A New Test for Stud Connectors in Ribbed Slabs

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Summary

A new push test was developed to investigate the behaviour of headed studs in ribbed slabs. The main reason was that the standard push tests are not suitable for the validation of numerical models. A series of tests with the new test set-up was carried out and evaluated. The test set-up and some test results are briefly described.

1. Introduction

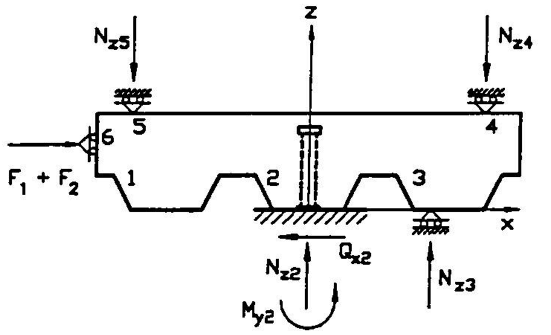
The behaviour of a composite steel-concrete beam is essentially influenced by the properties of the longitudinal shear connection. In present practice the resistance of shear connectors placed in the ribs of composite slabs are related to the resistance of a connector in a solid slab by means of one reduction factor. It is now generally agreed that this method is not satisfactory.

A research project was started to develop a numerical model for the simulation of the behaviour of a headed stud connector in the rib of a composite slab. It appeared that in the standard Eurocode 4 push tests as normally used, the boundary conditions were not unique. Therefore, a new push test suitable for the validation of the numerical model, was developed.

2. Tests: specimens, measurements and results

Contrary to the standard test procedure just one rib was tested (see Fig. 1). All loads and boundary conditions were carefully monitored. This included all support reactions (see Fig. 1), the internal forces in the stud (N_z, M_x and M_y) and the difference between the displacements of the concrete rib and the foot of the stud. This is essential for the validation of numerical models.

In total 22 tests were carried out. The parameters which were varied are: the steel sheeting (with and without sheet, with and without embossments, 'thin' and 'thick' sheet), the geometry of the rib, the place of the stud within the rib. the welding technique (through deck welded stud and stud placed in a precut hole in the sheet), the concrete strength and the hogging reinforcement.



The test results included: the shear force-slip relation, the load at first cracking of concrete, the place of the first crack (front side or rear side of the stud) and the failure mode. Material properties of the stud, the sheet and the concrete were determined.

Fig. 1 New test set-up

Most results confirmed the general behaviour known from standard push tests as described in Eurocode 4. Also, new ideas were gained which resulted in an improved insigth in the behaviour. It appeared that the steel sheeting had a significant influence on the behaviour. As well a larger thickness of the sheet as the presence of embossments increased the failure load. Both the failure modes 'tension shear failure of the stud' and 'concrete cone' were observed.

Analyzing all results some postulations about the behaviour could be made. One of them is that, for through deck welded studs, the behaviour for both failure modes is initially the same. At the moment that a crack originated at the rear side of the stud the behaviour became different, which finally resulted in a completely different failure mode.

Normally it is accepted that through deck welded studs in comparison with studs in precut holes, have higher failure loads. The new push test showed the contrary. This is probably caused by the fact that the concrete in front of the stud is completely restrained. For this reason it was found that the behaviour was quite different, although the failure mode is finally the same.

Although perhaps obvious, at small slip the headed stud connection transferred the load by stud bending and by a couple of normal forces: one in the stud and one in the rib at the rear side of the stud. The test results showed that, prior to the occurance of the maximum shear load, the full plastic moment of the stud was exceeded. Besides the plastic deformations of the stud, other non-linear phenomena were observed: buckling of the sheet, cracking of the concrete, crushing of the concrete, punching of the stud through the concrete, sliding of the concrete over the sheet and plastic deformations of the sheet caused by riding over. Most of them occurred at small slip already.

3. Conclusions

The results of the new push test are suitable for the validation of numerical models. The numerical model should be able to take complicated non-linear phenomena as described into account. Once the numerical model is able to predict the behaviour of the new push test, a powerful tool is available to determine design formulae for composite beams.

4. References

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Connection Characteristics for Joints between Hollow Core Slabs and Slim Floor Beams

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Matti Leskelä, born 1945, received his PhD in 1986 and has been carrying out research into composite structures from the early 1980's. His latest work has concerned problems of partial interaction and various shear connections in composite structures such as slim floors, composite slabs and concrete filled steel tubes.

Summary

Hollow core slabs become part of a composite flooring when they are supported on beams. When slabs are integrated with slim floor beams, a system of multiple longitudinal shear interfaces will form in which the webs of the hollow core slabs also become a shear interface. The load-slip characteristics of the connection interfaces are described so as to give an impression of their role in finite element modelling, and their typical behaviour as observed in the calculation is explained.

1. Introduction

In slim floor structures, hollow core slabs (HC slabs for short) supported on beams inevitably become a part of a composite system when grouted joints are used. Although it is a conservative assumption to neglect this interaction in beam design, it should be allowed for when designing the slabs, as it has been shown by experimental and theoretical research that the vertical shear resistance of the slabs is considerably reduced as compared with the maximum resistance of the slabs on non-flexible supports. The real structural system includes various longitudinal shear interfaces with highly non-linear characteristics, and the stiffness of the joints will decrease considerably during loading of the system, causing then a reduction in the composite interaction rate.

1.1 Discretization into layered beam elements

In order to discretize a typical system, as in Fig. 1, into beam elements, four layers of elements are required, which should be connected appropriately by coupling elements that model the behaviour of the shear interfaces (i1) to (i4), as described below.

1.2 Description of interfaces

Shear interfaces develop mainly through balancing of the longitudinal normal forces due to the bending moment in the composite cross-section, the forces to be balanced being the compressive force at the top hulls of the HC slabs, which serve as flanges to the beam, and the tensile force at the beam section. The behaviour of the top hulls of the HC slabs is similar to that of the concrete slab in contemporary composite beams, but the method of transferring the compressive force to the beam is more complex. With reference to Fig. 1, the interfaces to be distinguished are: (i1) concrete bonding to the concrete or steel surface, (i2) connection of a reinforced or unreinforced top concrete layer to the beam through a cracked or uncracked vertical interface, (i3) connection between the top and bottom hulls of the slab units, and (i4) connection of the bottom hulls of the HC slabs to the beam. The problems arising due to the composite behaviour are related to the forces transferred through interface (i3), in which the web ribs of the slabs serve as shear connectors between the hulls.

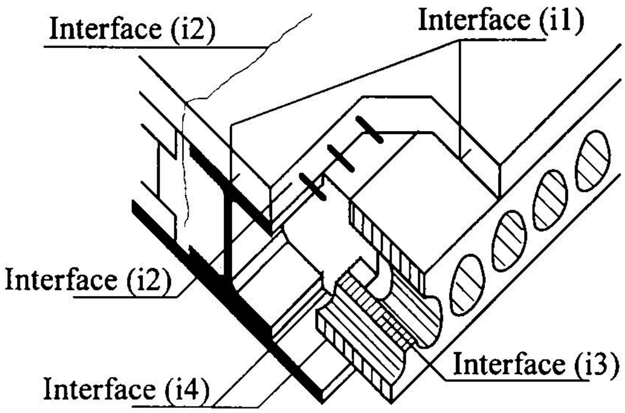

Fig. 1 General view of the various longitudinal shear interfaces activated in a slim floor in which HC slabs are integrated in the system

2. Connection characteristics

It is evident from Fig. 1 that if interface (i2) is inefficient, the majority of the force transfer from the top hull of the slab to the beam body must happen through (i3). This is always the case in structures with no reinforcement at interface (i2). The deformability of the system may be described in terms of the load-slip properties of the shear interfaces, of which (i1) and (i3) are characterized as non-ductile in the sense that the load drops considerably after the peak load is reached, and the slip required for reaching the peak load is quite small, normally much less than 1 mm. Independent of any transverse reinforcement at interface (i2), it should be characterized as ductile, as no sudden unloading will normally occur, and the same is also valid for (i4). The non-ductility of interface (i3) is critically reflected in the ability of the decking to bear loads, as a failure in the webs normally means collapse of the whole slab.

2.1 Methods to enhance behaviour of slabs

The considerable reduction in the vertical shear resistance of HC slabs is attributable to the transverse shear stresses in their webs, and any effective means of reinforcing the slabs must reduce the transverse shear stresses [1]. There are two methods for doing this, re-routing of the longitudinal shear forces and strengthening of the critical interface (i3). The practical importance of these measures is well demonstrated by the parametric studies carried out by finite element calculations, in that reinforcement of the top concrete across the beam normally means an enhancement of some 20 to 30 % and filling of the voids at the slab ends to a length equal to the depth of the voids approximately the same degree of improvement.

3. Reference

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Interlayer Bond Deterioration under Repeated Shear Load

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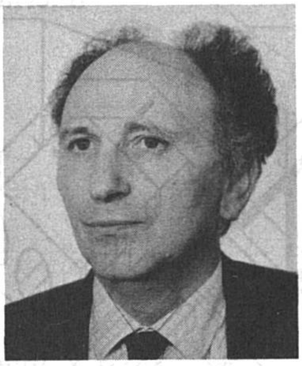

Ludovit Nasch, born 1938, received his civil engineering degree in 1962 from the Slovak Technical University (SVST), and his CSc. in 1978 from the Slovak Academy of Sciences, where he works at the Institute of Construction and Architecture (USTARCH) in the department of mechanics.

Summary

Non-reinforced interlayer connections, as they are known from the concrete composite elements, has been investigated experimentally under both, monotone increasing and repeated loading. The purpose of our research was not only to obtain the limiting stress values (or the respective interlayer displacements) at the rupture, but also to investigate the energy dissipation during the whole process of interlayer bond deterioration. Due to the repeated loading *the quick data acquisition and storing technique* has been used.

1. Specimens, loading, experimental set-up, and instrumentation

In this part of the research project made at the Institute of Construction and Architecture of the Slovak Academy of Sciences, two families of specimen, (Figure 1), were experimentally analysed under both, monotone and repeated loading. The first of the families (where the overall breaking mechanism was aimed at) comprise the full size 1200 x (70+170) x 6000 mm, (breadth x thickness x length) specimens based on the precast, prestressed wide planks KAPPA, produced by the ZIPP Ltd. - Bratislava. The second family of which only we will speak further, consists of the smaller size 200 x 200 x 600 mm three layer specimens, where the importance of such parameters as the interface roughness, the normal stress intensity, the cube strength, and the workability of the concrete, changes of the concrete mix, etc., to the overall behaviour of the interlayer connection can be investigated more readily. The surface roughness *left-as-vibrated, roughened by the sheep-leg roller* (as used for the KAPPA planks), and *trawled by the wooden lath trowel*, and three levels of the normal stress intensity *(0,0; 0,1; and 0,4 MPa)* has been chosen for analysis. The Hydropuls - Schenck loading apparatus was used to load the specimen placed appropriately between the upper cross beam and the loading piston of the loading machine. Both loading types were displacement controlled; *the monotone increasing load* by the constant loading piston velocity of 1/100 mm per second, and *the repeated loading* by the sinusoidal motion of the loading piston. Up to 2,0 x 10^6 loading cycles with the 14 Hz frequency were imposed on the specimen. The piston's lower and upper position were so adjusted, to have the upper loading force at the required level, and the lower value of the loading force to be approx. 10 or 15 kN. Altogether nine IWT 302 inductive gauges were used to measure changes of the length base across the interlayer connection, (channels 1-4), the vertical interlayer slip, (channels 6-9), and the loading piston position, (channel 10). The loading force intensity was acquired from the channel 5.

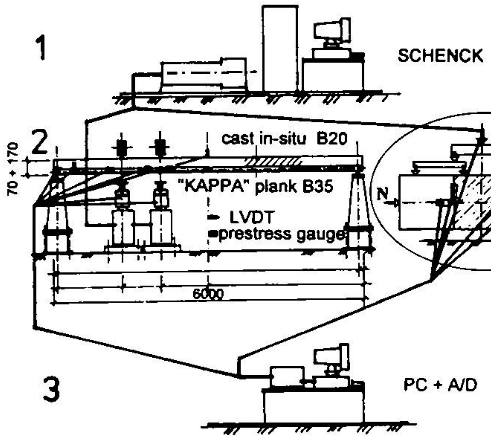

Figure 1

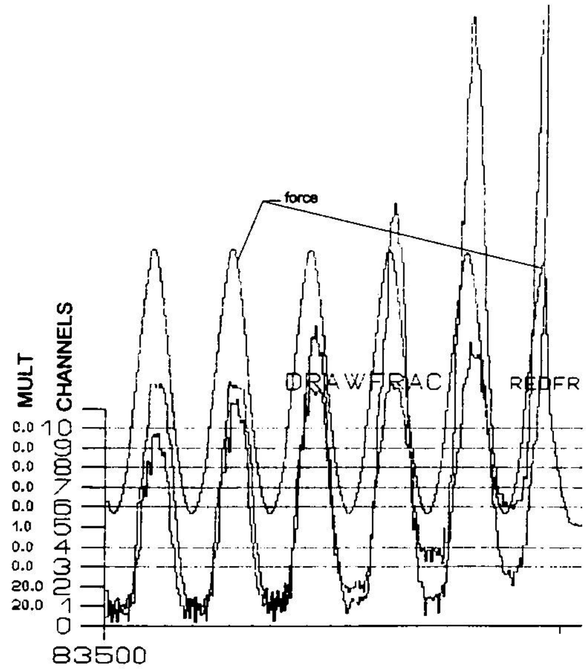

Figure 2

2. Results and discussion

On the Figure 2 we can see selected primary data as acquired during the last 6 cycles (from approx. 14000) before final rupture of the second interlayer connection of the 25/VI three layer specimen (surface roughness *left as vibrated*, and σ_n = 0,4 MPa). The whole experiment has been scanned with the sampling frequency of 600 Hz, and so the vertical line segments creating any of the depicted curves are, in this case, the 1/600 sec. apart. Length between the numbered ticks on the vertical axis equals 1/100 mm for the displacements, and 20kN for the loading force. We can see (alike the data for different normal stress, and interface roughness, and monotone load, published elsewhere, [1]) the significant increase of the measured displacements in the pre-critical part of the diagrammed data. There is an increase of displacements across the connection plane between 12 and 13 x10^{-2} mm for the last 100 cycles, (6 to 10 x10^{-2} mm for the last 10 cycles) without any significant loss of the connection's stiffness, as well as the energy accumulated during the loading phase. It should be reminded at this place, that the crack opening of about 1/100 mm is discernible by the naked eye. On the ground of the experimental evidence could be stated, that not the interlayer *adhesion* only, (accounted for in the fracture mechanics), but also the *mechanical interlock* due to the surface roughness, and the *friction* due to the normal stress intensity, together with the manufacturing conditions, play the main, if not the decisive role.

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Connection of New and Old Concrete with Bonded Reinforcement Bars

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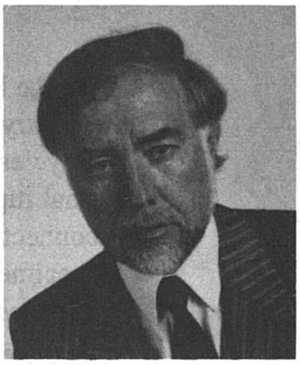


Erich K.R.Wisser, born 1940, formally head of powerhouse planning for hydro-electric plants, international civil engineering in cable way construction and anchor technology, since 1993 at Hilti Corporate Research, prime mover for anchor technology

Summary

In most cases connection of new and old concrete is performed by extending the reinforcement, often by means of post-installed bonded bars. New systems suitable of use on site have been developed, tested and introduced worldwide by Hilti Demolition and Fastening Technology, based on Eurocode 2, Design of Concrete Structures. Now engineers have a tested method to solve such problems as bridge renovation, retrofitting of industrial buildings, enhancement of earthquake resistance, etc.

1. Background Information

To get a monolithic behaviour of concrete structures cast in several parts it is necessary to establish continuity of the existing reinforcing. This can be accomplished either through cast-in-place starter bars or much more often by means of post-installed bonded rebars. Design rules for this applications have not been available up to now.

2. New connection systems

Following observation of jobsite working methods and on basis of experience with adhesive products, two connection systems were developed:

2.1 Rebar connection with HIT-HY 150

A hole is drilled close to a cast-in rebar and cleaned of dust. Adhesive is injected from the back of the hole to the surface using a dispenser. The rebar is then inserted. Owing to the thixotropic formulation of the adhesive, rod insertion can be accomplished vertically downwards, horizontally or vertically upwards.
As a hybrid system, the adhesive HY-150 combines the benefits of resins (fluidity, fast curing, strong compound) with those of cements (insensitive to humidity, post-hardening, heat-resistant) and exhibits the same behaviour as cast-in rebars.

2.2 Rebar connections with HVU

A hole is drilled close to a cast-in rebar and cleaned of dust. A foil cartridge is inserted in the hole and the rod driven through the cartridge with a hammer drill.
This modern resin gains full load capacity in a shorter time, also at temperatures lower than 0°C, has a matrix of high compression strength and shows no creep.

3. Design Concept for Rebar Connections

In cooperation with Prof. Marti [4] design rules were elaborated based on the safety concept of Eurocode 2 [1] and three failure modes:
- Limit of rebar utilisation
- Limit of adhesive bond utilisation
- Limit of concrete bond utilisation

The basic anchorage length (steel fully utilised) derived from tests ([2], [3]) corresponds with the European codes. Also, the splice length in beams show good conformity, given that the appropriate application-specific rules of EC2 are applied to the basic anchorage length (fig. 2).

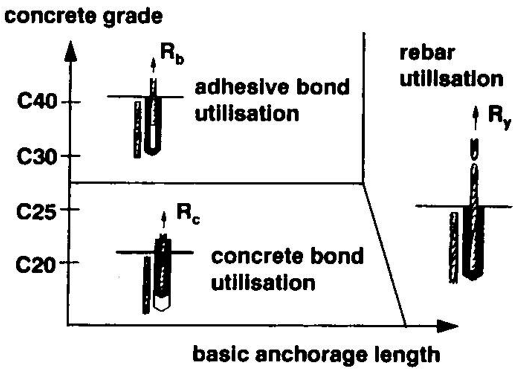
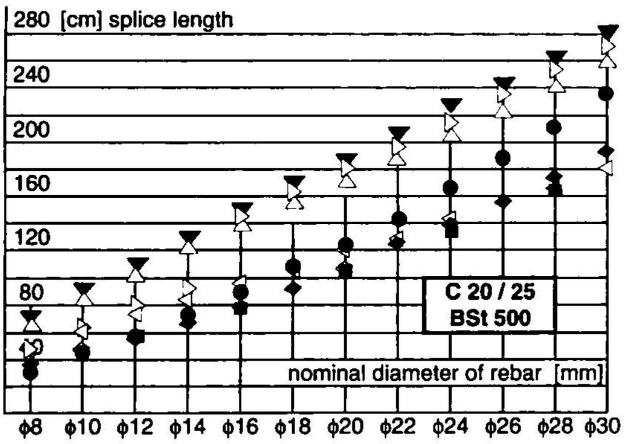


Fig. 1 Design Concept



	EU EC 2 ▼	GB BS 8110 △	D DIN 1045 ▷	A ÖN B 4200 ◆	CH SIA 162 ◁	HIT-HY 150 ●	HVU ■
ϕ8	75	69	52	37	48	30	[cm]
ϕ10	94	86	66	46	60	44	
ϕ12	113	103	79	55	72	58	58
ϕ14	132	120	92	65	84	74	
ϕ16	150	138	144	78	96	88	78
ϕ18	169	155	162	91	108	108	
ϕ20	188	172	180	106	120	122	104
ϕ22	207	189	198	121	132	144	
ϕ24	226	206	216	138	144	166	132
ϕ26	244	224	235	155	156	188	
ϕ28	263	241	253	173	168	212	164
ϕ30	282	258	271	192	180	236	[cm]

Fig. 2 Comparison of splice length in a beam at different European codes

The pull-out and beam tests verify that the two systems HIT-HY150 and HVU, result in a load-slip relationship nearly identical with that of cast-in-place reinforcing bars. For this reason they are ideally suited for the connection of new with old concrete.

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[2] Marti P.: Anchoring Concrete Reinforcement using Hilti HIT-HY150, Report no. 93.327-1, 1993
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A Design Method for Glass-Adhesive-Glass Composite Structural Elements

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Summary

The use of architectural glass in long span or high load applications is limited by the slenderness of glass plates which leads to excessive deflection. However, by using composite glass-adhesive-glass beam sections it is possible to carry greater loads, over longer spans with less deflection, Pye and Ledbetter (1997). This paper outlines current work at the University of Bath that will enable the quantitative design of T-beams fabricated from flat plates of toughened glass with a thin adhesive joint at the web-flange interface.

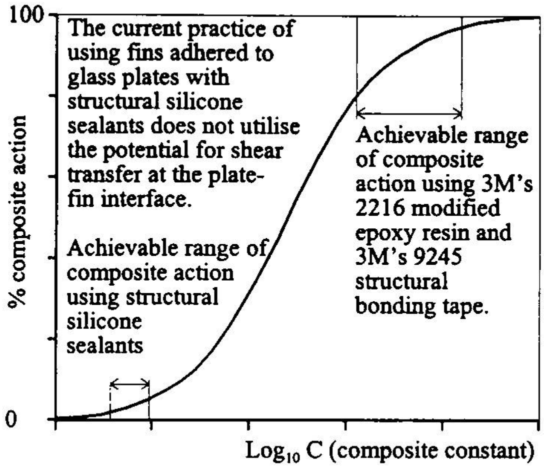
Composite Model

The authors have developed an expression which describes the behaviour of a thick-thin-thick composite with a flexible core, Equation (1). This demonstrates that the current practice of using fins to strengthen glass plates does not utilise the shear transfer at the plate-fin interface, Figure 1. It also demonstrates the increased degree of composite action which is possible using the adhesives that have been selected for this work. These are 3M 2216 B/A grey epoxy adhesive and 3M structural bonding tape 9245. The first is a flexible, two part, room temperature curing structural adhesive. The second is a new material that is applied as a tape and is heat cured to develop structural strength.

$$S_0 \frac{d^4y}{dx^4} - CS_1 \frac{d^2y}{dx^2} = \frac{d^2M}{dx^2} - CM \tag{1}$$

S_0 is the stiffness of the equivalent layered section
S_1 is the stiffness of the equivalent monolithic section
x is the distance along the beam
y is the deflection perpendicular to the span
C is the composite constant
M is the moment

The degree of composite behaviour is controlled by the composite constant, C, which is a function of the adhesive shear modulus, glass Young's modulus and the cross section geometry. However, in most practical designs it is the choice of the core material and joint dimensions that offers the greatest scope for improving composite action.



$$\text{percentage composite action} = \frac{\delta_C - \delta_L}{\delta_M - \delta_L} \times 100$$

δ_C deflection of the composite section
δ_L deflection of the equivalent layered section
δ_M deflection of the equivalent monolithic section

Figure 1 Comparing the performance of a glass-adhesive-glass T-beam constructed using a structural silicone sealant and a modified epoxy resin. Based upon equation (1).

Failure Mechanisms

The failure of a glass T-beam may be by one of the five mechanisms listed in Table 1. After having determined and appropriately factored the necessary loads and material properties, the occurrence of each mechanism must be checked

In addition to the composite failure mechanisms the glass may also fail because of very localised high stresses such as those generated by a stone impacting upon the glass. Fortunately it is possible to design against these types of failures by either over-designing the glass plates or by introducing a sacrificial layer.

A potential problem in assessing the performance of wide-flanged beams is that the full width of the flange does not work compositely with the web because of shear lag effects. By strain gauging the flange during physical testing the authors have quantified this behaviour and shown that it would be possible to approach the problem by determining an effective width as is currently practised with steel and concrete structures.

Failure Mechanism	**Assessment**
Glass bending failure	- Equation (1) may be developed to yield the maximum tensile glass stress and while this is below the surface compression of the toughened glass failure will not occur.
Glass shear failure	- The maximum shear stress may be determined in the same manner as steel sections. This must be less than the shear capacity of the glass. However, initial results from a series of punching shear tests conducted to determine the shear capacity of glass indicate that a glass shear failure is unlikely in most realistic support conditions.
Lateral torsional buckling	- The distribution of compression stresses must be such that the section is stable. It is possible to design to a reduced moment capacity by considering the slenderness of the beam, position of restraints and distribution of load. Assessing the reduced capacity has been based upon a combination of physical testing and finite element modelling.
Adhesive shear failure	- This is dependent upon the ability of the adhesive to yield and redistribute stresses. It is also sensitive to the rate of loading. Difficulties arise in quantifying the complex elasto-plastic behaviour. Current work is based upon a combination of physical testing and finite element modelling.
Adhesive tensile failure/glass plucking failure	- There may be a cohesive failure which is a function of the adhesive, an adhesive failure which is a function of the adhesive and the primer or a plucking failure which is a function of the glass. All of these mechanisms may be easily prevented by suitable joint detailing and increasing the adhesive contact area..

Table 1 A summary of the failure mechanisms of glass-adhesive-glass T-beams.

Conclusion

It is possible to predict the performance of composite glass-adhesive-glass T-beams and by applying a similar methodology it would be possible to assess the performance of other sections such as I's,π's and boxes. However, the current process of determining critical stresses is complex and would need to presented in a simplified manner if it were to be used in practice.

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Monotonic Behaviour of Fastening Systems for Sandwich Panels

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Summary

The influence of connecting system on the structural behaviour of sandwich diaphragms in pin-jointed steel frames is investigated in this paper. The importance of connections on the global response of the whole system has been evidenced through experimental as well as numerical analyses. Detailed tests on different sandwich panel connection typologies allow to set up an analytical model able to predict their monotonic performance. Such a model is useful to characterise the behaviour of connection when accurate global non-linear analyses are required.

1. Introduction

Light-weight curtain wall systems are more and more used in both industrial and civil buildings, where they can cooperate with the steel skeleton, giving rise to a composite action like a diaphragm action. Nowadays, the interest is therefore concentrated in the evaluation of their contributing effect on the structural behaviour of the building. Depending on the adopted connecting system, such panels may, in fact, provide a remarkable increasing of both lateral stiffness and ultimate strength of bearing steel frames subjected to horizontal loads. The linear analysis for infilled frames, as it is suggested in the present code (EC3-Part 1.3), allows to take account for the skin effect in terms of both strength and initial stiffness. Nevertheless, it does not allow to assess the actual ductility resources of the system as well as itsdissipative capacities. The interaction between steel cladding panels and structural framing system has been analysed within a general research project, sponsored by ECSC and developed through thecooperation between University of Naples and Italian Consortium CREA. With regards to the sandwich panels, experimental, theoretical and numerical activities have been performed [1].

2. The influence of connecting systems

Numerical as well as full-scale experimental analyses on different sandwich panel typologies connected each other and to the external frame by means of different kinds of connecting systems have emphasised that the contribution of the connections to shear flexibility of the panel is generally prevalent and plays a fundamental rule on the overall behaviour of infilled frames [1,2]. In addition, it has been shown that the actual behaviour of shear diaphragms, as the shear load increases, is more and more non-linear, depending on the behaviour of the adopted connecting system, being the major source of non-linearity just concentrated in connection elements. In order to develop an accurate analysis, the complete shear load-lateral displacement relationship should be therefore determined. The aim can be pursued by means of adequate non linear numerical tools. The proper structural characterisation of connections becomes therefore the starting point for the correct interpretation of the response of steel shear walls.

3. The behaviour of connections

The actual behaviour of connecting systems has been investigated by means of experimental tests [3,4]. Sandwich panels with trapezoidal, embossed as well as plane external sheets has been tested. Besides, a special panel with an internal steel reinforcing profile has been considered. The analysis have concerned both panel-to-panel connections and panel-to-external frame connections With reference to the former, screwed connections and bonded connections, using glue and biadhesive bands has been analysed. Riveted and welded connections have not been taken into account. The *riveted connections*, in fact, have not a good behaviour under cyclic loads because they present a brittle mechanism of fracture, while the *welded* ones are not applicable owing to the very thin thickness of panel sheets.
The collapse mechanisms have found obviously to be strictly related to the connection typology. As regard to *screwed connections*, the sheeting resistance is demonstrated as the weak point of the joint, being the collapse always characterised by a large holeovalisation. The connection for corrugated sheets have shown a bad performance due the impossibility to connect both sheets on the two sides. On the contrary it is to emphasise the good behaviour of reinforced panel connections which allow to rely on both strong resistance and stiffness.
As far as *bonded connections* are concerned, the collapse phenomenon has been characterised by the slipping between the two opposite parts of the specimens, which follows a more or less sudden disjunction. In particular the connections with *biadhesive band* has provided a too low ultimate load, which makes the good ductility qualities useless. The *glued* bonded connections has instead shown a good ultimate strength value, joined to a very high brittle behaviour.
Based on the previous experimental results, two panel typologies have been selected for testing panel-to-external frame connecting system: the embossed sheet one and the flat sheet with reinforcing cold-formed profile. As far as connecting system typologies are concerned, only screwed connections using self-tapping screws have been considered.
The collapse of connection was characterised by a large holeovalisation, with bearing and tearing of the sheets. The maximum load value is found to be strictly depended on the number of resisting sheets, as well as on their thickness. The corresponding load-slip curves are typical for this failure mechanism, showing the great displacement capacity as consequence of sheet holeovalisation.

4. Further development

In order to develop accurate global non-linear analyses contemplating the diaphragm effect of sandwich claddings, a mechanical model able to predict the monotonic behaviour of connection typologies should be set up. Such a model could be based upon mechanical properties of connections as pointed out from experimental test results and generalised by using a simple appropriate analytical formulations.

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Long Term Behaviour of Composite Concrete Structures

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Summary

The numerical simulation of the widened Javroz bridge deck demonstrates that it is essential to consider the early age behaviour of the composite deck consisting of new and old concrete layers. Actually, internal stresses mainly due to restraint of thermal shrinkage may decrease the strength of these hybrid structural elements and affect their durability. The most effective measure to limit the early age damage is to decrease the maximum temperature of the new concrete during hydration by a system of cooling pipes.

1. Introduction

The 45 year-old Javroz bridge in Switzerland, a 170m long concrete arch bridge spanning 90m, will be improved to account for future traffic needs. The existing deck slab will be modified by an additional concrete layer and larger cantilever slabs (fig.1). The challenge being to restore a service life comparable to that of a new structure. The long term behaviour and thus the durability of the modified slab must therefore be studied by considering the composite action of the new section consisting of two concrete layers of different ages.
According to [1], the adherence between old and new concrete layers decreases continuously, and the hybrid system may fail after 17 - 20 years of service. This can be explained by the fact that the influence of the early age behaviour of the new concrete is usually disregarded, and the only criterion considered is the short term adhesive strength between the two materials.

2. Description of the domain studied

During the lifespan of hybrid structural elements, three stages can be distinguished (fig.2). First, during *hardening of the new layer*, the effects of cement hydration must be considered to determine the internal stress state mainly due to restraint of thermal shrinkage caused by the old concrete support. These internal stresses are at the origin of cracking of the new concrete layer which may affect strength and durability. To evaluate this effect, the *initial damage coefficient* α can be defined as $\alpha = \left(1 - \frac{resid.\ strength}{init.strength}\right)$; small reduction of α (curve $\alpha_2 < \alpha_1$) leads to a significant extension of lifespan. Secondly, during *service life*, effects of temperature variation as well as dynamic and fatigue action due to traffic loading are superimposed to the initial stress state and play a major role in damage propagation. Finally, *failure* of the hybrid system is determined according to ultimate state criteria.

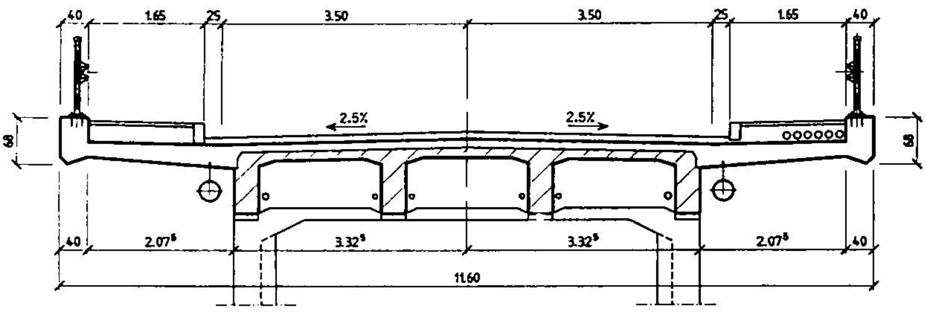

Fig. 1 Cross section of the widened Javroz bridge deck

3. Analysis of early age behaviour of a hybrid concrete bridge deck

The early age behaviour of the modified bridge deck is analysed by a numerical study [2]. The initial stress state and the likelihood of crack formation are determined [3], taking into consideration both the hydration heat release and variable environmental conditions.
For three different construction sequences, the initial state of internal stresses is obtained in terms of parameters such as cement content, temperature of fresh concrete, duration of cure, thickness and modulus of elasticity of the new concrete. The results show that reducing the cement content by 50 kg/m^3 has the same effect of avoiding early age damage of the hybrid deck as pouring of fresh concrete the temperature of which has been lowered by 5°C. The most effective measure is to decrease the maximum temperature of the new concrete during hydration. Numerical simulation shows that a system of pipes for cooling water placed in the new concrete layer allows for sufficient temperature decrease to reduce significantly internal stresses (fig.3). Without specific measures, the coefficient α is 0,75 600 hours after pouring the new concrete. Comparatively, with the use of cooling pipes in the young concrete, this coefficient is reduced to 0,30.

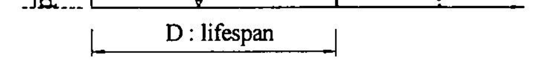

Fig. 2 Damage curve of composite concrete structures

Fig. 3 Evolution of out of plane stress σ_{zz} in the cantilever slab versus tensile strength f_{ct}

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