POSTER SESSION 6

Mixed Structural Systems

Systèmes de structures mixtes

Gemischte Konstruktionen

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A New Building System – Spiked Steel Sheet – Composite Slab

Un nouveau système de construction: dalle mixte avec une tôle et des ancrages particuliers

Ein neues Bausystem: Verbunddecke mit Stahlblech mit Verbundhaken

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Background

In many countries fluted steel sheets are used also as form and reinforcement of slabs in the concrete construction. In these solution the bond between concrete and thin sheet is achieved by shaping sheet and/or by making knobs, grooves or folds on the surface of the sheet. The drawback in these solutions is poor bond between concrete and fluted sheet. That is why the ends of the fluted sheet have to be welded to the structure below, which on its half limits the use of the structure especially in steel frame buildings.

The aim of this development work has been to find such bonding projections which would turn thin sheet into a usable reinforcement. As a result of the work is the spiked steel sheet. The developed "nailty" ensures the thin sheet and concrete working together so that the area of steel cross-section can be taken as effective reinforcement.

The fluted and spiked steel sheets are usable with steel structures, ordinary reinforced concrete structures, reinforced concrete and prestressed element structures and wooden structures in almost all house construction.

Simplifying of reinforcement in slab constructions is possible because the thin sheet reinforcement is so effective that ordinary slab can be designed as one-way reinforced slab.

Compared with traditional concrete structures, the spiked steel sheets are lightweight building elements with which the construction work can also be carried out succesfully at sites where hoisting equipment cannot be used or is not available.

Technical features

- surface anchorage by means of the "nails"
- no additional end anchorage required at support
- application: all floor constructions in multistore buildings
- continuous manufacture of the anchorage "nails"
- use of dimensioning methods for reinforced concrete structures

Patents

Patent granded in Finland, Sweden, Norway, BRD, Switzerland, USA and Canada. Registered designs granded in Finland, England, France and Benelux.

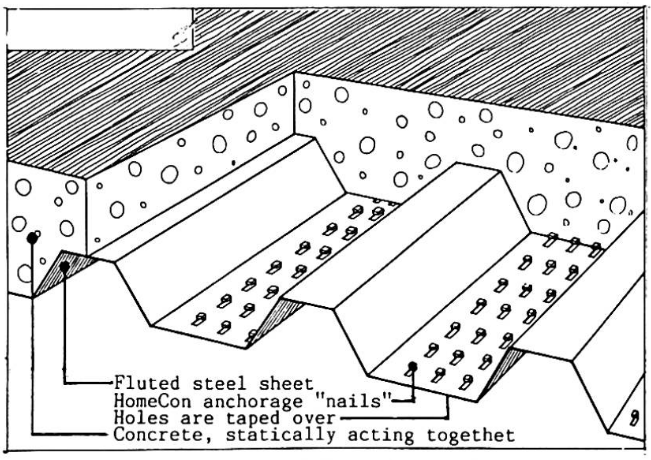

Fig. 1 Principle of HomeCon-fluted and spiked steel sheet.

Ultimate shear between spiked steel sheet and concrete

Shear capasity of "nailty" R_{ud} in the composite slab structure is obtained from the formula:

$$R_{ud} = n\ r_{ud}\ b\ a$$

where

- n is amount of "nails" (pcs/m^2)
- r_{ud} is shear capasity of one "nail" ($r_{ud} = b_n\ t_{cal}\ f_{yd}$)
- b is width of slab under consideration
- a is calculated value of bond length
- b_n is width of one "nail" (b_n = 4 mm)
- t_{cal} is calculated thickness of steel cross-section of thin sheet
- f_{yd} is design strength of steel sheet

Tensile force Z in thin sheet reinforcement is received from the formula:

$$Z = A_s\ \varepsilon_s\ E_s \leqslant A_s\ f_{yd}$$

where

- A_s is area of thin sheet reinforcement
- ε_s is strain of steel (^{o}/oo)
- E_s is modulus of elasticity of steel (E_s = 210 kN/mm^2)

When Z = R_{ud}, it is possible to check the value of bond length a. When all capasity of steel sheet are used the folloving formula is received:

$$a = \frac{A_s\ f_{yd}}{n\ b\ r_{ud}}$$

Table 1 The value of bond length a and shear capasity R_{ud}

Fluted steel sheet	t_{cal} mm	n pcs/m^2	b m	r_{ud} kN	A_s mm^2/m	f_{yd} N/mm^2	a mm	R_{ud} *) kN/m^2
HomeCon 45	0.63	588	1	0.73	820	291	556	429.2
	0.82	588	1	0.95	1070	291	556	558.6

*) Holorib R_{ud} = 50 kN/m^2

The HomeCon building system

- simplifies construction work
- reduces construction time
- enables widely the use of unskilled labour
- eliminates the need and transport of formwork materials
- improves order at the building site
- reduces the need for storage space because the sheets can be piled up in a small area without damage to the "nails"
- facilidates do-it-yourself construction
- is the important invention for on-site construction

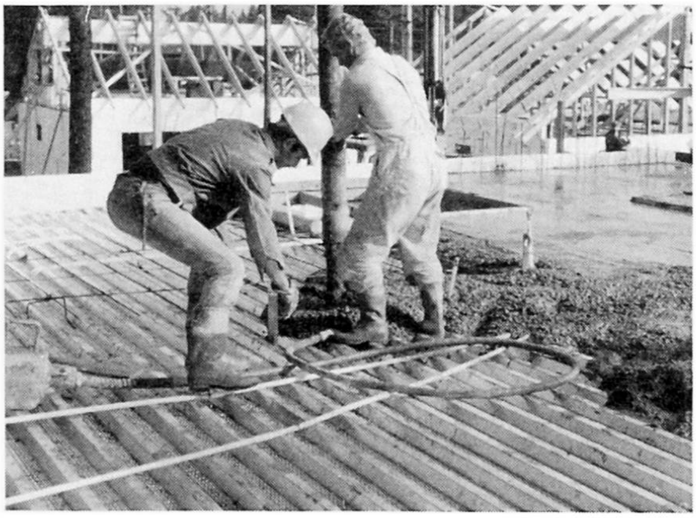

Fig.2 The fluted and spiked steel sheet is lightweight building element

Stiffness and Strength of Composite Beams in Frames

Rigidité et résistance de poutres mixtes dans des cadres

Steifigkeit und Widerstand von Verbundträgern in Rahmen

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1. INTRODUCTION

A composite beam in a frame under loadings which occur during an earthquake has a positive and a negative moment regions simultaneously. Hence, the composite beam becomes a sort of a beam with changes in cross section and presents complex dynamic behavior. This poster describes the evaluation of stiffness of composite beams based on frame experiments and a frame analysis using the accurately calculated stiffness of the composite beams. The stiffness evaluation is made by replacing two kinds of moment of inertia of a composite beam under positive bending and negative bending with a single equivalent moment of inertia. Also, regarding the strength, story shear force capacity of the frames and positive bending moment capacity of the composite beams upon crushing of concrete slabs are discussed in the poster.

2. EQUIVALENT MOMENT OF INERTIA OF COMPOSITE BEAMS

The test was performed in six steel frames each with a fully composite beam and one frame with a steel beam (see Table 1). Specimens were designed so that a composite beam may fail prior to steel columns. Figs. 1 and 2 illustrate the details of a specimen and the loading apparatus, respectively. Circles in Fig. 4 represent the experimental elastic story stiffness ($P/{}_sP_p/u/{}_su_p$) of the frames (see Fig. 3). In the figure, t_c means the thickness of concrete slabs. On the other hand, the flexural stiffness matrix of a composite beam of Fig. 5 is given as Eq.(1) from the accurate analysis.

$$\begin{Bmatrix} M_{AB}/{}_sM_p \\ M_{BA}/{}_sM_p \end{Bmatrix} = \begin{bmatrix} 0.57\beta + 0.10 & 0.16\beta + 0.17 \\ 0.16\beta + 0.17 & 0.14\beta + 0.52 \end{bmatrix} \begin{Bmatrix} \theta_A/{}_s\theta_p \\ \theta_B/{}_s\theta_p \end{Bmatrix} \quad \cdots\cdots\cdots\cdots\cdots \quad (1)$$

where ${}_sM_p$: full plastic moment of steel beam, ${}_s\theta_p={}_sM_pL/6E_sI$, E: Young's Modulus, ${}_sI$: moment of inertia of steel beam, β: ratio of moment of inertia of composite beam under positive bending moment to moment of inertia of steel beam. Story stiffness of the frames shown in Fig. 4 as a solid line are calculated by employing Eq.(1). In this calculation, an effective width of concrete slabs of the test frames was determined as follows: The longitudinal stress distribution in width direction within the concrete slab was assumed to be a trapezoid distribution $\triangle$ ABDC in Fig. 6. To obtain the effective width, a rectangular distribution $\square$ EFHG in the same figure was considered. The effective width $\overline{EF}$ was determined by equalizing two areas of $\triangle$ ABDC and $\square$ EFHG. As a good agreement between the story stiffnesses from the experiments and the analysis was shown, an attempt was made to substitute a single equivalent moment of inertia, ϕ_sI, for the moment of inertia of the composite beam consisting of two kinds of moment of inertia. The equivalent coefficient, ϕ, was determined on the condition that a story stiffness of a frame with a composite beam, whose flexural stiffness is predicted by Eq.(1), is equal to that of a frame with an equivalent beam.

3. STORY SHEAR FORCE CAPACITIES AND POSITIVE BENDING MOMENT CAPACITIES

Story shear force capacity of the frames (see Fig. 3) and positive bending moment capacity of the composite beams (see Fig. 7) upon crushing of concrete slabs in the tests are indicated with circles in Figs. 8 and 9, respectively. In the same figures, a solid line represents calculative values. The story shear forces were obtained by assuming one end of the composite beam in the frame as the full plastic moment under positive bending and the other as the full plastic moment under negative bending. On the other hand, bending moment capacities in the calculation mean the full plastic moments themselves under positive bending. The width of the concrete slabs used in the calculation of the full plastic moments is the same as the afore-mentioned effective width of the concrete slabs.

4. CONCLUSIONS

The equivalent coefficients, ϕ, which represent the equivalent moment of inertia, $\phi_s I$, of the composite beam range from 1.53 to 1.78 for one-bay one-story frames tested. The equivalent moment of inertia is also found to be about 70 % of the moment of inertia of the composite beams under positive bending regardless of the thickness of the concrete slabs. The story shear force capacities upon the crushing of concrete slabs nearly coincide with the story shear forces calculated by the method mentioned under Chapter 3. The experimental positive bending moment capacity of the composite beams upon the concrete slab crushing are nearly as large as the full plastic moments under positive bending.

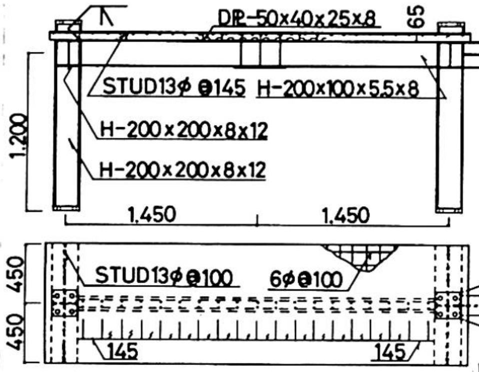

Fig. 1 Test Specimen

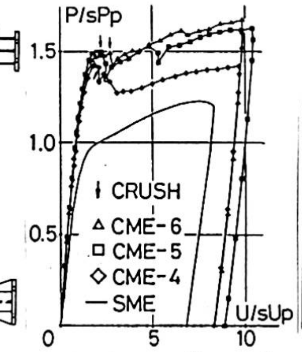

Fig. 3 Story Shear Force - Displacement

Table 1 Moment of Inertia and Full Plastic Moment of Beams

Specimen	sI	cIn	cIṅ	sMp	cMp	cMṗ
S M E	1792	—	—	795	—	—
CME-4	1788	3690	2086	741	1140	802
CCE-4	1809	3703	2106	748	1146	809
CME-5	1802	4252	2159	730	1181	797
CCE-5	1781	4147	2124	721	1195	787
CME-6	1802	4670	2193	772	1348	842
CCE-6	1797	4679	2189	770	1349	840

sI,cIn,cIṅ : [cm⁴] sMp,cMp,cMṗ : [tcm]

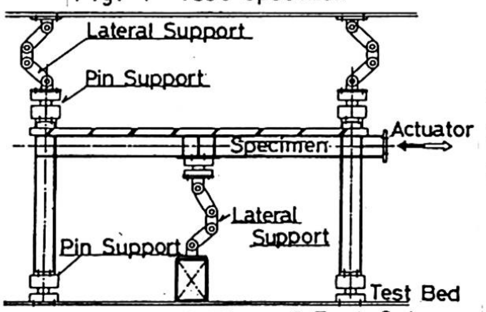

Fig. 2 General View of Test Setup

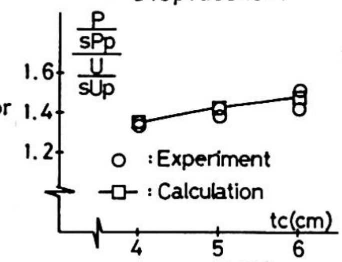

Fig. 4 Story Stiffness of Frames

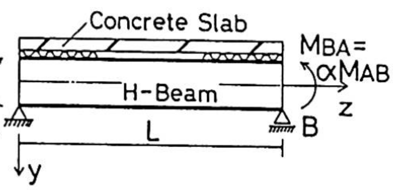

Fig. 5 Composite Beam for Analysis

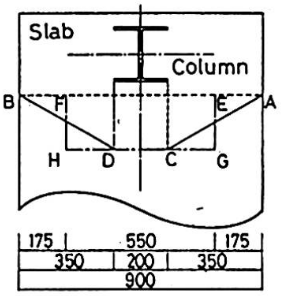

Fig. 6 Effective Width of Concrete Slab

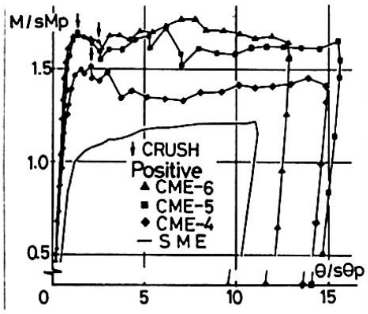

Fig. 7 Moment - Rotation of Beams

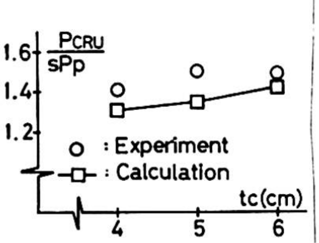

Fig. 8 Story Shear Force upon Concrete Crushing

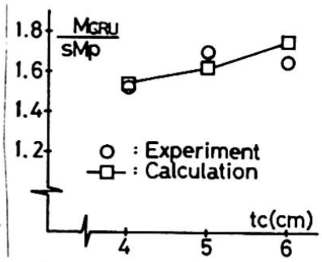

Fig. 9 Moment upon Concrete Crushing

Optimal Design and Erection Problems of a Slender Box Girder Bridge

Projet et montage d'un pont mixte en caisson élancé

Optimale Konstruktion und Montagefragen einer schlanken Kastenträgerbrücke

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1. INTRODUCTION

The construction of a slender three-span bridge across lake Foxen in the province of Värmland, Sweden, is under way. The span lengths are 25, 125 and 25 m and are closely adapted to the natural conditions, see Fig. 1. Normally, a main span of 125 m is not regarded as very long, but it is unusually long for a composite steel-concrete bridge. Further, in view of the slenderness of the main span, expressed as span to height = 125/3.5 m, the bridge is remarkable.

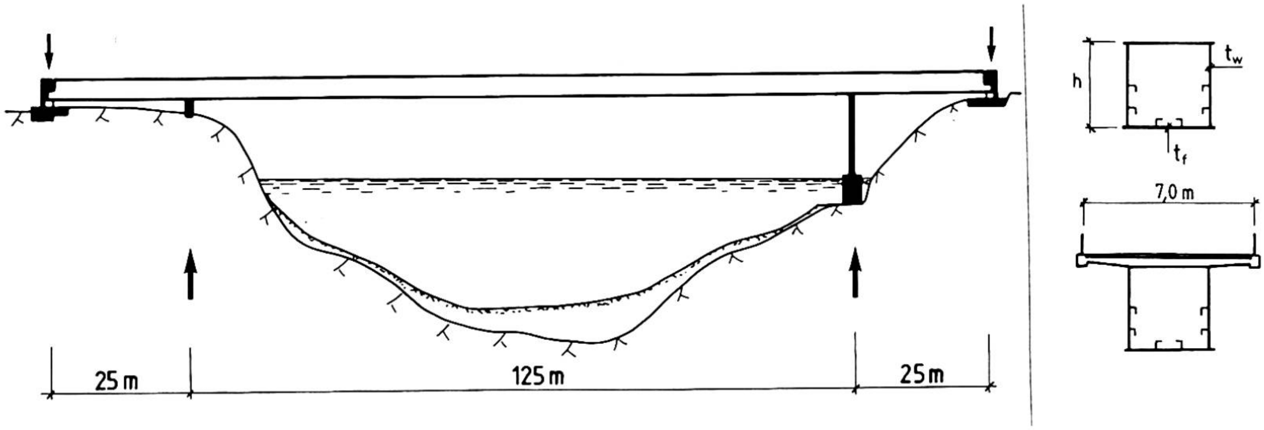

Fig. 1 Bridge across lake Foxen

The extreme relation between the span lengths has led to special considerations for the erection procedure (launching), which in its turn had to be considered in the design stage. In all erection stages and in the permanent serviceability state the outer support reactions are directed downwards due to the short end spans. The steel girder is made as a closed box-section in the support regions but in the central region of the main span it is made as an open U-section. After incremental launching from both ends and when the two parts had been welded together, the end supports were pressed down in order to introduce negative bending moments. That is the situation in November 1986, when this paper is written.

In the next stage the concrete slab that constitutes the roadway is going to be cast. When the concrete slab has hardened, the outer supports are once again manipulated and are this time lifted to impose positive bending moments. By a careful determination of the outer support movements it is thus possible to achieve an optimal composite action of the structure for all loading conditions including the greater part of the dead load.

A study of the crack formation in the slab in the support regions during construction as well as during the permanent state is also planned.

2. DEFLECTION AND STRAIN MEASUREMENTS

The steel structure had an extremely long free cantilever when launched (62.5 m) and furthermore it has very slender webs (175 < h/t < 290) and comparatively small thickness ratios between flanges and webs ($1.58 < t_f/t_w < 1.78$). Thus it was an especially interesting object for a full-scale field test to compare with design methods with respect to the stability of slender webs under patch loading. During launching the reactions from the supports were taken by temporary bogies with two or four rollers respectively. Measurements were made at seven cross sections in connection with their passage over the launching rollers. At each occasion readings were made at every 100 or 150 mm. Typical results for the case of four rollers are shown in Figs. 2 and 3.

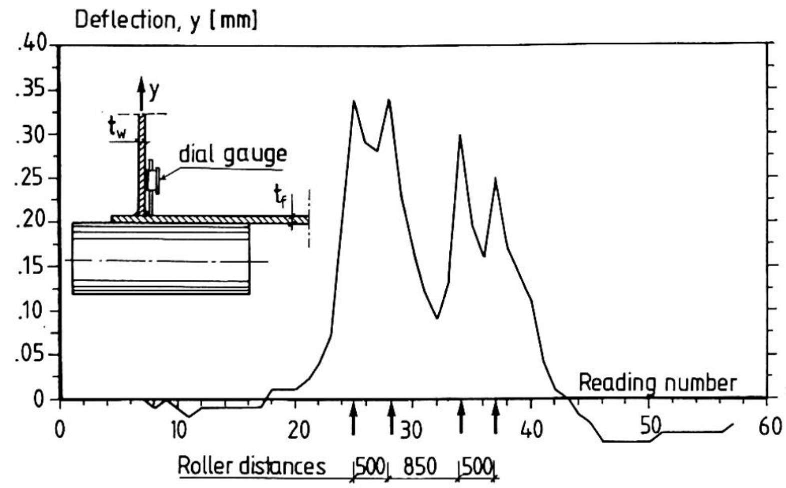

Fig. 2 Vertical flange deflection

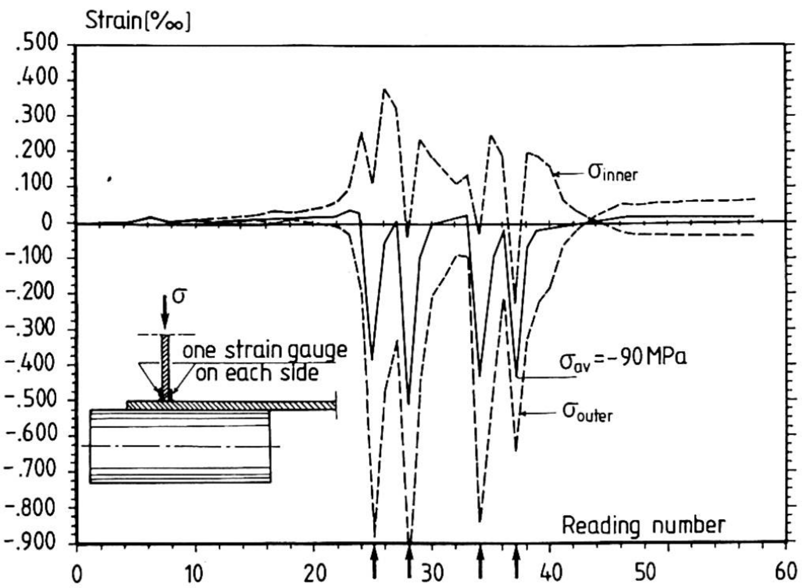

Fig. 3 Web strains close to the flange

Some conclusions based on the measurements:

1. High local membrane stresses in the web over the rollers increase the risk for local buckling, because a yield hinge may form in the web close to the flange and this effect was facilitated due to moving load. At small ratios of t_f/t_w yield hinges may also appear in the flange.

2. The load distribution on two or more rollers reduces the normal stresses in the web close to the flange in a linear manner, but the reduction of vertical flange deflection (and therefore also of the ultimate patch load) is considerably smaller.

Effect of Location of Shear Connectors on Behavior of Composite Beam

Influence de l'emplacement des joints sur le comportement de poutres mixtes

Einfluss der Schubdübelanordnung auf das Verhalten von Verbundträgern

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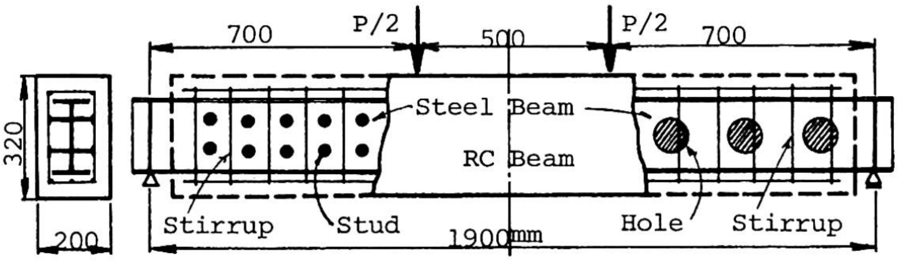
The composite beams, where steel beams are embedded in reinforced concrete, are usually applied in practice to the case where the depth of a concrete bridge is severely restricted. A sufficient number of shear connectors are necessary on the embedded steel beams, so that they may act monolithically together with the reinforced concrete. It is, however,undesirable from a view-point of the restriction of the depth of a bridge to plant shear connectors on the flanges of the steel beams.

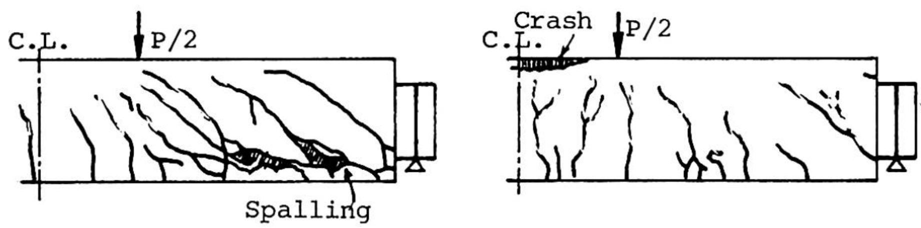
The authors conducted a series of experiment on about twenty specimens, in which mainly the number of shear connectors, their location and the amount of stirrup were varied. The outline of two specimens is illustrated as examples in Fig.1. As a result, it was revealed that the location of shear connectors, i.e., the upper flange, the web or the lower flange had a minor influence on the behavior of the composite beam, if they were provided in a sufficient number. Thus, it was confirmed that a beam where studs were planted on the web of a steel beam embedded in reinforced concrete could be used as a perfectly composite beam. It was also found that if there was no shear connector on the steel beam, the concrete may fracture by the shearing force as shown in Fig.2 (a) and, consequently, a considerable amount of stirrup was necessary in the concrete to prevent the shearing fracture. On the contrary, a composite beam with studs on a steel beam or with sufficient stirrups failed in such a manner as shown in Fig. 1(b), and if shear connectors were planted on a steel beam, the stirrups in the concrete could be reduced as much.

The influence of number and location of the shear connectors on the behavior of compo-



(a) Spec. with Studs (b) Spec. with Holes

Fig.1 Examples of Specimens Used for Experiment



(a) Shearing Fracture (b) Bending Fracture

Fig. 2 Examples of Fracture Mode

site beams was also analytically investigated. There both the reinforced concrete beam and the steel beam in composition were devided into many bar-elements and they were connected with each other by elasto-plastic springs both in the axial and transverse directions, as shown in Fig.3. The shear connectors between the reinforced concrete beam and the steel beam were also represented by elast-plastic springs. The analytical results concerning stiffness, stresses and strength for each type of the composiste beams presented a considerably good agreement with those experimental results.

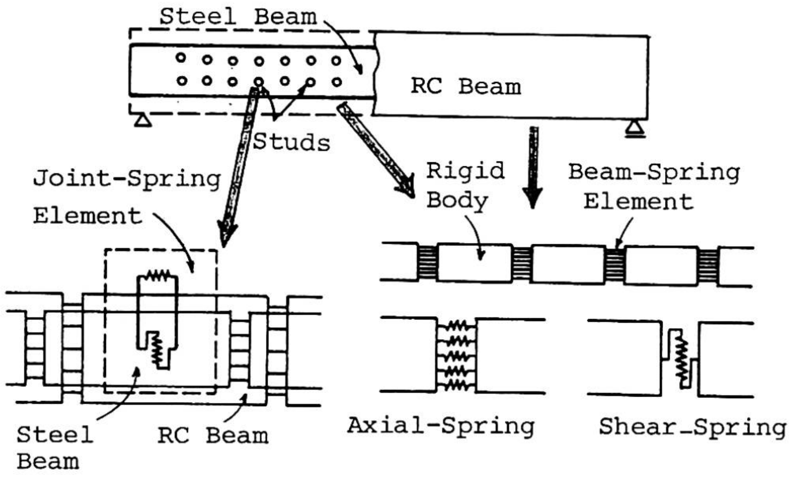


Fig.3 Rigid Body-Spring Element Model

The experiment showed also that in some cases only the perforation in the web of the steel beam made as shown in Fig.1 was as effective to accomplish the composite action. The holes in the steel beams are also useful to facilitate the transverse reinforcing bars through them in order to connect the steel beams arranged in parallel in concrete. It should, however, be noted that the resistance of the steel beam to shearing force may be reduced due to the perforation of a large diameter.

According to the results of the present experiment, even the specimen with four studs developed almost the same ultimate bending strength as the one with 56 studs, that was greater than the value obtained from the super-imposed strength method, which has been currently used in Japan. According to the analytical calculation, the specimens with more than 16 studs have the full ultimate bending strength(see Fig. 4). But the end slippage between the upper flange of the steel beam and the concrete is far larger in the former specimen than in the latter one as shown in Fig. 5. Fig. 4 also indicates that the studs of the specimen with four studs reach the yielding stress and about ten studs will be required under a repeated loading, assuming the two million cycle fatigue allowable strength as about 80 N/mm^2 (820 kgf/cm^2).

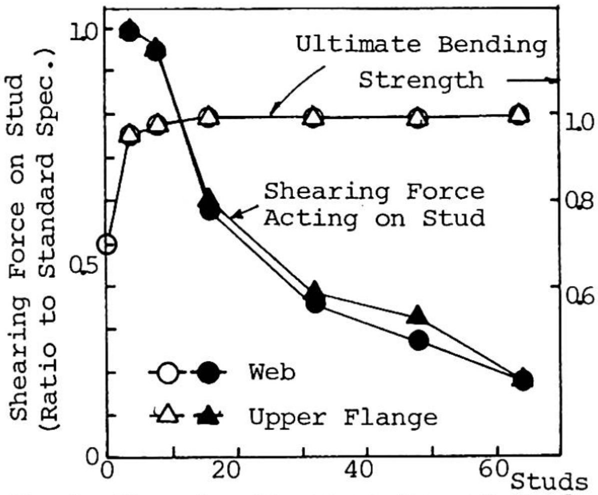


Fig.4 Shearing Force vs. No. of Stud (By Calculation)

Based on the investigaton presented here, it can be concluded that the studs on the web of the steel beam encased in concrete are as effective as those on the upper flange and the number of studs may be determined according to the calculation method the authors have developed, which will be more simplified in the future. Thus, composite bridges which are structurally more efficient and easier in construction than the conventional ones can be expected to be developed in practice.

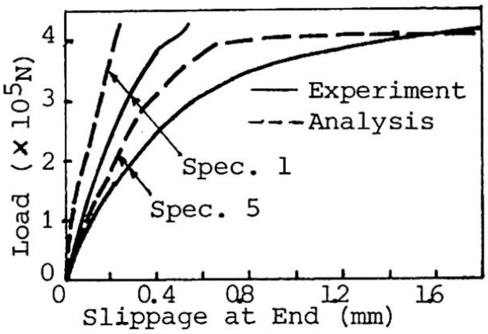


Fig. 5 Load vs. Slippage at End

Timber-Concrete Composite Structures

Structures mixtes bois et béton

Holz-Beton Verbundkonstruktionen

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SUMMARY

Two timber concrete composite structures, TP-slab and BP-slab, developed by the author and manufactured in Finland since 1979 and 1985 will be presented. Both structures are concrete slabs with timber joists connected to concrete with nail plates. TP-slab is used mainly as a floor and PB-slab as a wall structure. The composite behavior creates a considerable increase of strength and stiffness. Also other structural benefits are gained e.g. deformation stability, dynamics, acoustics, fire and moisture resistance.

1. INTRODUCTION

Two slabs which consist of the timber joists and the concrete slab connected with nail plates is presented. The special thing lies in the nail plate connector which brings some advantages e.g. allows a gap between the joist and the concrete. Further, the nail plate has turned out to be a reliable connector with a low ratio of prize/stiffness and prize/strength.

Many other similar slabs have been developed with different connection means e.g. nail, stable, bolt, steel rod, friction (caused by grooving the joist edge). These timber concrete composite slabs have been studied in Lund, Sweden; Luleå, Sweden; Lodz, Poland; Krakow, Poland; Hildesheim, Germany; Lausanne, Switzerland; Florence, Italy.

2. PB-SLAB

PB-slab is manufactured in a prefabricated element plant on a horizontal mold, fig. 1. The joists will be located 600...1200 mm apart. PB-slab is mainly used as a wall element of 1 or 2 store dwelling or hall type buildings. Figure 5 shows a typical wall cross section. The building has usually a trussed rafter roof. The trusses are supported directly on the concrete slab and the forces are transported to foundation in the concrete. The joists make the buckling support and resist the moment due to wind load. The joist are cut ≈30 mm above the ground floor level, so the contact between timber-concrete and also timber-humid air is eliminated. According to analysis and test a wall like this carries a very high compression force.

3. TP-SLAB

TP-slab is a floor structure, fig. 2, 3. Concrete is on the upper side and it will be cast on site. Concrete casting is made on 0.4 mm noncorrugated, membrane behaving steel sheet which also works as a reinforcement, no other reinforcement is usually used. The concrete can be liquid, selflevelling type and as thin as 25 mm. However, 50 mm standard concrete is mainly used and recommended due to dynamics and acoustics. The steel sheet is punched 12 mm into the nail plates. This is made with a lath and a mallet or with an especially developed machine. The nail plates connect the joists to the concrete and also the steel sheet to the concrete. Before the concrete will be cast the steel sheet can be used as a working platform. While the concrete is cast a temporary support is used at the middle of the span. This support is utilized to get a camber of appr. span/500...1000.

4. NAIL PLATE

The nail plate has especially been developed for the purpose, fig. 4. To increase the strength of the plate spikes are not punched on the gap area, further this part of the plate is slightly corrugated. The spikes which are cast into concrete have anchoring barbs to ensure the connection. The nail plates are usually pressed only on the other side of the joist, in cases where high strength or stiffness is needed double plates are used. The nail plate is strong enough to allow a gap of ...50 mm between the joist and the concrete.

5. DESIGN METHOD

At present the design is based on a finite element vierendel model with a semirigid connection between the nail plate and the joist. The constraint forces due to concrete shrinkage, temperature and moisture deformation are either neglected (assumed to be included safety factor because these forces are small in controlled climate conditions) or assumed to be ±0.02% which is added to all other loads.

6. EXPERIENCE

The main idea in developing the PB-slab and the TP-slab was to utilize the composite behavior of the joist and the concrete. The benefit due to composite action is significant because the I-cross-section of the joist is changed to T-cross-section and in a typical case strength is increased 2...3-times and stiffness 5...10-times compared to noncomposite behavior. However, these slabs have turned out to have many other properties which in many cases are more important. E.g. concrete is fire resistant, water tight, deformation stabile, acoustically and dynamically well behaving, the thermal insulation is got easily by having the insulation between the joists, the piping and the wiring may be set between the joists, the sheeting may be nailed or screwed directly to joists. The structure is light enough to have similar structural details as other light weight structures but it still has most of the benefits of heavy structures e.t.c.

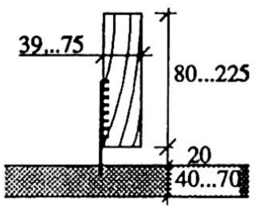

Fig 1, PB-slab

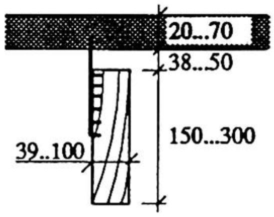

Fig 2, TP-slab

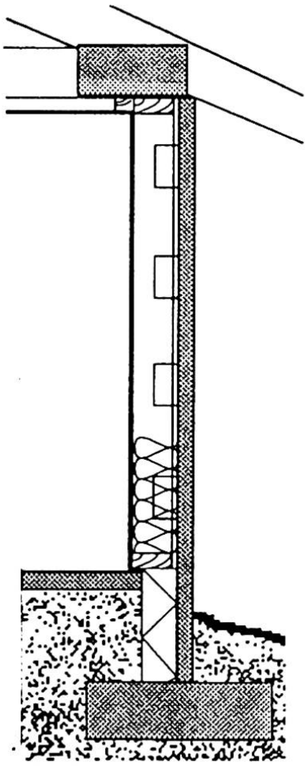
Fig 5, PB-slab-wall

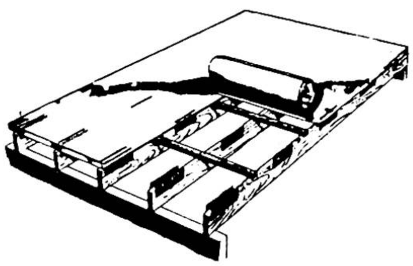
Fig 3, Principle of TP-slab

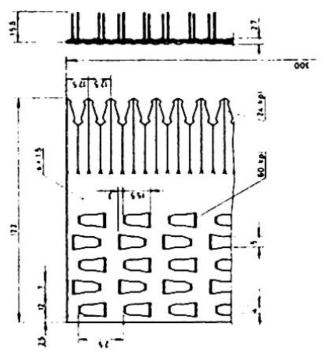
Fig 4, Nail plate

Prestressed Composite Bridges in Belgium

Ponts mixtes précontraints en Belgique

Vorgespannte Verbundbrücken in Belgien

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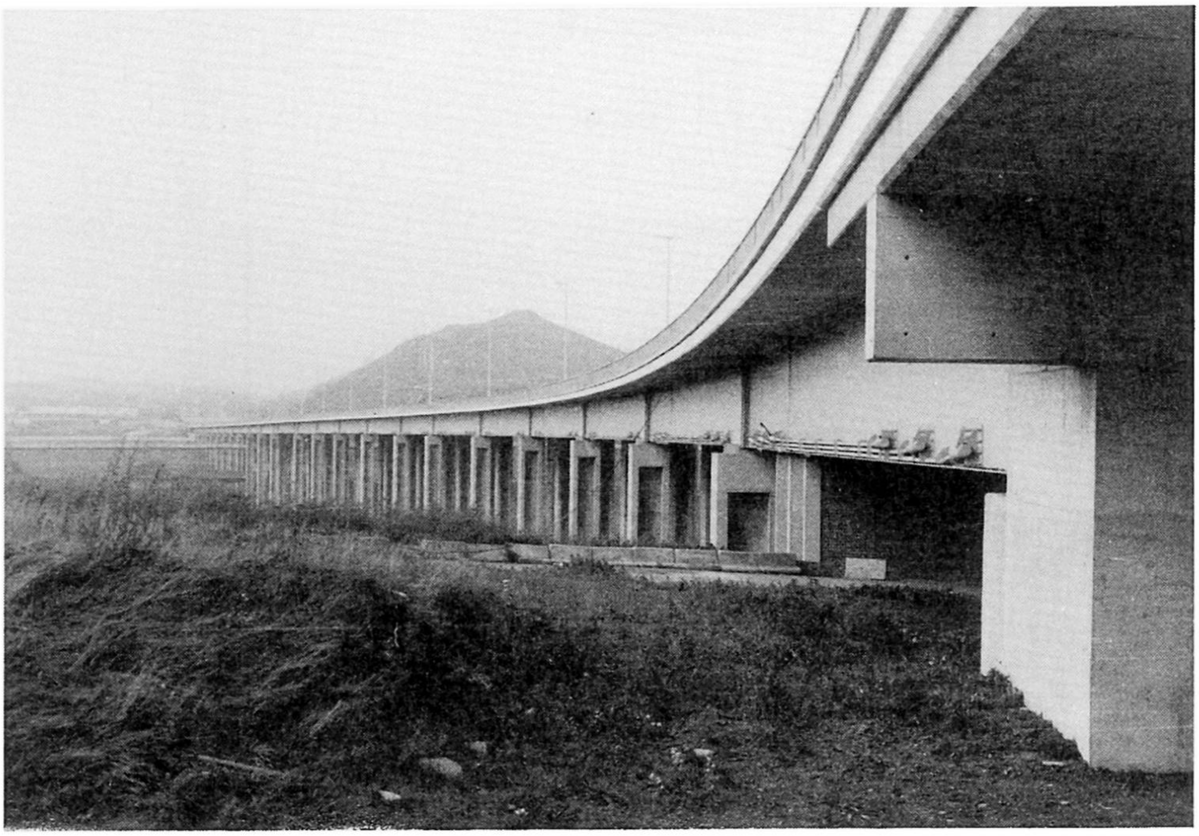
SUMMARY

An important developement in the field of bridge design seems to be a large combination of materials like concrete, light weight concrete, rolled steel, prestressing steel. This combination makes it possible to use, for each element of the structures, the adequate materials, working at the utmost of their possibilities : - concrete (compression strenght, stiffness) - light concrete (compression strenght, light weight) - rolled steel (tension strenght) - prestressing steel (high tension strenght). The purpose of the poster is to present two recent composite bridges, built in Belgium. The viaduct over the river Sambre at Châtelet (see fig. 1) is 1043 m long and has a prestressed composite, light concrete-steel structure. The new "Devallee" bridge over the river Scheldt at Tournai (see fig. 2) is a statically undetermined structure in prestressed concrete including prebended beams in the main span. The bridge has been designed for heavy convoys up to 360 t. The deck depth/main span ratio is 1/47.

Fig 1 VIADUCT OVER THE RIVER SAMBRE AT CHATELET



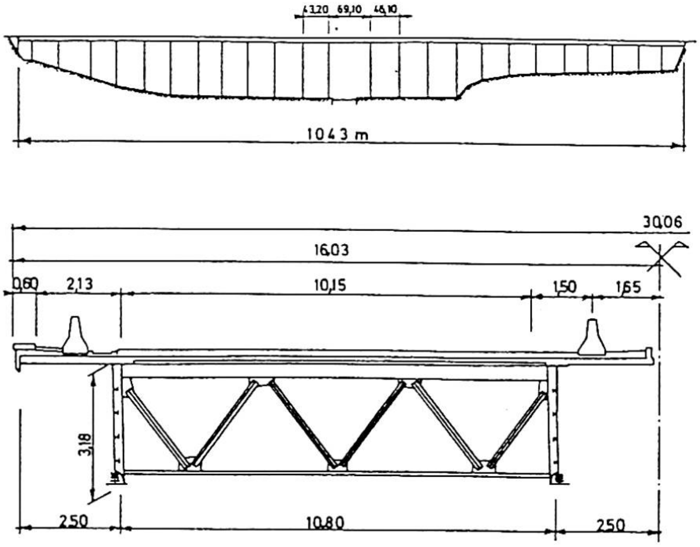
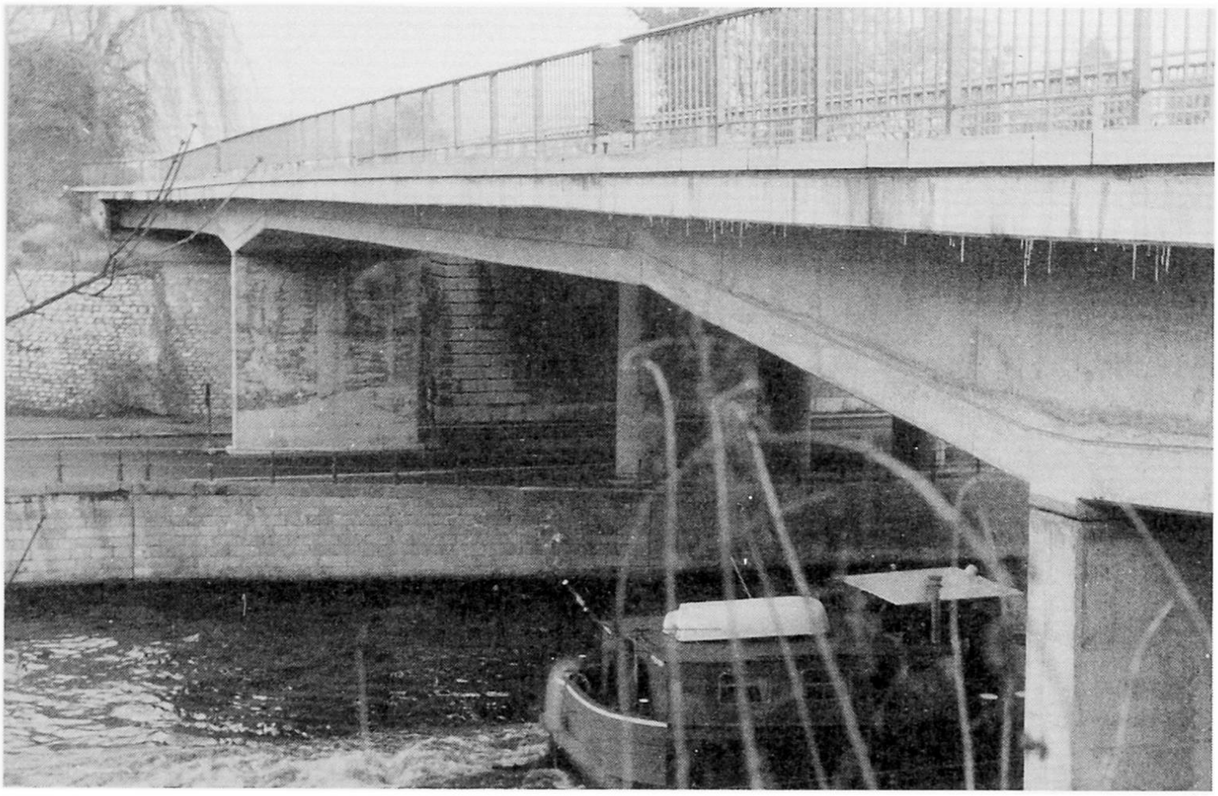
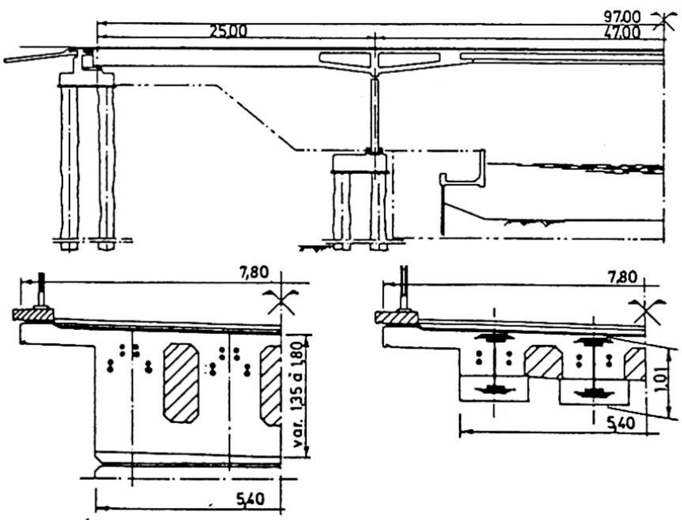


Fig 2 NEW "DEVALLEE" BRIDGE OVER THE RIVER SCHELDT AT TOURNAI





Seismic Load Resisting Building Framing Systems of Large Precast Elements

Comportement aux séismes de structures préfabriquées en béton

Erdbebenbeanspruchte Stahlbetonrahmen in Grossfertigteilbauweise

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1. INTRODUCTION

New Zealand is located on the seismically active Pacific rim where the Indian and Pacific plates meet. Thus for multi-storey construction the loads from potential earthquake shaking are the dominant horizontal loads to be considered in design in all but the tallest buildings. Framing members for multi-storey buildings tend to be relatively large and reinforcing relatively congested in order to comply with New Zealand seismic design philosophy.

The most popular form of seismic load resisting systems in multi-storey buildings are reinforced concrete perimeter frames. There is relatively little structural steel used.

Recent multi-storey reinforced concrete buildings constructed in New Zealand have included substantial use of precast elements for major structural members.

2. BUILDING FRAMING SYSTEM

In this display the perimeter seismic load resisting frame of a 13 storey building in Wellington, New Zealand, is illustrated. The building is octagonal in plan with an area of approximately 860m^2 per floor. The major and minor axis of the octagon are 37.3m and 27.3m respectively. All facets of the octagon are nominally of equal length, with three columns located on each facet giving a total of 24 columns in the perimeter frame. The frame members were designed to the New Zealand Concrete Design Code (1) using capacity design for both beams and columns, the general principle being that the beams yield in flexure while shear in beams, and flexure and shear in columns, are maintained in the elastic range.

Figure 1 shows the partly erected perimeter frame. The precast units in the perimeter frame consist of a column member two storeys in height, with two levels of beam stubs cast as an integral part of the precast unit, see figure 2. This configuration gave the precast units an overall weight of approximately 11 tonnes.

The display shows details of the precast units, particularly the beam column joints, the cast-in-situ beam splices, and grouted column splices.

The insitu beam splices were detailed so that the reinforcing to be placed on site in the insitu section was minimal. The splice consisted of overlapping hooks on the longitudinal bars with stirrups that could be slid into position once the adjacent precast members were located.

The column splices were located at mid storey height to keep the splice region clear of the most highly stressed section of the columns. The reinforcing bars were joined using NMB splice tubes which are a steel sleeve filled with an epoxy grout. The same grout was used for the butt joint between column sections. The grouting operation for both the butt joint and the splice tubes was carried out concurrently once a full floor of precast units was erected and temporarily braced.

3. CONCLUSIONS

Precast reinforced concrete moment resisting frames are a viable alternative to more conventional cast-in-situ frames from the point of view of speed of erection and structural integrity.

4. ACKNOWLEDGEMENTS

The author wishes to thank the Aurora Group Limited for permission to present the details of the precast system illustrated in the display.

5. REFERENCES

NZS 3101 Part 1: 1982, "Code of Practice for the Design of Concrete Structures", Standards Association of New Zealand.

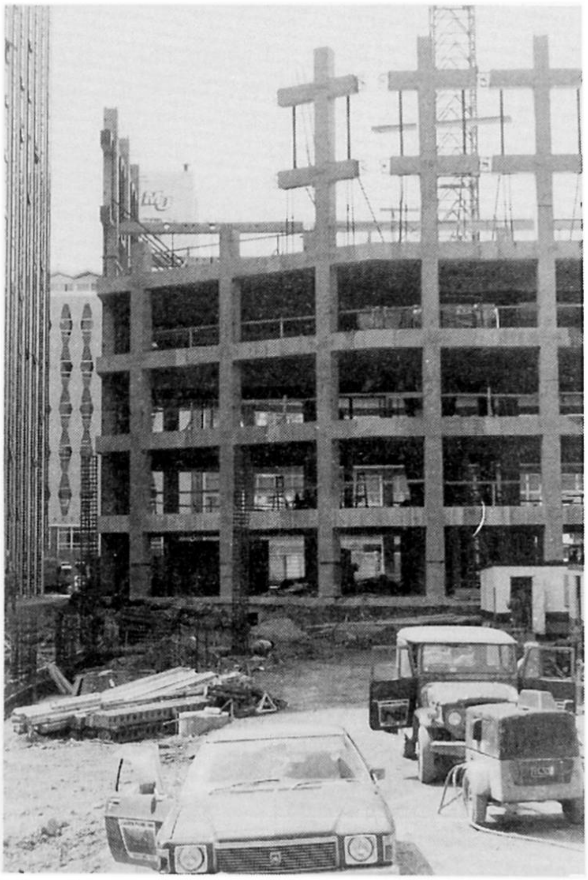

Fig 1. Partly Completed Perimeter Precast Frame

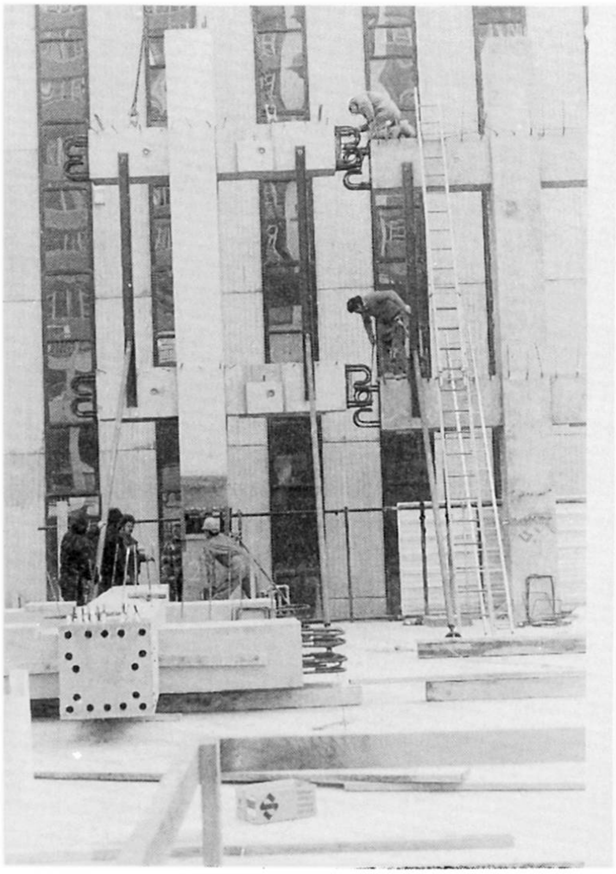

Fig 2. Precast Units During Erection

Strength of Embedded Type Corner Steel Column Base

Résistance d'un pied de poteau d'angle en acier encastré

Widerstand einbetonierter Eckstützen aus Stahl

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The mechanical properties of the corner steel column base embedded in concrete are not yet fully clarified. So, the experiments were conducted using models to clarify these mechanical properties.
Table 1 indicates the test program and Fig. 1 shows the outline of the test. Tie spacing was taken as the test variable.
Cold formed square steel tube was used for the columns to which alternating positive and negative lateral forces were repeatedly applied by hydraulic jack. Fig. 2 shows the load (Q) - deformation (δ) relations. Fig. 3 shows the envelope curves for these Q-δ relations. The test results are indicated in Table 3. In all of the specimens, the ultimate strength of the column bases was governed by the shear fracture of end concrete. The test results indicated that as the ties were spaced closer, the ultimate strength and ductility increased and the degree of decrease in maximum strength due to repeated loading became small. The initial rigidity, however, remained unchanged.
For calculation of the ultimate strength, the fracture planes of concrete due to punching shear were assumed for each of the positive and negative loadings as shown in Fig. 4. [1] It was assumed that the capacity of concrete against punching shear was to be obtained firstly by developing these areas on the end wall of foundation beam and then by multiplying such developed areas (Su and Sℓ) by $\sqrt{Fc}$ where Fc was concrete strength. (Rcu = Su$\sqrt{Fc}$ and Rcℓ = S$\ell\sqrt{Fc}$) However, since the ultimate strength increases as the tie spacing becomes shorter, the capacity formula should take into account the effects of reinforcement as well. For the purpose of this paper, the capacity in which the effect of reinforcement is considered is taken as the capacity (Rcu or Rcℓ) of concrete against punching shear fracture plus the capacity (Rhu or Rhℓ) of that part of reinforcement which is located above or below the neutral axis of the embedded portion of the steel column.
As for the load-resistance mechanism of the embedded portion of the column, three cases were assumed as indicated in Fig. 5. Then, the bearing stress resultants (Bu and Bℓ) caused in portions above and below the neutral axis by the assumed lateral force were calculated for the aforesaid three cases. For each case, the ultimate strength was calculated by equating it: (I) to the capacity of concrete against punching shear fracture (Bu = Rcu and Bℓ = Rcℓ); and (II) to the aforesaid capacity plus the capacity of the reinforcement (Bu = Rcu + Rhu and Bℓ = Rcℓ + Rhℓ). Since the tie bars did not yield in the present experiment, the actual stresses were used for calculation. The ratios of the values obtained by the experiments to the calculated values are indicated in Fig. 6. The experiment results clearly indicate that, where the effects of reinforcement are taken into account, the values calculated in the way as shown in Case 2 (see Fig. 5) approximately coincide with the experiment values, which means that the load causing the fracture of concrete covering outside the corner column base can be estimated from such calculated values.

Table 1 Test program

No.	Specimen Symbol	L (mm)	L/D	E (mm)	E/D	*1 (mm)	*2 H, F	*3 M, R
1	CH-RH$_N$ II'i 16	250	2.50	100	1.00	—	H	R
2	CH-RH$_{75}$ II'i 16	〃	〃	〃	〃	75	〃	〃
3	CH-RH$_{50}$ II'i 16	〃	〃	〃	〃	50	〃	〃
4	CH-RH$_{25}$ II'i 16	〃	〃	〃	〃	25	〃	〃

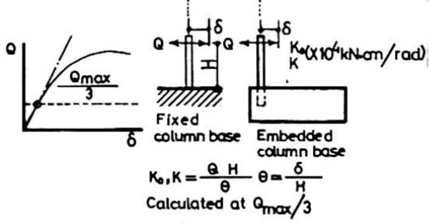
Table 3 Test results

No.	Specimen Symbol	Qy (kN)	Qp (kN)	Q_{max} (kN) (+)	Q_{max} (kN) (-)	Q_{max}/Qp (+)	Q_{max}/Qp (-)	Ko ($\times10^4$ kN-cm/rad)	K ($\times10^4$ kN-cm/rad) (+)	K (-)	K/Ko (+)	K/Ko (-)
1	CH-RH$_N$ II'i 16	53.0	63.9	37.6	25.6	0.59	0.40	33.6	17.9	17.1	0.532	0.508
2	CH-RH$_{75}$ II'i 16	〃	〃	61.5	42.0	0.96	0.66	〃	18.1	17.7	0.539	0.528
3	CH-RH$_{50}$ II'i 16	〃	〃	63.0	42.5	0.99	0.67	〃	17.4	16.7	0.517	0.499
4	CH-RH$_{25}$ II'i 16	〃	〃	74.0	61.2	1.16	0.96	〃	17.9	17.2	0.533	0.513

Table 2 Mechanical properties of steel column and concrete

		σy (N/mm²)	σ_{max} (N/mm²)	E ($\times10^5$ N/mm²)	El. (%)
Column	□-100×100×6.0	458	509	2.18	22
Tie	D6	479	522	1.92	21
Concrete		—	24.3	0.217	—

*1 Tie Spacing
*2 Hollow or Filled
*3 Monotonic or Repeated
No anchor bolt
No axial load



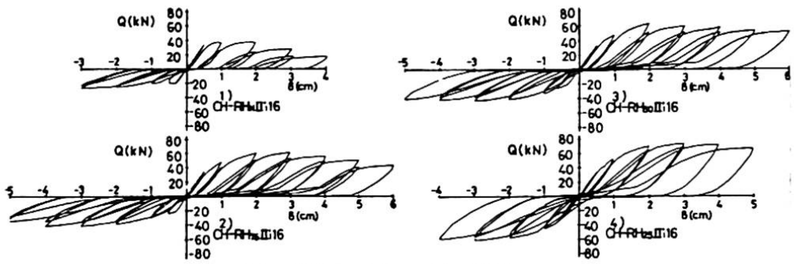


Fig. 2 Load(Q)−deformation(δ) curves

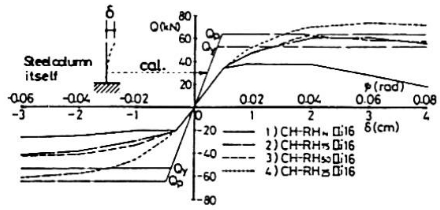


Fig. 3 Envelope curves of Q-δ relations

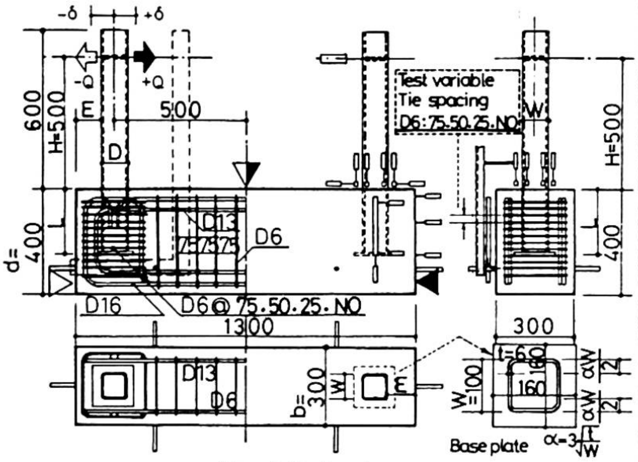
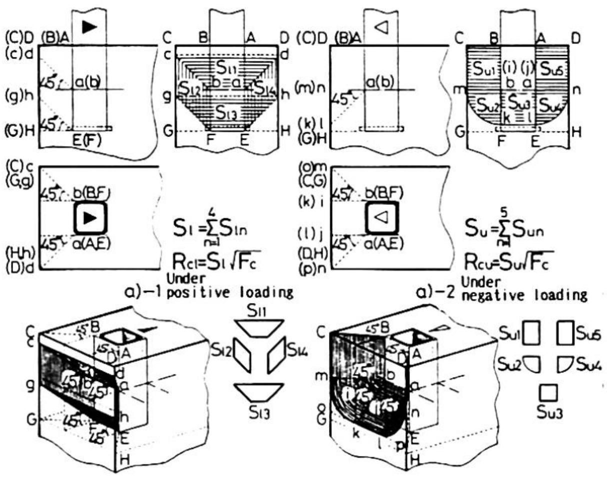
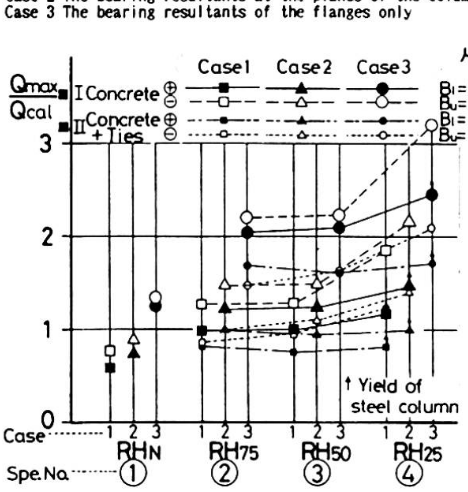


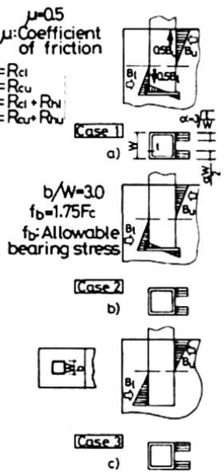
Fig. 1 Test set-up

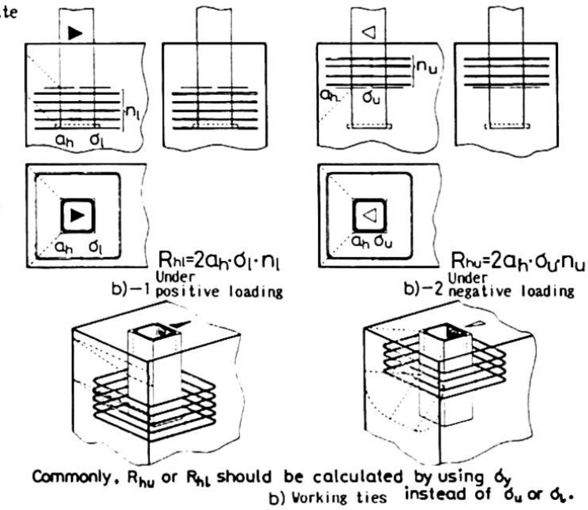


a) Punching fracture planes

Case 1 The bearing resultants and the frictional forces at the planes of the column flanges and the bearing resultants of the base plate
Case 2 The bearing resultants at the planes of the column flanges and the base plate
Case 3 The bearing resultants of the flanges only







b) Working ties

Fig. 6 Ratios of test values to calculated capacities

Fig. 5 Assumption of three resistance conditions

Fig. 4 Assumption of punching fracture planes and working ties

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1. MORITA K. et al., Experimental Studies on the Ultimate Strength of the Embedded Type Steel Column-to-Footing Connections. Trans. of AIJ, Jan. 1985.

Die Beton-Verbindungselemente der Dachträger in der Oulu-Halle

Concrete Connection of the Roof Beams in the Oulu Dome

Elément d'assemblage en béton des poutres du dôme d'Oulu

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1. ALLGEMEINES ZUR OULU-HALLE

Die Oulu-Halle ist ein Netzbewölbebau mit einem Volumen von 145 000 m^3 auf einer Fläche von 10 700 m^2. Der Innendurchmesser der Halle beträgt 115 m, ihre Höhe am First 24 m und am Rand 3 m. Der Radius des Großkreises ist 90 m und der Abgangswinkel 40°. Die Oulu-Halle ist Europas größte Gewölbehalle mit Holzkonstruktion.

Die Netzgewölbekuppel der Oulu-Halle wird von einander überschneidenden Bögen aus Kerto-Schichtholz gebildet. In das Netz eingefügt sind ebenfalls bogenförmige Sekundärträger und auf ihnen liegt eine Spundbretterschalung mit der Wärmeisolierung und einer wasserabweisenden Schicht. Das Netz besteht aus bogenförmigen, 10-12 m langen Träger. In der Spitze der Halle schneiden sich im Winkel von 60° die drei Hauptbögen, welche die Kuppel in sechs symmetrische Sektoren aufteilen. Es gibt 127 Verbindungselemente. Diesen Verbindungselementen zwischen den Träger kommt in der Dachkonstruktion eine zentrale Bedeutung zu.

2. BETONILIITOS - DAS VERBINDUNGSELEMENT FÜR KERTO-SCHICHTHOLZ IN EINEM NETZGEWÖLBE

Beim Verbinden von Holzkonstruktionen werden traditionsgemäß verschiedene Stahlelemente verwendet. Der Gedanke, man könnte hier Beton einsetzen, erscheint fremd. Bei der Oulu-Halle hat man sich jedoch für diese ungewöhnliche Materialzusammenstellung entschieden - Kerto-Schichtholzträger werden mit Betonelementen verbunden.

Als Betoniliitos bezeichnet man das stählerne, einbetonierte Verbindungselement von Kerto-Schichtholzträger, bei dem das Betonteil die Hauptbelastung, den Druck, und das Stahlteil die Zugbelastung aufnimmt. Im Betoniliitos befindet sich am oberen und unteren Ende je eine Stahlachse, die sternförmig mit sechs Stahlplatten versehen ist. An diesen Platten werden die Enden der in die sechs Kerto-Schichtholzträger eingelassenen Nagelplatten mit Bolzen-ReibeschluB verbindungen befestigt. SchlieBlich wird Betoniliitos armiert und mit Beton ausgegossen (Bild 1).

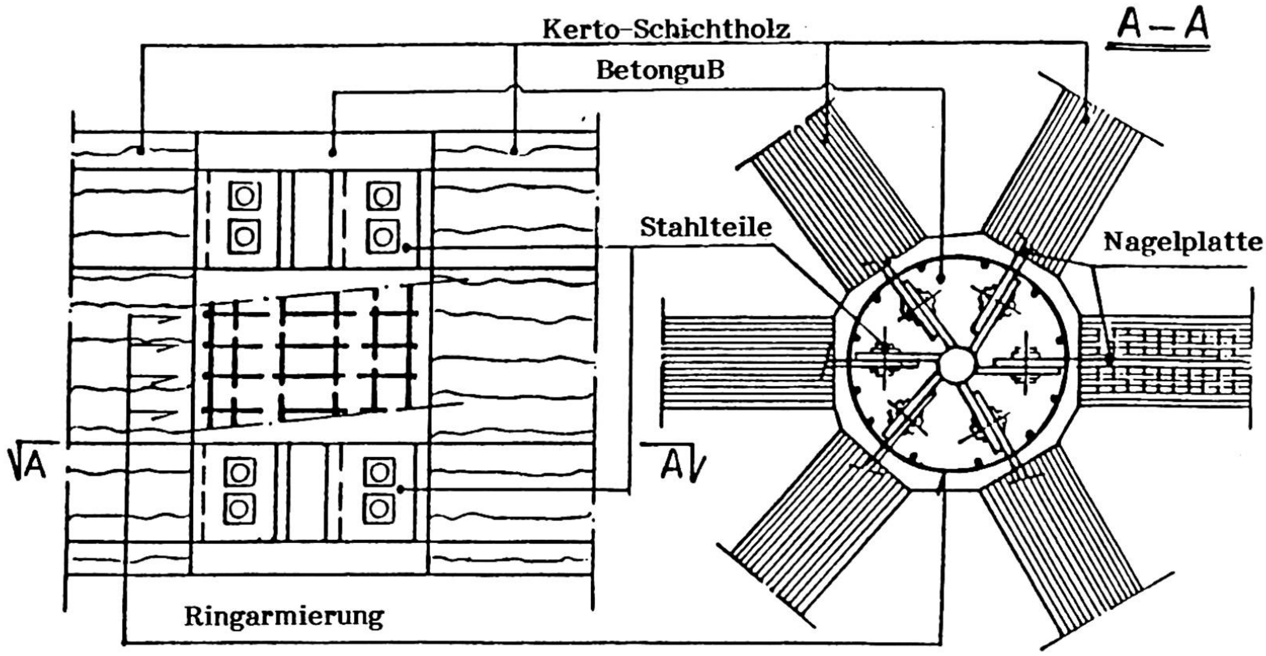

Bild 1. Quer- und Längsschnitt durch ein Betoniliitos.

Die mit Betoniliitos durchgeführten Laboratoriumsversuche haben gezeigt, daβ ein mit Verbindungselementen versehener Kerto-Schichtholzträger sowohl unter Druck- als auch Biegebelastung weniger reagiert als ein unbehandelter (Bild 2). Unter Zugbelastung waren die Werte beider Versuchsobjekte fast identisch. Aufgrund der Laboratoriumsversuche läβt sich feststellen, daβ das Verhalten von Betoniliitos unter Druck-, Zug- und Biegebelastung den Anforderungen entspricht, die an die Verbindungselemente von tragenden Balken in Netzgewölbebauten gestellt werden.

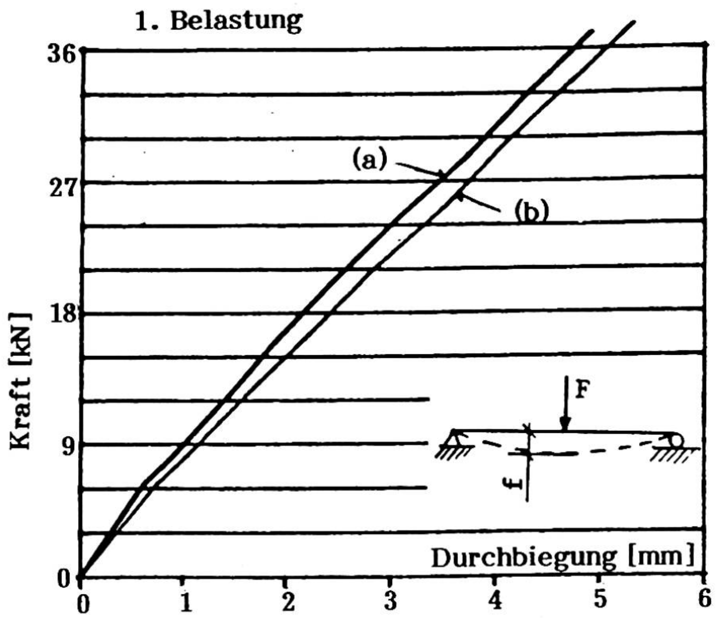

Bild 2. Durchbiegung von Kerto-Schichtholzträger in der Mitte (L = 7 500 mm) mit (a) und ohne (b) Betoniliitos, wenn die Last in der Balkenmitte aufliegt.

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