PRACTICAL APPLICATIONS OF SHEARBANDS IN NORTH AMERICA AND AUSTRALIA

In this section, practical applications of shearbands in North America and Australia are introduced. Information on the project titles, structural engineers, general contractors and reinforcing bar fabricators is not provided in this paper. Construction was completed in low-to-high seismic regions, including in Camarillo, Fullerton, Glendale Huntington Beach, Oceanside, Oakland, Riverside and San Pedro, CA. All these areas are located in very high seismic zones. Other areas for projects with shearbands include Las Vegas, NV; Toronto, Montreal, Canada; Melbourne, Australia; Atlanta, GA; Dallas, Austin, TX; Orlando, Daytona Beach, Tampa and Fort Lauderdale, FL. Figures 14(a) and 14(b) show pictures of post-tensioned concrete slab-column connections with shearbands during the construction process. One connection had a square column and the other connection had a circular column. The shearbands were all oriented in the same direction (Figures 1 and 14). No vertical legs of the shearband were engaged with the bottom bonded reinforcement. It is shown that the spacing of vertical strips of the shearbands was adjusted while post-tensioning tendons were horizontally curved. The shearbands were placed very close to the column, also covering the slab-column interface. The construction photos appear to show that no obstruction exists between slab flexural reinforcement and shearbands. The shearbands were simply placed on top of the post-tensioning and top slab reinforcement just prior to pouring the concrete.

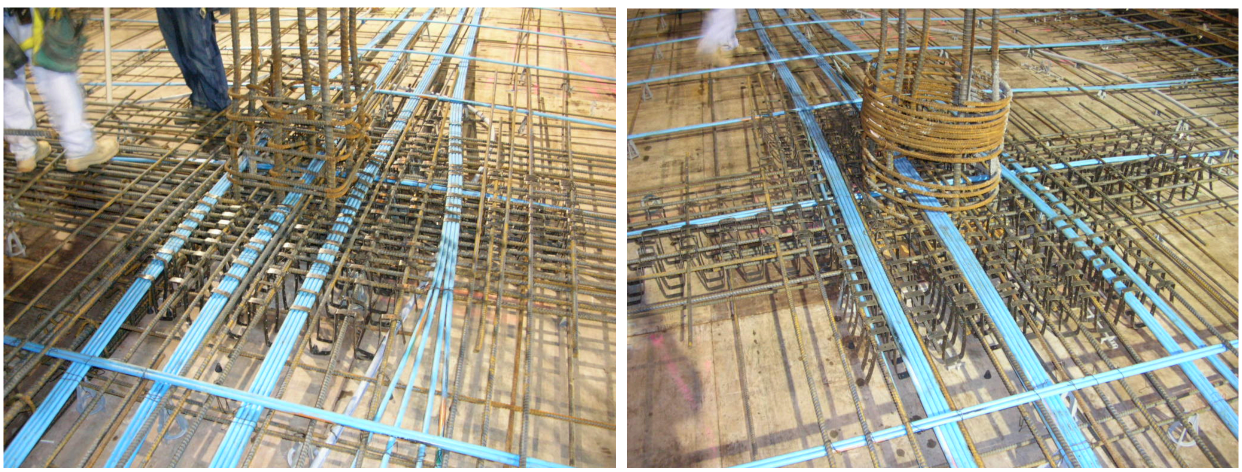

Figure 14 — Post-tensioned slab-column connections with shearbands during the actual construction process.

CONCLUSION

The purpose of this paper was to summarize and reexamine prior and recent test programs on the performance of shearbands in slab-column connections. In particular, special attention was paid to the anchorage details and their performance. Based on the review, the following conclusions are drawn.

1. Overall, shearbands in slab-column connections performed as well or better than comparable headed studs.
2. The shearbands with punched holes can be used as shear reinforcement of slab-column connections assigned to all Seismic Design Categories. The details and placement used in the test programs ensured effective anchorage.
3. Unlike the recommendations for stirrups in R11.12.3 of ACI 318-11, no engagement of vertical elements with slab flexural reinforcement would be necessary. Engaging vertical elements with only a few top bars and placing the top strip of the shearband over the top bars is recommended.
4. The shearbands with the thickness of 0.8 to 3 mm (0.03 to 0.12 in.) and the width of 25 to 30 mm (1 to 1.18 in.) have been used for the test programs and in practice, and the strains in the shearbands exceeded the yield strain or reached three-quarters of the yield strain during the tests without any signs of anchorage failure.

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SHEAR CAPACITY OF SLABS AND SLAB STRIPS LOADED CLOSE TO THE SUPPORT

Eva O. L. Lantsoght, Cor van der Veen and Joost C. Walraven

Synopsis: In reinforced concrete one-way slabs, two limit states related to shear need to be checked: beam shear over an effective width at the support and punching shear on a perimeter around the load. Current code provisions are based on shear tests on heavily reinforced slender beams under point loads. The question remains if these procedures are valid for wide beams and slabs under point loads close to the support. To evaluate the shear capacity of reinforced concrete slabs and the associated effective width, a series of experiments is carried out on eight continuous one-way slabs and twelve continuous slab strips loaded close to the simple and continuous supports. Test results are compared to current code provisions and methods to calculate the shear capacity from the literature. The influence of the shear span to depth ratio, the size of the loading plate and the overall width of the specimen are discussed. From these results follows that the behavior in shear of slabs and beams is not identical. The effective slab width, used for calculating the beam shear capacity, is recommended to be based on load spreading under 45° from the far side of the loading plate towards the support.

Keywords: effective width; experiments; one-way slabs; punching; shear.

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INTRODUCTION

In the Netherlands, 60% of all bridges have been built before 1975. These existing structures have deteriorated over time, while traffic loads and volumes are continuously increasing. After the Laval bridge collapse in Quebec (Wood 2008) and the observation of shear cracks in the webs of a prestressed box-girder bridge, concerns regarding the safety of the existing bridges rose. The Dutch Ministry of Transport, Public Works and Water Management (Rijkswaterstaat) decided to reexamine the existing structures. For an optimal assessment of these structures, a better estimate of the real bearing capacity (all failure modes) is needed. Calculations of design offices based on the current code provisions showed that the loading exceeds a multiple of the shear capacity, while during site checks no signs of distress were found (Walraven 2010). For solid slab bridges, the loading case in which concentrated loads are placed close to the supports was found to be governing for the shear capacity. Due to the larger member width and the transverse reinforcement of slabs as compared to beams for which the one-way shear criteria were developed, a larger shear capacity could be expected for slabs. In this paper, the shear capacity of one-way slabs under concentrated loads close to the support is discussed.

Shear in reinforced concrete one-way slabs loaded with a concentrated load near the support is typically checked in two ways: by calculating the beam shear capacity over a certain effective width of the support and by checking the punching shear capacity on a perimeter around the load. The beam shear (one-way shear) capacity formulas have been derived from experiments on beams under a concentrated load. These heavily reinforced slender beams generally had a total width smaller than their depth so that the web width shows up in the expression for the shear capacity. However, for slabs with point loads near to the support, the beam shear capacity should be calculated taking into account a defined effective width b_{eff}. This effective width represents the width of the support which carries the load. The punching shear (two-way shear) capacity in code formulas was basically developed for two-way slabs. Most empirical methods for punching shear have been derived from tests on slab areas around a column.

Recent research concerning shear in slabs has mainly focused on one-way slabs under line loads (Sherwood et al. 2006; Lubell 2006; Sherwood 2008). It was experimentally shown that one-way slabs under line loads behave like beams and that beam shear provisions led to good estimates of the shear capacity. However, test data regarding the shear capacity of one-way slabs under concentrated loads representing wheel loads are scarce. Even less data (36 reported tests) are available regarding the shear strength of one-way slabs loaded close to the support (shear span to depth ratio $\leq$ 2.5), **Table 1**, in which the following symbols and abbreviations are used:

b	specimen width,
d	effective depth,
a	center to center distance between load and support,
b_{load}	width of the loading plate,
l_{load}	length of the loading plate,
ρ_l	flexural reinforcement ratio,
$f_{c,cyl}$	cylinder concrete compressive strength: for references which include the cube compressive strength, the cylinder compressive strength is calculated as 85% of the cube compressive strength,
V_u	maximum load reached during test,
FM	reported failure mode,
SS	loading near to simple support,
CS	loading near to continuous support,
C	combined failure mode of punching and wide beam shear,
P	punching failure,
WB	wide beam shear failure.

The data in **Table 1** show that the majority of the cited experiments have been carried out on small-scale specimens. No results for slabs with d > 160mm (6.3 in.) are available.

Table 1 — Overview of test data regarding slabs in shear under concentrated loads close to the support ($a/d \leq 2.5$)

Reference	Nr.	b (m)	d (mm)	a/d	$b_{load} \times l_{load}$ (mm × mm)	ρ_l (%)	$f_{c,cyl}$ (MPa)	V_u (kN)	FM
Regan 1982	2SS	1.2	83.5	2.16	100 × 100	0.6	23.0	130	P
	2CS							180	P
	3SS			1.68			30.1	195	P
	3CS							250	WB
	4SS			1.44			35.1	230	P
	5SS			2.16	200 × 100		30.3	190	P
	7SS			1.68			36.7	200	P
	7CS			2.16				230	P
Furuuchi et al. 1998	A-10-10	0.5	160	1.75	100 × 50	2.23	26.1	294	C
	A-10-20						20.2	294	WB
	A-10-30						23.8	333	WB
	A-20-10				200 × 50		19.6	340	-
	A-30-10				300 × 50		23.8	450	-
	B-10-10	0.65			100 × 50	2.29	29.4	368	-
	C-10-10	0.5	160	1.25	100 × 50	2.23	34.6	480	WB
	C-20-10				200 × 50		32.1	525	WB
	C-30-10				300 × 50		31.5	626	WB
	C-50-10				500 × 50		34.9	811	WB
	C-10-20				100 × 50		36.4	483	-
	C-10-30				100 × 50		30.7	520	-
	D-10-10			2.25	100 × 50		35.2	294	-
Graf 1933	1243 a_1	2	115	1.30	100 × 150	0.65	19.1	314	WB
	1243 a_2			2.17				235	C
	1243 b_1			0.65				355	P
	1243 b_2			1.52				206	WB
	1244 a_1		104	1.92		1.14	13.3	275	WB
	1244 a_2			2.40				196	WB
	1244 b_1			1.68				157	WB
	1244 b_2			2.16				147	WB
	1245 a_1	2.4	106	1.89		1.52	23.6	333	C
	1245 a_2			2.36				257	WB
	1245 b_1			1.65				196	C
	1245 b_2			2.12				206	C
Richart & Kluge 1939	2-2	6.1	140	1.64	150 (disc)	0.91	29.1	369	C
Leonhardt & Walther 1962	P12	0.5	142	2.46	80 × 80	0.95	12.6	101†	WB
Ekeberg et al. 1982	2nd fl nr. 3	5	108	2.18	100 × 100	0.52	17.8	465	-

Note: 1m = 3.3ft, 1mm = 0.04 in, 1MPa = 0.145 ksi, 1kN = 0.225 kip.
-: Photographs or a description of the failure mode were not provided.
†: self weight is reported to be included in the value of the peak load.

Regan (1982) suggested that slabs under point loads close to the support have a higher shear capacity than expected based on beam shear formulas. To investigate this claim and quantify the real bearing capacity of solid slab bridges, an extensive experimental program is carried out at Delft University of Technology, The Netherlands. The research is limited to non-prestressed slabs. The variables in the test program are:

- the shear span to depth ratio (a/d),
- the reinforcement layout,
- the concrete compressive strength,
- the position of the load along the width,
- the size of the loading plate,
- the type of support (simple or continuous),
- the overall width of the specimen,
- the type of reinforcement (ribbed or plain bars),
- the support conditions (line support or bearings).

RESEARCH SIGNIFICANCE

Only a small number of results from shear tests on one-way slabs subjected to concentrated loads close to the support is available. The experimental data presented in this paper extend the test data and knowledge on slabs under concentrated loads failing in shear. These data allow for a comparison with the test data of beams failing in shear. Indications are found that shear behavior in one-way slabs under concentrated loads is different from that in beams. The outcome of this research will be used to assess existing slab bridges in the Netherlands.

ONE-WAY SHEAR PROVISIONS

The one-way shear capacity of slabs is calculated using the beam shear formulas over a certain effective width (b_{eff}). The method of load spreading, resulting in the effective width, depends on local practice. In most cases, such as in Dutch practice, load spreading is assumed under a 45° angle from the center of the load towards the support, **Figure 1(a).** The lower limit for the effective width is taken in Dutch practice as $2d$ for loads in the middle of the width and d for loads at the edge and corner of the slab. In French practice, load spreading is assumed under a 45° angle from the far corners of the loading plate towards the support, **Figure 1(b).**

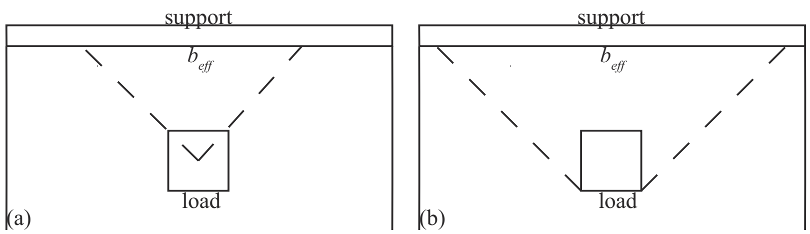

Figure 1 — Effective width (a) assuming 45º load spreading from the center of the load; (b) assuming 45º load spreading from the far corners of the load; top view showing concentrated load and support.

According to EN1992-1-1:2005 §6.2.2 (1) the maximum shear force for a section without stirrups is calculated as follows (SI units, $k_1 = 0.15$):

$$V_{Rd,c} = \left[C_{Rd,c}k\left(100\rho_l f_{ck}\right)^{1/3} + k_1\sigma_{cp}\right]b_w d \geq (v_{\min} + k_1\sigma_{cp})b_w d \tag{1}$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \tag{2}$$

with

f_{ck} the characteristic concrete cylinder compressive strength in MPa (1MPa = 0.145ksi),
d the effective depth in mm (1 mm = 0.04in.),
σ_{cp} the average normal concrete stress over the cross section, positive in compression,
b_w the smallest width of the cross section in the tensile area, not to exceed b_{eff}.

Eq. 1 applies for members with or without prestressing.

The values of $C_{Rd,c}$ and v_{min} depend on the National Annex. It is recommended to take $C_{Rd,c} = 0.18/\gamma_c$ where γ_c is the partial factor for concrete (γ_c=1.5 in general) and v_{min}:

$$v_{\min} = 0.035k^{3/2}f_{ck}^{1/2} \tag{3}$$

In the French National Annex (Chauvel et al. 2007) a different approach is used for v_{min}. For slabs benefiting from transverse redistribution under the load case considered v_{min} is defined as:

$$v_{\min} = 0.34 f_{ck}^{\ 1/2} \tag{4}$$

and for beams and slabs, other than those described by eq. 4, the expression becomes

$$v_{\min} = 0.053k^{3/2}f_{ck}^{1/2} \tag{5}$$

Loads applied within a distance $0.5d \leq a_v \leq 2d$ from the edge of a support can be reduced by $\beta = a_v/2d$, where a_v is the clear shear span from the face of the load to the face of the support.

According to ACI 318-08 the inclined cracking load is described by formula (11-5) from §11.2.2.1. The sectional shear force is not allowed to exceed the inclined cracking load for elements without web reinforcement. For normal weight concrete (λ = 1) and with notations altered to facilitate comparison with the previously used SI notations, the expression becomes:

$$V_c = \left(0.16\sqrt{f_{ck}} + 17\rho_l \frac{V_{ACI}d}{M_{ACI}}\right)b_w d \leq 0.29\sqrt{f_{ck}}b_w d \tag{6}$$

with

f_{ck} the characteristic concrete cylinder compressive strength in MPa (1MPa = 0.145ksi),
ρ_l the flexural reinforcement ratio,

d the effective depth,
b_w the width of the web, not to exceed b_{eff}.
V_{ACI} factored shear force at a section,
M_{ACI} factored moment at a section.

Regan's method (1982) was developed especially for slabs under point loads close to the support ($a_v \leq 2d$; where a_v is the clear shear span from the face of the load to the face of the support) based on the punching provisions from the British Code of Practice CP110 (Technical Committee CSB/39 1982) and the results of small-scale experiments (reported in **Table 1**). The method is based on the definition of a critical perimeter around the concentrated load, **Figure 2.** For loads close to the free edge the method is extended based on the principles used when determining the punching perimeter, **Figure 2(c, d).** As shown in **Figure 2** the support is subdivided into different parts of the perimeter u_1 and u_2. Likewise, the contributions to the capacity of u_1 and u_2 (influenced by the nearby support) are accounted for differently. The resistance of the part of the perimeter parallel to the support u_2 (**Figure 2**) is defined as:

$$P_{R2} = \left(\frac{2d}{a_v}\right)\xi_s v_c u_2 d < \frac{\sqrt{f_{cu}}}{\gamma_m} u_2 d \tag{7}$$

$$\xi_s = \sqrt[4]{\frac{500}{d}} \tag{8}$$

$$v_c = \frac{0.27}{\gamma_m}\sqrt[3]{100\rho f_{cu}} \tag{9}$$

in which
f_{cu} the cube concrete compressive strength, in MPa (1 MPa = 0.145 ksi),
γ_m the partial safety factor for materials.

In eq. (8), d needs to be taken in mm (1mm = 0.04in). The resistance of the remainder ($\Sigma u = u_1$) of the perimeter (**Figure 2**) is defined as:

$$P_{R1} = \sum \xi_s v_c u d \tag{10}$$

For the sections parallel to the support, the longitudinal properties are to be used (d_l and ρ_l of the longitudinal reinforcement), similar to a calculation for beam shear. For the sections perpendicular to the support, the transverse properties are to be used (d_t and ρ_t of the transverse flexural reinforcement), as indicated in **Figure 2.** The contributions of P_{R2} from eq. (7) and P_{R1} from eq. (10) are then summed to give the total resistance against shear failure. At a continuous support, the total shear resistance is multiplied with a factor depending on M_1 (larger moment at the end of the shear span) and M_2 (smaller moment), with M_1 and M_2 as absolute values.

$$\alpha = \sqrt{\frac{M_1 + M_2}{M_1}} \tag{11}$$

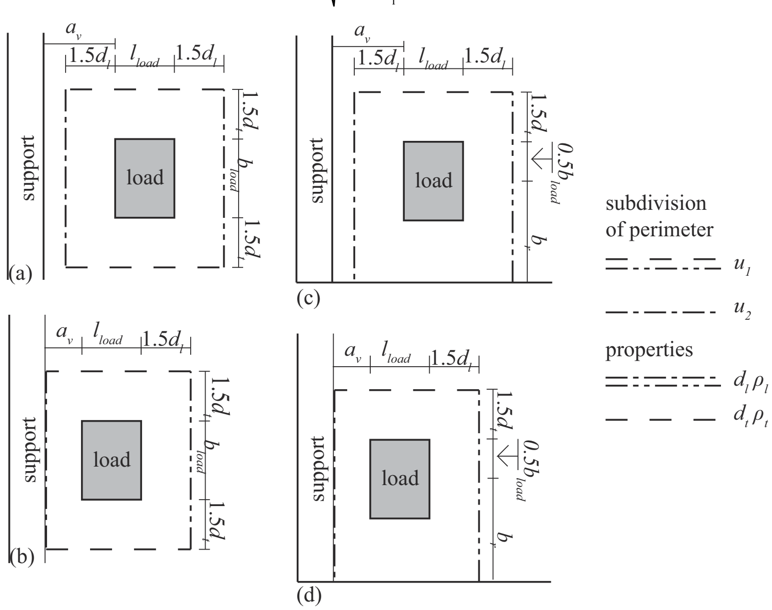

Figure 2 — Subdivision of perimeter and slab properties to be used for parts of the perimeter: for a load in the middle of the width of the slab (a) for $2d_l > a_v > 1.5d_l$, (b) for $a_v < 1.5d_l$ and for a load close to the edge of the slab (c) for $2d_l > a_v > 1.5d_l$, (d) for $a_v < 1.5d_l$, based on (Regan 1982).

RESEARCH CARRIED OUT AT DELFT UNIVERSITY OF TECHNOLOGY

Test setup

A top view of the test setup with a slab is presented in **Figure 3**. The line supports (sup 1 and sup 2 in **Figure 3**) are composed of a steel beam (HEM 300) of 300mm (11.81 in.) wide, a layer of plywood and a layer of felt of 100mm (3.94 in.) wide. Experiments are carried out close to the simple support (sup 1 in **Figure 3**) and close to the continuous support (sup 2 in **Figure 3**). The rotation at support 2 is restrained by vertical prestressing bars which are fixed to the strong floor of the laboratory. This restraint results in a moment over support 2 which hence behaves as a continuous support. The prestressing force is applied on the bars before the start of every test, offsetting the self weight of the slab. During the course of the experiment, some rotation could occur over support 2 due to the deformation of the felt and plywood and the elongation of the prestressing bars.

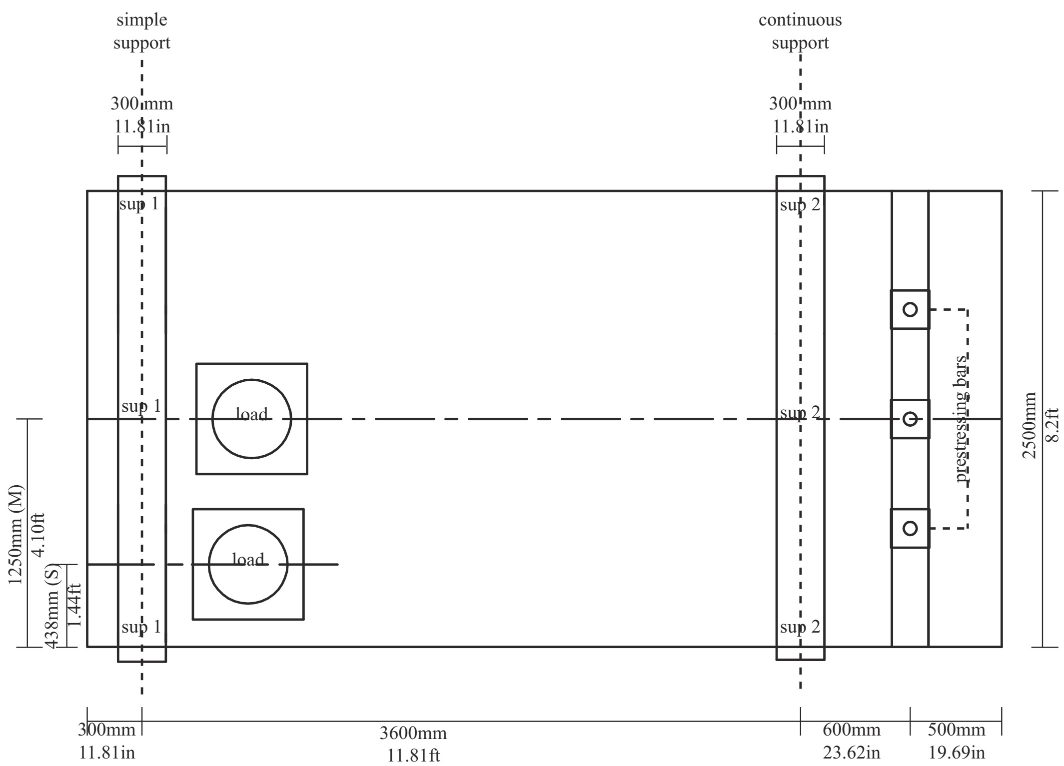

Figure 3 — Sketch of test setup, top view.

Specimens

The main properties of the eight slabs and twelve slab strips are given in **Table 2,** in which the following symbols are used:

f_c' the cube compressive strength of the concrete at the age of testing the slab,
f_{ct} the splitting tensile strength of the concrete at the age of testing the slab,
a the center-to-center distance between the load and the support,
M loading at the middle of the slab width,
S loading at the side at 438 mm (1.44 ft.) from the free edge, **Figure 3.**

All slabs and slab strips had a thickness of 300 mm (11.81 in) and an effective depth d of 265mm (10.43 in).

The numbering for the slabs starts with "S", while for the beams ("B") the numbering is subdivided according to the size: S, M, L, X. The slabs were either loaded at the middle of the slab width (position M) at the simple and continuous support, resulting in two tests per slab, or consecutively at the east and west side (position S) at the simple and continuous support, resulting in four tests per slab.

Ribbed reinforcing bars with a diameter of 10mm (0.4 in, cfr. #3 bars) (measured yield strength f_y = 635 MPa = 92.1 ksi and measured ultimate strength f_u = 710 MPa = 102 ksi) and 20mm (0.8in, cfr. #6 bars) (f_y = 601 MPa = 87.2 ksi and f_u = 647 MPa = 93.8 ksi) were used. The flexural reinforcement was designed to resist a moment caused by a load of 2 MN (450 kip, maximum capacity of the jack) at position M (**Figure 3)** along the width and 600 mm (23.62 in) along the span (a/d = 2.26). In practice, the amount of transverse flexural reinforcement for slabs is taken as 20% of the longitudinal flexural reinforcement. In the slabs discussed, 13.3% of the longitudinal flexural reinforcement was used in S1 and S2 and 25.9% in S3, S5, S6, S8, S9 and the slab strips. In S4 the amount of transverse flexural reinforcement was only doubled as compared to S1 and S2 in the vicinity of the supports. **Figure 4** shows elevation, cross-section and detailing of the reinforcement in S1 and S2.

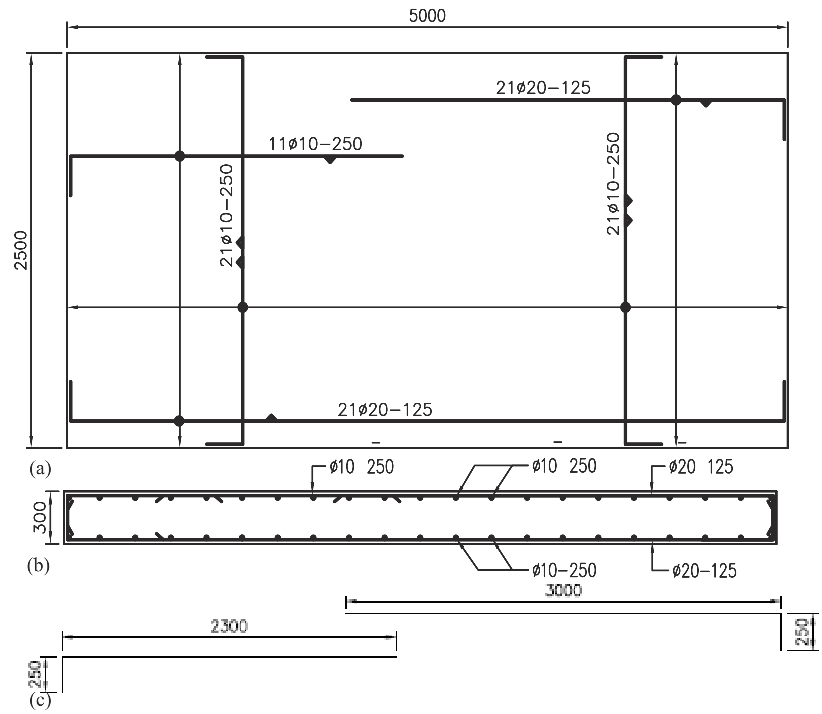

Figure 4 — Reinforcement layout of slabs: (a) plan view of S1 and S2, (b) section of S1 and S2, (c) Detail of top reinforcement. All slabs had similar longitudinal reinforcement. Units: mm (1 mm = 0.04 in.)

Two types of concrete have been used: normal strength concrete with a target cylinder strength $f_{c,cyl}$ of 43MPa (6.2 ksi) for slabs S1 – S6 and high strength concrete with a target strength $f_{c,cyl}$ of 73MPa (10.6 ksi) for slabs S8, S9 and the slab strips. Glacial river aggregate with a maximum aggregate size of 16 mm (0.63 in.) was used.

Two center-to-center distances between load and support (600mm = 23.6in, a/d = 2.26 for S1 – S4, S8 – S9, BS1 – BX1, BS3 – BX3 and 400mm = 15.8in, a/d = 1.51 for S5, S6, S9, BS2 – BX2) were used to study the influence of direct load transfer to the support, which becomes significant for $a/d \leq 2.5$ (where a is the center-to-center distance between the load and the support) and less (Kani 1964).

The 200 mm × 200 mm (7.9 in. × 7.9 in.) load is a 1:2 scale representation of the 400 mm × 400 mm (15.8 in × 15.8 in) axle load used in load model 2 of EN1991-2:2002.

Table 2 — Properties of slabs S1 – S6, S8 – S9 and slab strips BS1 – BX3.

Slab nr.	b m	f_c' (MPa)	f_{ct} (MPa)	ρ_l (%)	ρ_t (%)	a/d	M/S	$b_{load} \times l_{load}$ (mm × mm)	cast date (dd-mm-yy)	test date (dd-mm-yy)
S1	2.5	35.8	3.1	0.996	0.132	2.26	M	200 × 200	08-10-09	05-11-09
S2	2.5	34.5	2.9	0.996	0.132	2.26	M	300 × 300	08-10-09	03-12-09
S3	2.5	51.6	4.1	0.996	0.258	2.26	M	300 × 300	20-11-09	22-01-10
S4	2.5	51.7	4.2	0.996	0.182	2.26	S	300 × 300	20-11-09	04-02-10
S5	2.5	48.2	3.8	0.996	0.258	1.51	M	300 × 300	02-02-10	05-03-10
S6	2.5	50.6	3.9	0.996	0.258	1.51	S	300 × 300	02-02-10	15-03-10
S8	2.5	77.0	6.0	0.996	0.258	2.26	M	300 × 300	23-02-10	12-04-10
S9	2.5	81.7	5.8	0.996	0.258	1.51	M	200 × 200	10-03-10	26-05-10
BS1	0.5	81.5	6.1	0.996	0.258	2.26	M	300 × 300	23-02-10	19-04-10
BM1	1.0	81.5	6.1	0.996	0.258	2.26	M	300 × 300	23-02-10	26-04-10
BL1	1.5	81.5	6.1	0.996	0.258	2.26	M	300 × 300	23-02-10	31-08-10
BS2	0.5	88.6	5.9	0.996	0.258	1.51	M	200 × 200	10-03-10	14-09-10
BM2	1.0	88.6	5.9	0.996	0.258	1.51	M	200 × 200	10-03-10	14-09-10
BL2	1.5	94.8	5.9	0.996	0.258	1.51	M	200 × 200	10-03-10	06-09-10
BS3	0.5	91.0	6.2	0.996	0.258	2.26	M	200 × 200	22-03-10	20-09-10