1.25 \bullet 6.9(1 - 339'207/(1038 \bullet 333))$

$M_{pl.Rd} = 64.7 + 31.9 + 0.2 = 96.8\ kNm/b$

Maximum moment obtained from test;

$M_{test} = M_{dead\ load} + M_{live\ load} = 1/8 \bullet 4.96 \bullet 4.8^2 + 1/8 \bullet 135.1 \bullet 4.8 = 95.3\ kNm/b$

M_{test} is approximately equal to $M_{pl.Rd}$. Therefore the reinforcement and decking can be combined, using the τ method. This has been used to determine the allowable imposed loads for the span lengths and depths as tested. The results are tabulated in Appendix III.

7. COMPARISON WITH SMALL-SCALE TESTS

The small-scale shear bond push-off tests, described in [1], were used to estimate the maximum τ–value of the decking. A reduction factor of 1.7 taking into account non-linear shear bond was empirically evaluated using data gained from previous experience with similar decking [2]. The results are:

$$\tau_{max,\ push\text{-}off\ tests} = 0.395\ N/mm^2$$

and, with a reduction factor of 1.7:

$$\tau_{full\text{-}scale,\ push\text{-}off\ tests} = 0.232\ N/mm^2$$

The τ–value actually obtained from the full-scale tests is 0.241 N/mm^2. In 5.2.3, for design purpose, this values has been reduced with a partial safety factor of 1.25 and a factor of 0.9 for the observed test dispersion, which resulted in:

$$\tau_{Rd} = 0.241 \bullet 0.9 / 1.25 = 0.173\ N/mm^2$$

8. CONCLUSIONS

This extensive research project has been carried out in order to develop the range of applications in composite floor systems consisting of cast-in place single or continuous slabs.

With the aid of small-scale tests shear-bond characteristics of several decking profiles have been investigated. One has been chosen for further development.

The longitudinal shear-bond capacity of this HODY-SB 60x202x0.75 profile has been verified by means of full-scale tests in accordance with the Eurocode 4.

Totally nine slabs with a span length/slab depth of 2300/190 mm and 4800/230 mm, with and without reinforcement, have been subjected to static and cyclic loading tests.

With the aid of 4800/230 mm specimens an almost fully composite behaviour of slabs has been proven, whereas the 2300/190 mm specimens have shown sufficient ductility.

According to the two longitudinal shear debonding methods the following reduced values, including a factor of 0.9 for the observed test dispersion, have been established:

- The m&k method:

  m = 176.8 en k = 0.0817, which are characteristic values calculated for use with formula 7.6 of the EC4 (not including concrete strength):

  $V_{l.Rd} = b\ d_p\ [(m_2\ A_p/\ b\ L_s) + k_2)]/\ \gamma_{vs}$

- The τ method:
  $\tau_{Rd} = 0.1735\ \text{N/mm}^2$, which is a design value.

The test performed on slabs reinforced with single bars diameter 10 mm per rib have shown that the load capacity of the decking and reinforcement can be combined using the τ–method, described in annex E of Eurocode 4.

The above mentioned test results shall be evaluated for the purpose of the actual design situations.

Since the m&k-values are valid for the tested range only, the τ–method shall be dealt with as more suitable for various practical applications.

A global structural analysis of the behaviour of composite slabs combined with the analysis of test results considering shear-bond capacity of slabs with and without reinforcement may justify wide applications for HODY-SB 60x202x0.75 decking.

Slabs with HODY-SB 60x202x0.75 decking having an overall span length greater than approximately 5200 mm may be assumed to have full composite interaction. Between 4800 and 5200 mm the partial shear connection method can be used. Reinforcement in the ribs at the location of the top flange may be assumed to be fully effective in such slabs. The failure mode due to bending is relevant for these cases.

The design τ-value 0.173 N/mm^2 is valid for all span lengths. The failure mode due to shear debonding is relevant for slabs with the span length less than 5200 mm. Reinforcement in the ribs at the location of the top flange may be assumed to be fully effective in slabs with slab depth of 150 mm and higher and span length of 2.3 m and larger. For shorter spans and slabs less than 150 mm extra reinforcement in the ribs can only be accounted for on the basis of NLE calculations.

The design shear resistance is, in accordance with Eurocode 4, conservative for the shorter spans.

The m&k-method can give higher results, which are accepted within the range tested.

Test results can be used directly for applications with boundary conditions as tested (see EC4 clause 10.3.2 'Specific test')

The aforementioned m&k-method and τ-values may be utilised for design purposes using the nominal cross-sectional dimensions and under condition that the geometry of the actual HODY decking fits within the tolerances given in the Dutch National Application Document.

Furthermore two basic requirements considering materials must be similar to:

- the decking yield strength of S320GD + Z275 (Fe E 320G);
- the concrete grade of C25/30 (Dutch B30); a minimum concrete grade of B25 in practice is acceptable.

Longitudinal shear debonding, however, is only one of the checks that must be made to determine the ultimate limit state (ULS) strength of a composite slab. Other ultimate limit states such as vertical shear and punching shear must also be checked but are not in the scope of this report. The Eurocode 4 shall be followed.

REFERENCES

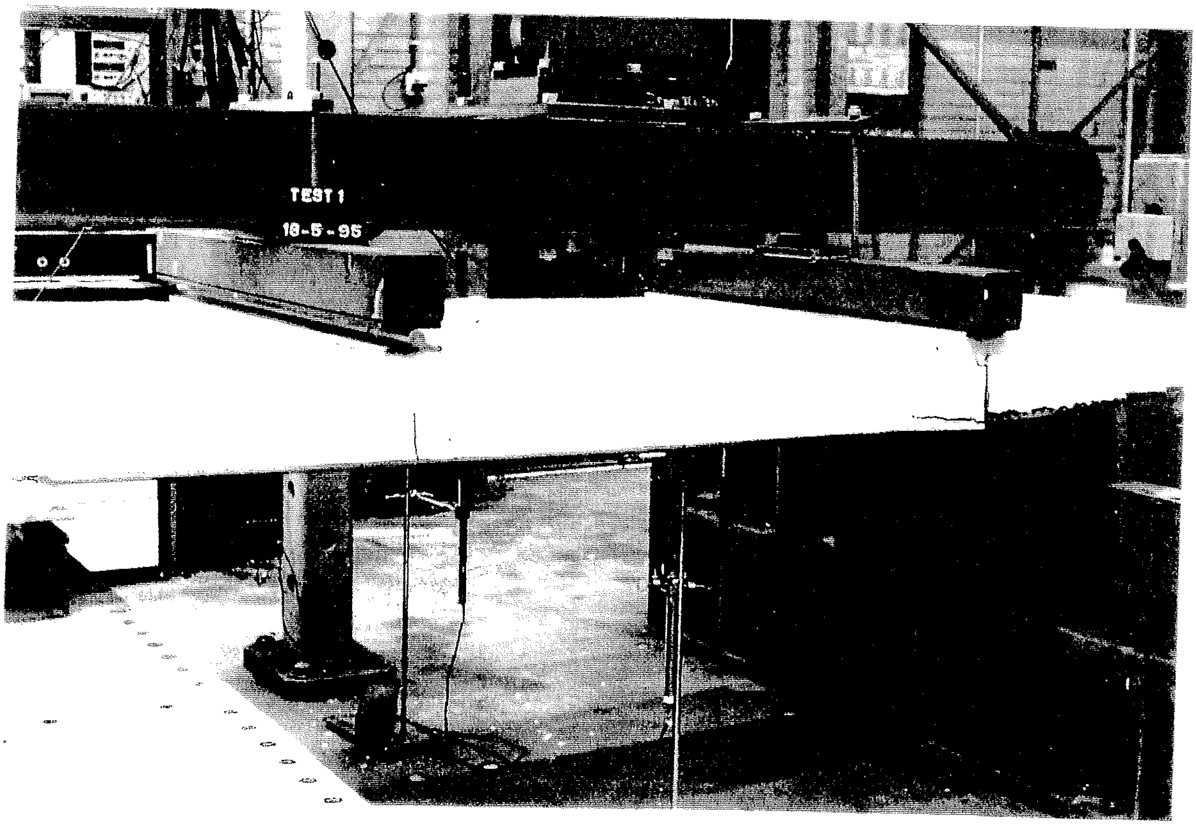
[1] Interim report: HODY composite floor system, Shear bond optimisation of a new composite decking. TNO-report 94-CON-R1312. 14 September, 1994 (not public)

[2] HODY: Phase III. Letter of 2.2.94 (not public).

[3] Eurocode 4; Part 1-1. Design of composite steel and concrete structures, General rules and rules for buildings. October, 1993.

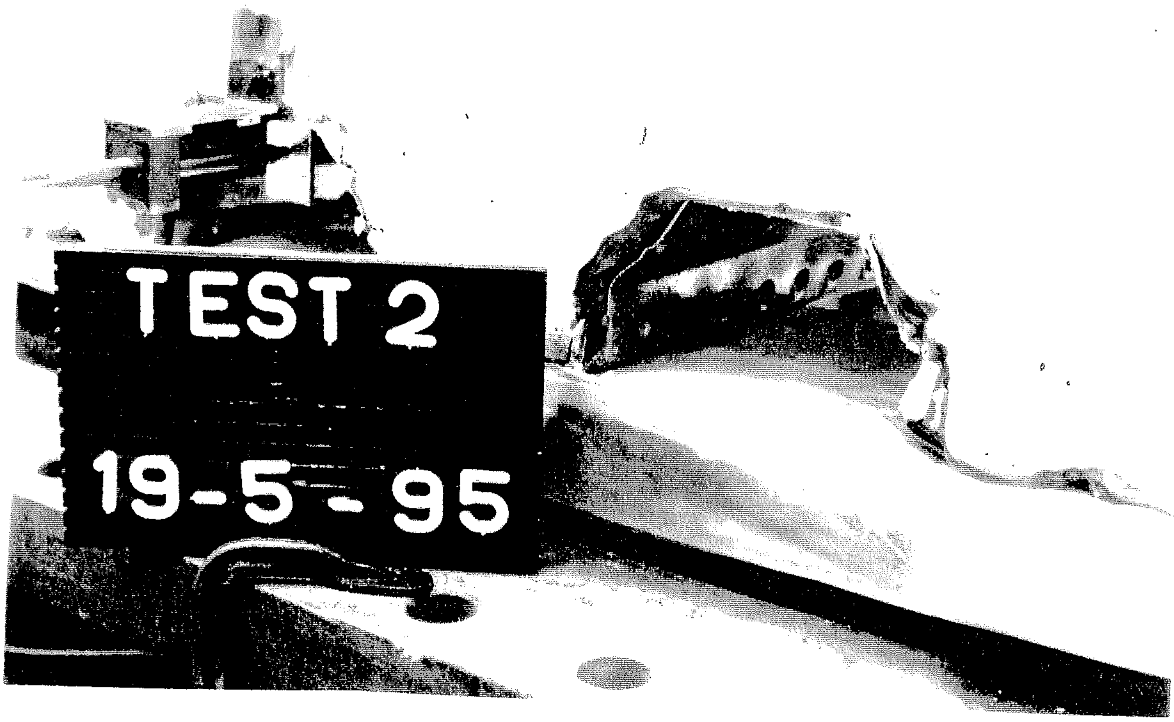
[4] Eurocode 3; Part 1-1. Design of steel structures, General rules and rules for buildings. February, 1992

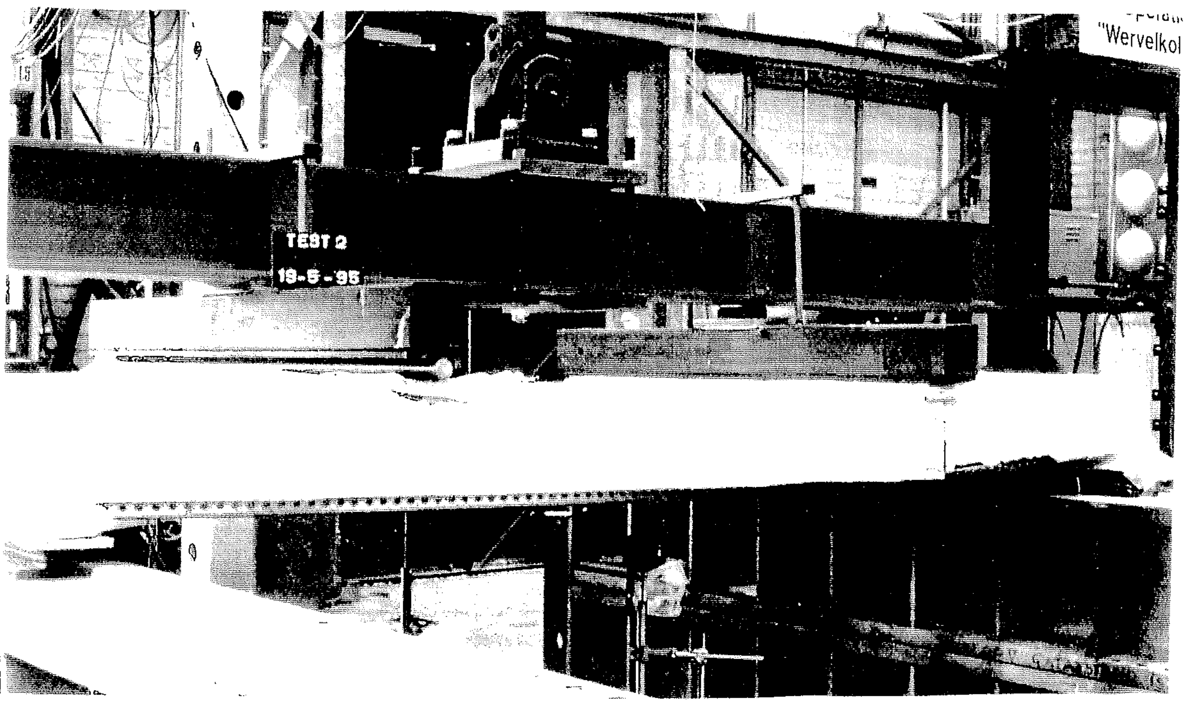
APPENDIX I: PHOTOS OF SPECIMENS AFTER FAILURE



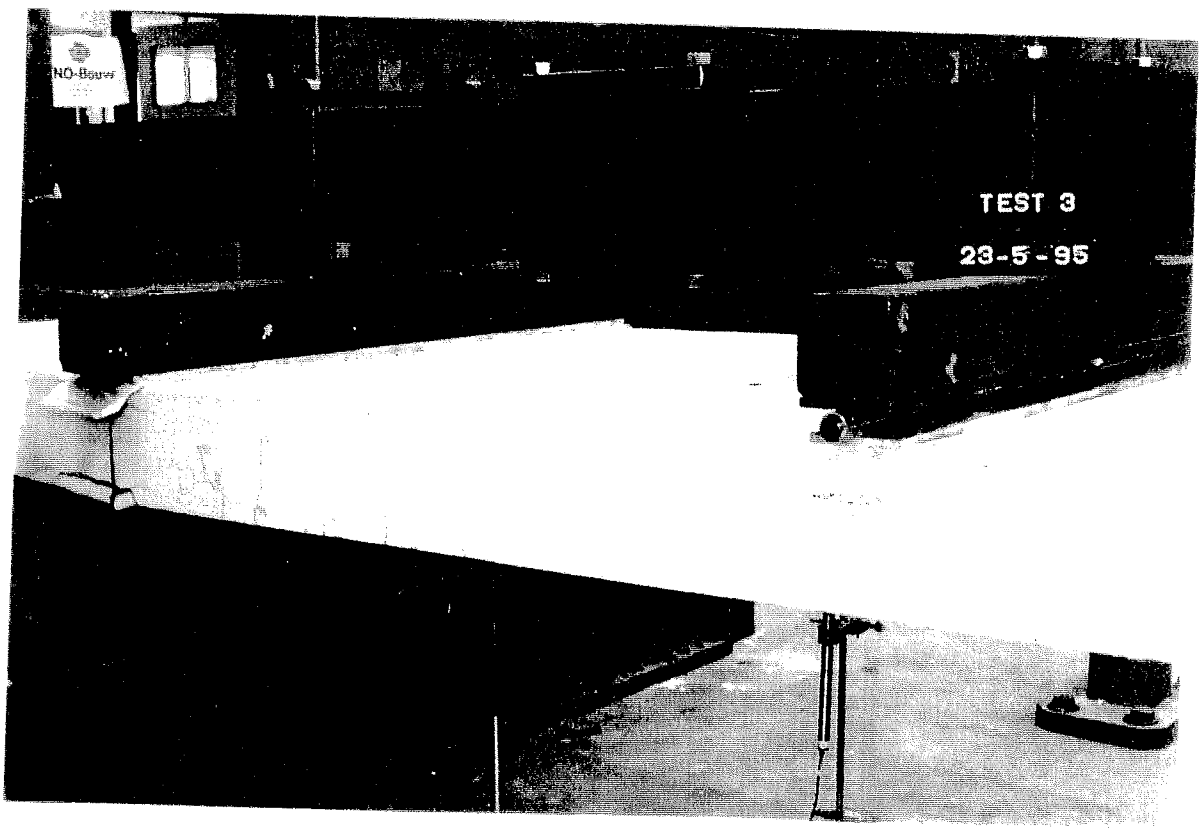
Specimen 1: L = 2300 mm, h = 190 mm w/o reinforcement.

...

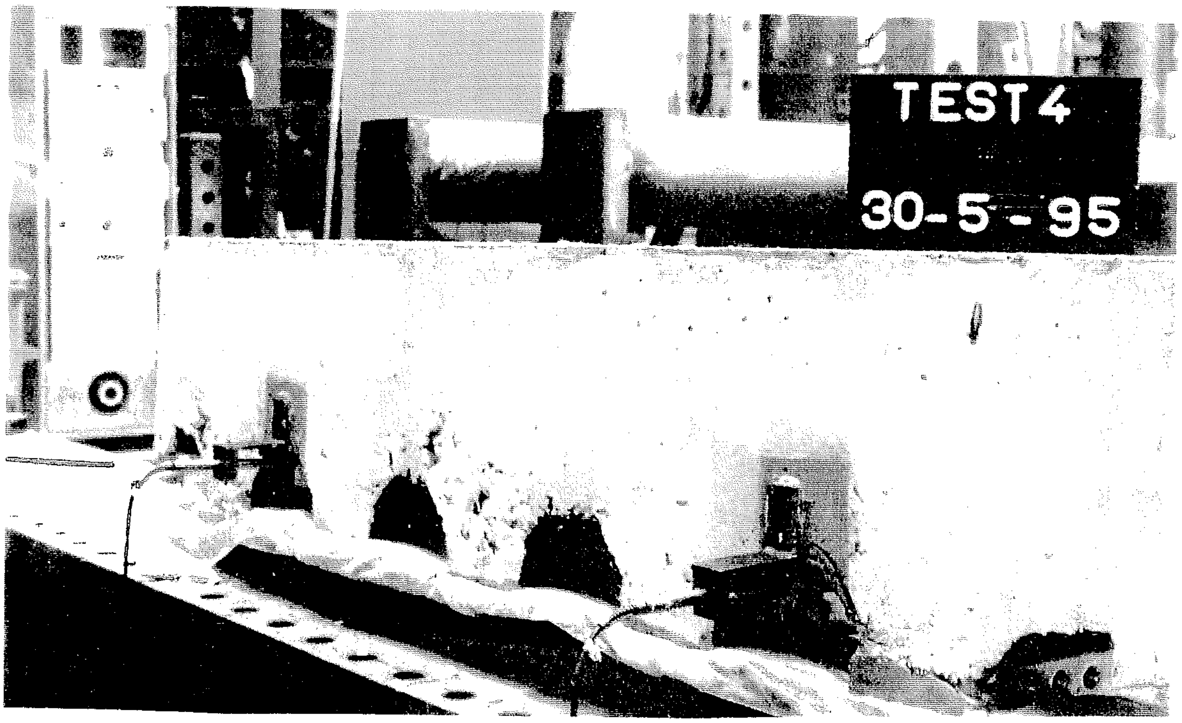


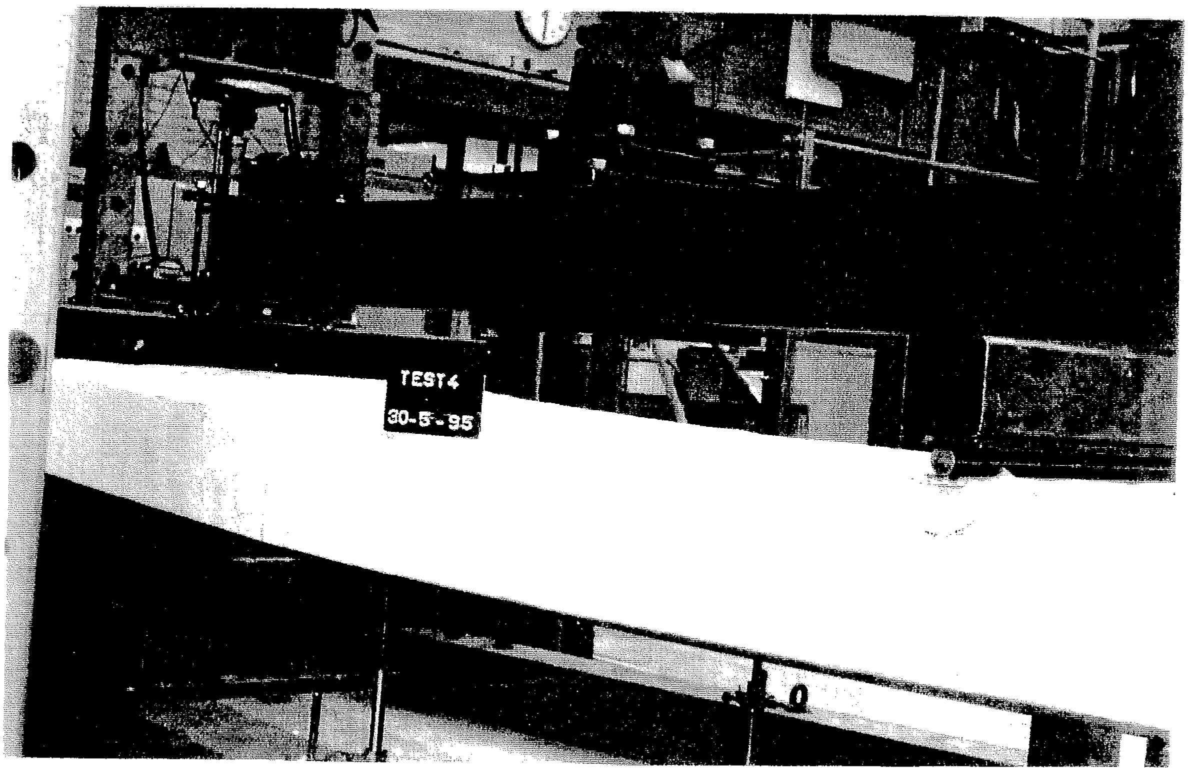


Specimen 2: L = 2300 mm, h = 190 mm w/o reinforcement.

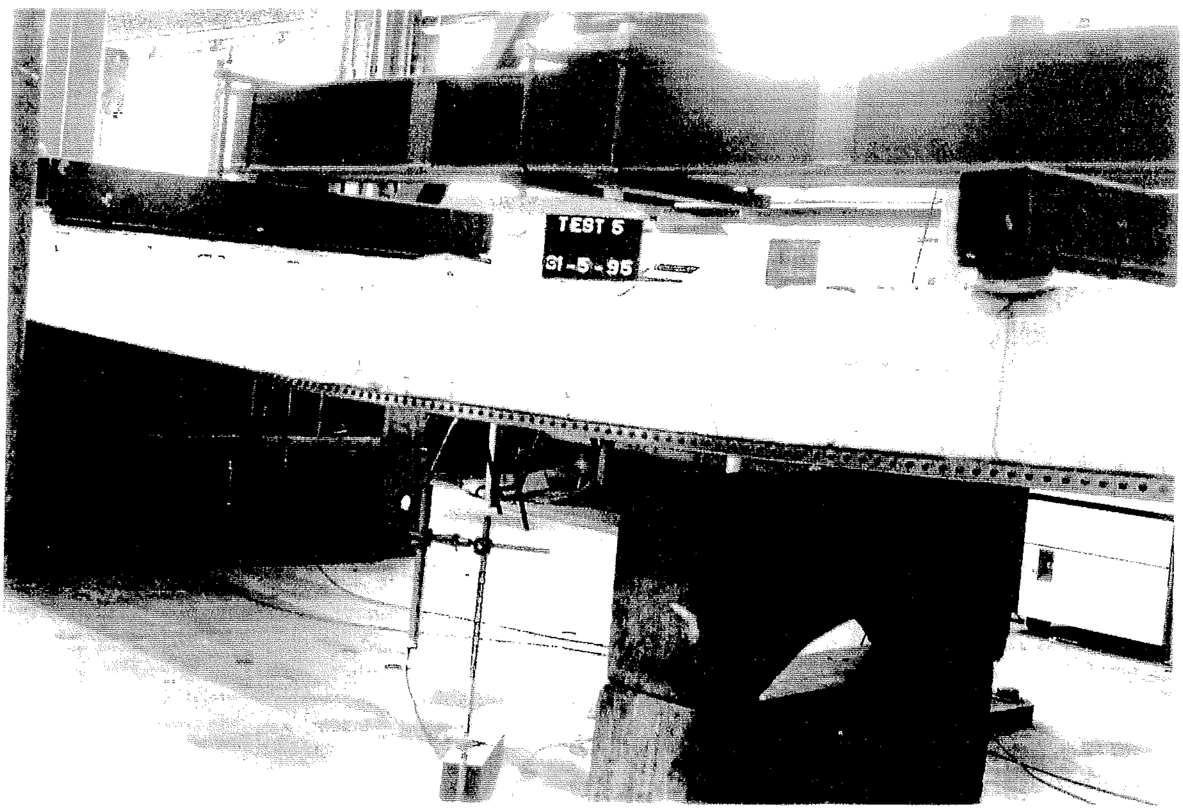


Specimen 3: L = 2300 mm, h = 190 mm w/o reinforcement.

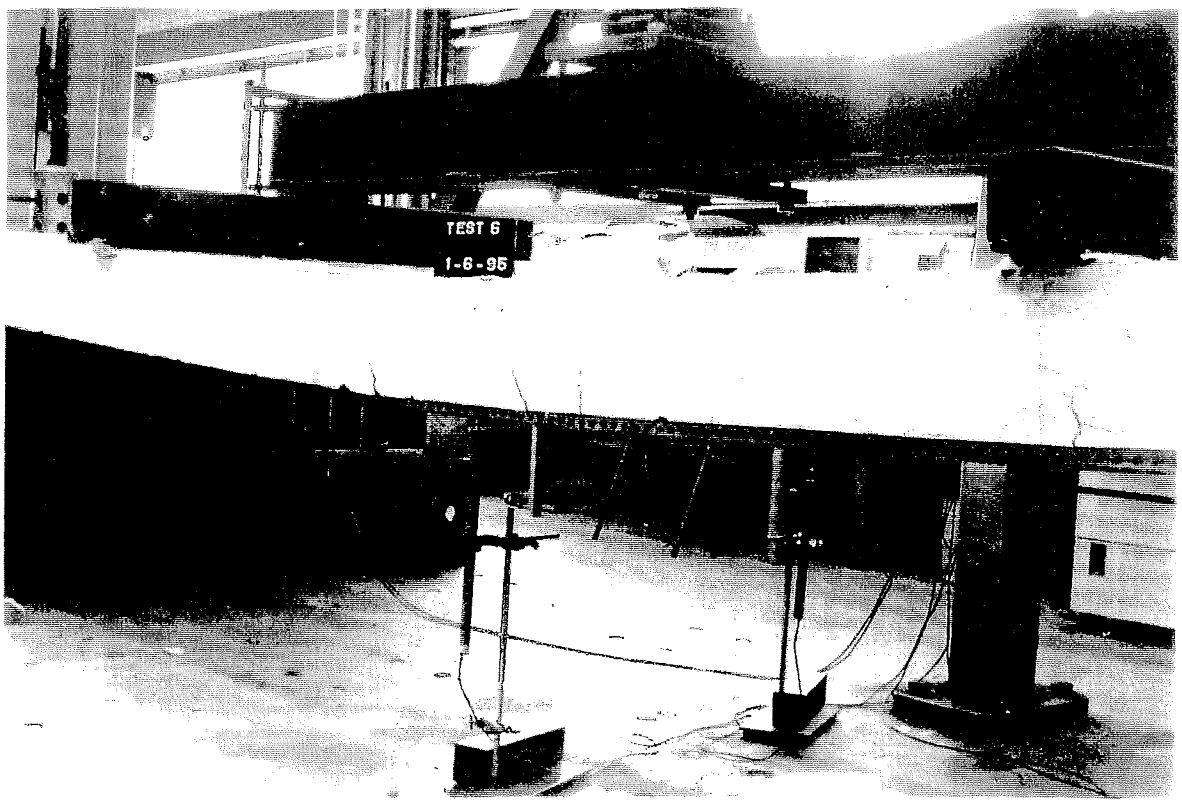




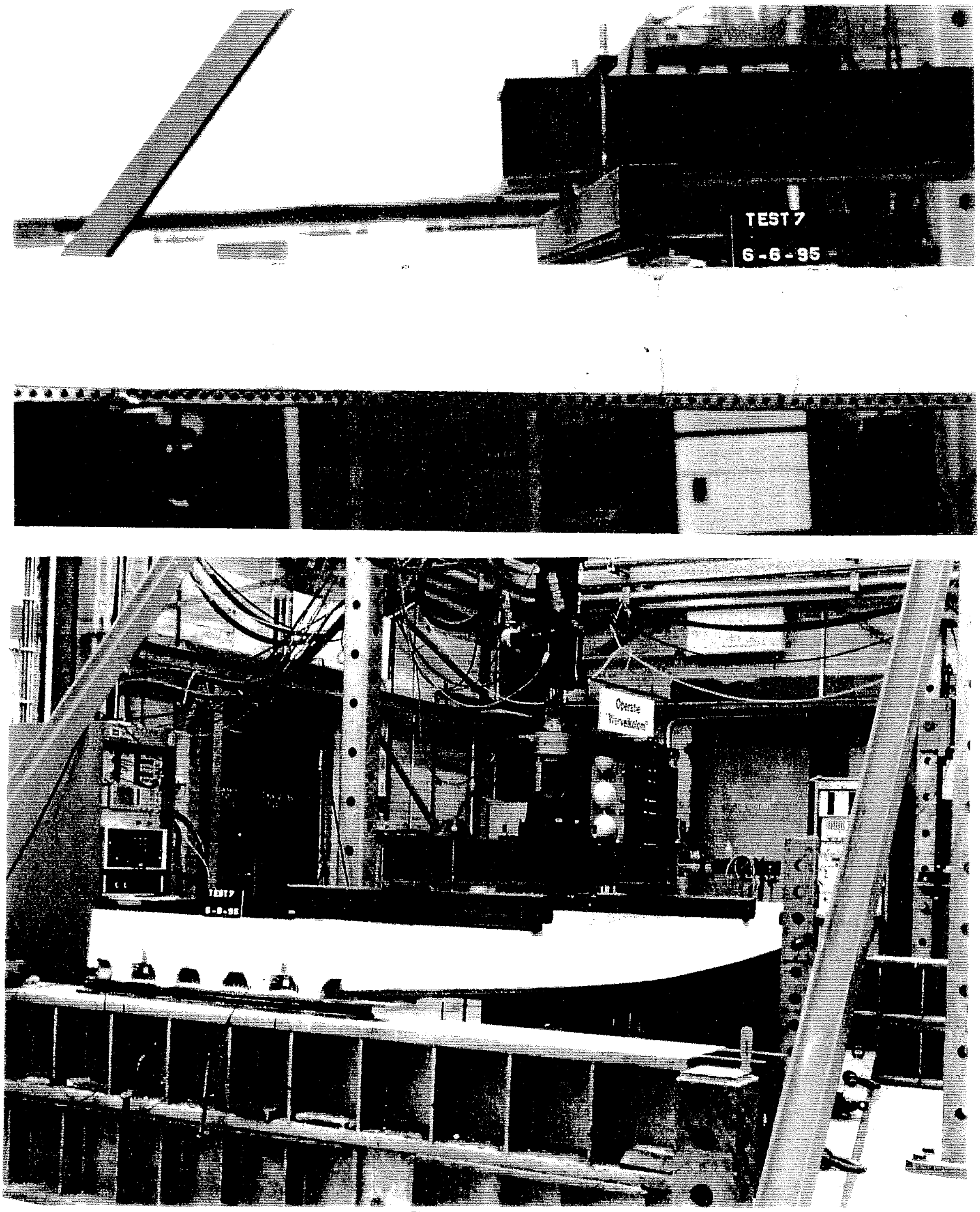
Specimen 4: L = 4800 mm, h = 230 mm w/o reinforcement.



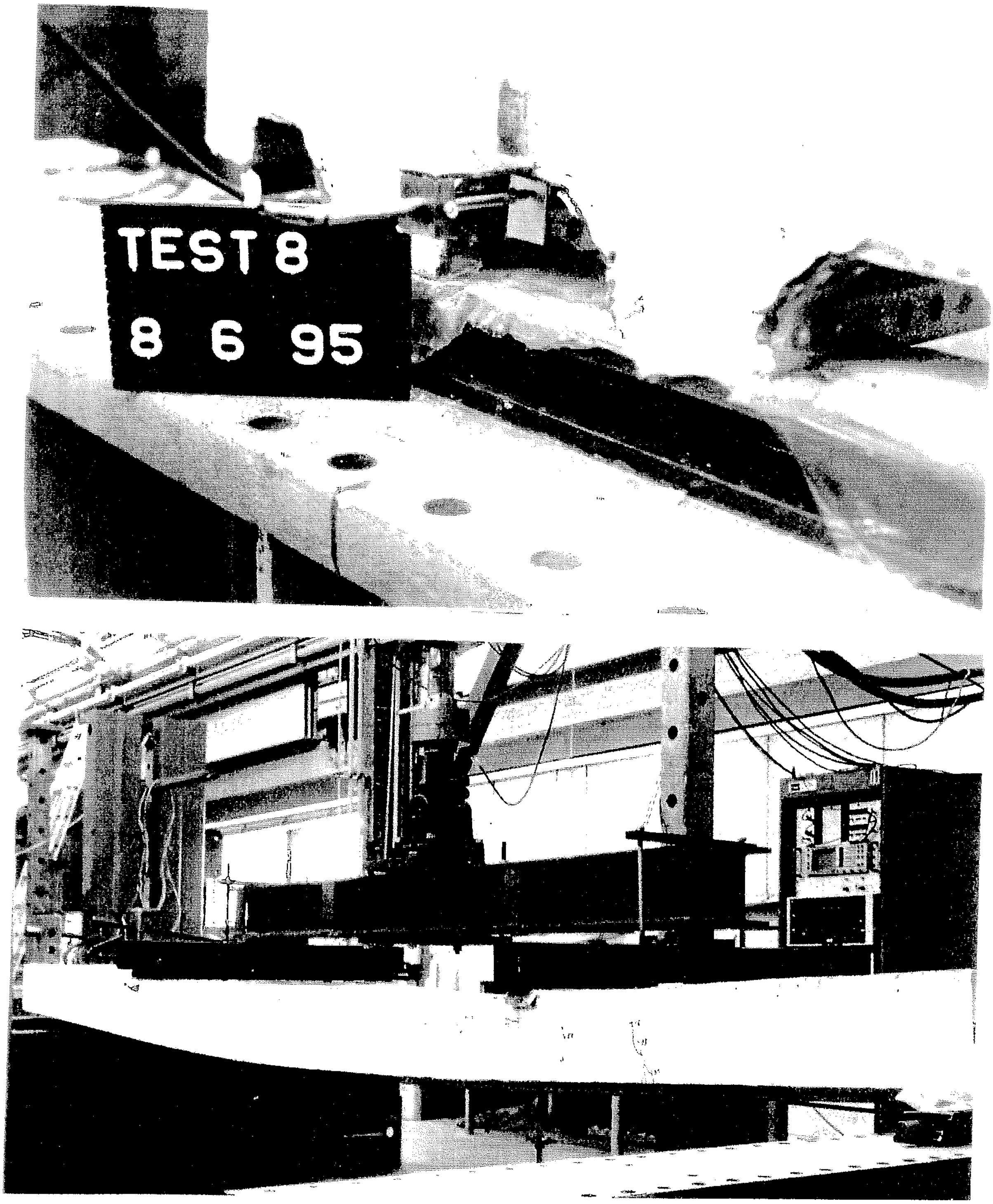
Specimen 5: L = 4800 mm, h = 230 mm w/o reinforcement.



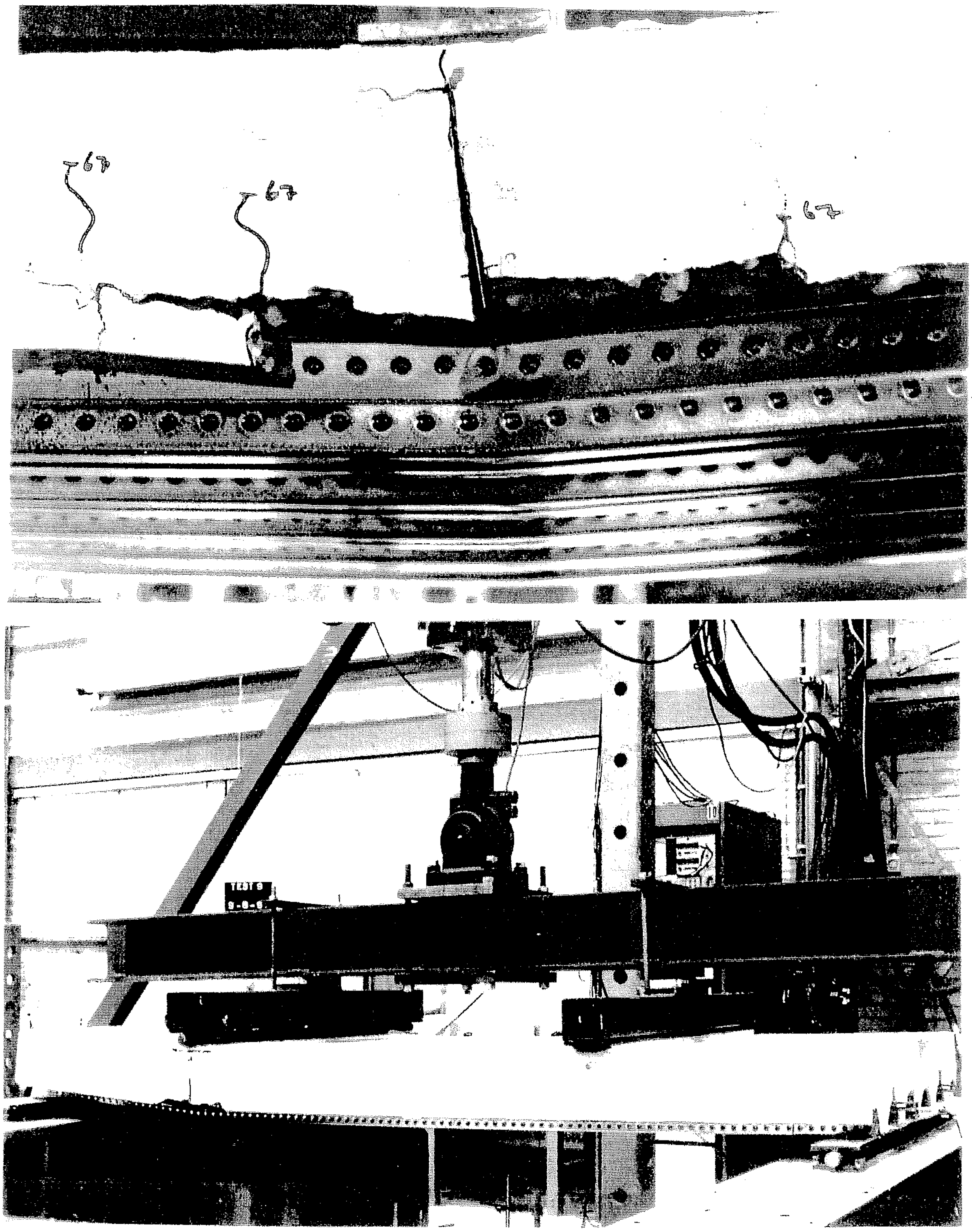
Specimen 6: L = 4800 mm, h = 230 mm w/o reinforcement.



Specimen 7: L = 4800 mm, h = 230 mm with reinforcement.



Specimen 8: L = 4800 mm, h = 230 mm with reinforcement.



Specimen 9: L = 2300 mm, h = 190 mm with reinforcement.

APPENDIX II: SHEAR BOND EVALUATION VALUES

The following values were used to determine the shear-bond capacity of the specimens tested without reinforcement:

Test	No. props	M_{pa} kNm/b	A_p mm^2/b	$f_{cm}(*)$ N/mm^2	q_{dl} kN/m^2	P_{max} kN	L_s mm	L_o mm	b mm	f_{yp}
1	6	6.9	1038.5	31.2	4.00	149.32	575	50	1126	333
2	6	6.9	1038.5	29.7	4.00	136.87	575	50	1126	333
3	6	7.1	1017.0	23.9	4.00	139.54	575	50	1126	346
4	9	6.9	1038.5	26.6	4.96	82.27	1200	50	1126	333
5	9	6.9	1038.5	26.6	4.96	84.49	1200	50	1126	333
6	9	6.9	1038.5	33.0	4.96	82.02	1200	50	1126	333

* Note: the cylinder strengths used in all calculations are $0.8f_{cm}$

APPENDIX III: ABBREVIATED LOAD TABLES

The following tables have been compiled based upon the following assumptions:

$\tau_{Rd} = 0.1735$ N/mm^2
$A_p = 998$ mm^2/meter
$f_{yp} = 320$ N/mm^2
$y_p = 28.8$ mm
$y_r = 70$ mm
$M_{pr} = 4.411$ kNm/meter
$A_r = 389$ mm^2/meter (ϕ10 -202)
$f_{yr} = 500.0$ N/mm^2
$f_{ck} = 25.0$ N/mm^2
$\gamma_c = 1.5$
$\gamma_r = 1.15$
$\gamma_{ap} = 1.0$
$\gamma_{ll} = 1.5$
$\gamma_{dl} = 1.2$

Table III-1: ***$q_{live\ load}$ without end anchorage <u>without</u> reinforcement.***

h (mm)	190	230
$q_{dead\text{-}load}$ (kN/m^2)	4.00	4.96
L= 2300	$M_{max} = 19.2$ kNm $M_{dead\text{-}load} = 2.6$ kNm $q_{live\text{-}load} = 16.2$ kN/m^2	
L = 4800		$M_{max} = 42.1$ kNm $M_{dead\text{-}load} = 14.3$ kNm $q_{live\text{-}load} = 5.7$ kN/m^2

$M_{max} = 1.2\ M_{dead\text{-}load} + 1.5\ M_{live\text{-}load}$

or :

$q_{live\text{-}load} = 8\ [M_{max} - 1.2\ M_{dead\text{-}load}]\ /\ [1.5\ L^2]$

Table III-2: $q_{live\ load}$ without end anchorage <u>with</u> reinforcement.

h (mm)	190	230
$q_{dead\text{-}load}$ (kN/m^2)	4.00	4.96
L = 2300	M_{max} = 33.8 kNm $M_{dead\text{-}load}$ = 2.6 kNm $q_{live\text{-}load}$ = 30.9 kN/m^2	
L = 4800		M_{max} = 63.9 kNm $M_{dead\text{-}load}$ = 14.3 kNm $q_{live\text{-}load}$ = 10.8 kN/m^2

$M_{max} = 1.2\ M_{dead\text{-}load} + 1.5\ M_{live\text{-}load}$

or :

$q_{live\text{-}load} = 8\ [M_{max} - 1.2\ M_{dead\text{-}load}]\ /\ [1.5\ L^2]$

APPENDIX IV: SAMENVATTING VAN HET ONDERZOEK VOOR DE HODY-SB 60x202x0.75 VLOER

De Hody-vloer is een staalplaat-betonvloer die gebruikt kan worden in een groot gebied van vloer-overspanningen.

De Hody-vloer kan worden toegepast in combinatie met wapening.

Door TNO Bouw zijn een tweetal onderzoeken uitgevoerd om bij de veelvoorkomende toepassingsgebieden de randvoorwaarden vast te stellen.

Het eerste onderzoek betrof uittrekproeven op kleine schaal om de τ–waarde vast te stellen, alsmede de kracht ten behoeve van een ingestorte eindverankering.

Het tweede onderzoek is uitgevoerd op ware grootte proefstukken volgens Eurocode 4 met het bijbehorend Nederlandse NAD.

Het onderzoek werd gedaan bij twee verschillende overspanningen met wel en geen lastwisselingen.

De eerste overspanning was 2300 mm bij een totale vloerdikte van 190 mm. Drie proefstukken hadden geen wapening en één proefstuk had 1 rond 10 per golf. De tweede overspanning was 4800 mm bij een totale vloerdikte van 230 mm. Drie proefstukken had-den geen wapening en twee proefstukken hadden 1 rond 10 per golf.

Aan de hand van de uitkomsten van deze proeven konden de volgende randvoorwaarden worden vastgesteld.

Vloeren met een overspanning tot 5200 mm hebben als maatgevend bezwijkpatroon langsafschuiving van de HODY-plaat. De schuifspanningscapaciteit in het rekenstadium bedraagt :

$$\tau_{Rd} = 0.1735 \text{ N/ mm}^2$$

Bij de m en k-methode zijn de karakteristieke waarden:

m = 176.8 en k = 0.0817

Deze waarden zijn berekend met formule 7.6 van Eurocode 4, waarin geen invloed van de betondruksterkte is opgenomen:

$$V_{l.Rd} = b\, d_p\, [(m\, A_p/\, b\, L_s) + k)]/\, \gamma_{vs}$$

Uitgaande van de ontwerpwaarde voor de schuispanning τ_{Rd} volgt uit berekening dat vloeren met een overspanning groter dan 5200 als maatgevend bezwijkpatroon momentbreuk hebben; dus vloeien van de staalplaat. Als kracht in de plaat mag maximaal in het rekenstadium 320 N/mm^2 • 998 mm^2 = 319 kN per strekkende meter breedte worden aangehouden.

Bij het onderzoek is uitgegaan van betonsterkteklasse B30 en staalsoort S320GD + Z275 (FeE320G) voor de geprofileerde staalplaat. In de praktijk dient minimaal de betonsterkteklasse B25 te worden toegepast.

Uit de proeven is gebleken dat extra wapening in de rib ter plaatse van de bovenflens van de geprofileerde staalplaat, voor vloerdikten groter dan 150 mm en overspanningen groter dan 2,3 m volledig mag worden meegenomen bij de bepaling van de momentcapaciteit in de uiterste grenstoestand.

In de gafieken hieronder zij de toepassingsgebieden aangegeven.