New hybrid bridge decks using carbon fibres, very high performance concrete and wood

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ABSTRACT

The studies presented are aimed at improving the mechanical performance of wooden bridges. The mechanical performance of mixed or composite decks made of timber, concrete and carbon fibre reinforced plastic (CFRP) are studied. More precisely, the research concerns the evaluation of the rigidity of the connection systems between the concrete slab and the glulam beams and the contribution of the carbon-fibre reinforcement. The experimental study of a set of connectors covering most types is first presented. The way in which each type functions and the different failure methods are identified. It is the adhesive connection system which affords the best rigidity, while offering the highest ultimate strength. A method designed to calculate the stiffness of a connection system based on bending tests is presented. Lastly, two bending tests on UHPFRC-Timber-CRFP mock-ups have demonstrated that the carbon fibres play a positive role with respect to ultimate limit states.

Key words: Beam, Timber, Concrete, Connector, Carbon, Composite, Deck, Mixed bridge, Glulam, Stiffening

1 INTRODUCTION

At ENPC's Laboratory for the Analysis of Materials and Identification (LAMI), one of the research projects aimed at limiting the use of non renewable materials concerns the design of bridges using timber. The research, developed in an ENPC doctoral thesis (Pham, 2007), is part of the European Project NR2C (New Road Construction Concepts), in which one of the subgroups is examining civil engineering structures in forty years time.

Several foreign research projects concern the design of mixed bridges consisting of ordinary reinforced concrete slabs connected to glulam beams [Ahmadi et al. 1993, Gattesco 2001, Clouston et al. 2004, etc.] Other research considers reinforcing the wood with a composite in tension zones in order to increase the mechanical performance characteristics. Dagher shows experimentally that the stiffness is increased by 10% to 20% [Dagher et al. 1998, Dagher et al. 2002]. Chajes, on the other hand, finds that in comparison with a deck without shear connectors or stiffening, a 500% increase in the stiffness of a wood-concrete-carbon connected deck can be obtained [Chajes et al. 1995]. The stiffening in the tension zone enables the breaking loads to be increased. Plevris shows, for example, that it enables the bending strength to be increased from 50 to 100% [Plevris et al. 1992]. According to Dagher, it reduces the variability of the stress in the tensile zone due to local defects in the wood [Dagher et al. 2003]. Other similar research concerning stiffening with other composites such as glass-fibre reinforced plastics (GFRP) has also been carried out [Weaver et al. 2004, Davids et al. 2005].

While the above research is concerned with improving the strength of the tension zones, it also seems interesting to try to improve shear transmission between the concrete part and the wood. In all the existing publications, the wood-concrete connection is imperfect since the slab and the girders are separately subjected to bending stress. In the case of a perfect connection, the concrete is subjected to compressive stress. Assuming that this solution is possible, it can be imagined that ultra-high performance fibre-reinforced concrete (UHPFRC) is a promising solution, since their compressive strength can reach 90 MPa. In addition to the low dead weight of these structures, which allows for prefabrication, the protection of the wood provided by the concrete slab is a design which corresponds to practice in terms of durability [Guide SETRA, 2007].

As a result, a certain number of factors indicate that the design of composite UHPFRC-wood and composite UHPFRC-wood-CFRP bridges should be studied.

In this article, we will be considering the connection between wood and high performance and ultra high performance concrete components, and strengthening of these mixed structures with carbon. The first part is devoted to the presentation of bending tests on wood-HPC composite girders and a method for calculating the stiffness of a connection system based on bending tests. The study of a set of connectors covering most of the different types is presented. The second part validates the technique selected for the UHPFRC-wood connectors and provides practical information on surface preparation techniques. The last part of the article presents the bending test results for a HPC-wood-CFRP composite girder carried out with one of the connector systems studied previously. A calculation of normal maximum shear stresses taking the properties of the wood-concrete interface into account is proposed. The mechanical advantage of CFRP is highlighted.

2 STUDY OF THE WOOD-UHPFRC INTERFACE

21 Experimental procedure

The behaviour of the interface is traditionally evaluated on shear tests [Gattesco, 2001; Clouston et al. 2004; Tommola et al. 2005]. The sample is designed so that two shear planes, symmetrical with respect to the load applied, are subjected to stress during the test. The principle is simple, but it is not as easy to put into practice as expected. In the case of linear connectors or adhesive bonding, stress concentrations appear at the edge [Caron et al. 2002] which must be taken into account in the analysis. In order to carry out exploratory tests that are both simple and close to the phenomena to be studied, it was decided to conduct three-point bending tests. Nineteen girders were made. The girders consist of an upper layer of very high performance fibre-reinforced concrete, connected to a glulam beam forming the lower layer. Each type of connector was tested on 2 girders, except for the adhesive bonded connection which was tested on 3 girders.

211 Material properties

The wooden part is a glulam beam class GL28 (characteristic bending strength of 28 MPa), 135 mm high, 90 mm wide and 1.25 m long. The longitudinal modulus was measured and found to be 12 GPa. The concrete part is a very high performance fibre-reinforced concrete (VHPFRC) with an average strength at 28 days of 120 MPa, containing 1.5% 14 mm metal fibres. The theoretical modulus is 45 GPa (± 3 GPa). The concrete is 50 mm thick and 90 mm wide, like that of the wooden beam. The relative proportions of the two materials were adjusted to maximise the shear stress at the interface. The connection systems were the subject of a bibliographical study. Several types of connection systems are distinguished, i.e. notched connections [Deperraze, 1998, Martino, 2005], those using mechanical components such as nails [Ahmadi et al. 1993), metal tube connections [Alain, 1988, Gaymond, 1995], metal plates [Baton et al. 2005] or glued-in connections [Pincus, 1970, Brunner et al. 2000]. In the above, three different groups can be defined: local connectors, linear connectors and glued-in connectors. This study concerns 6 types of connectors from all 3 groups (Figure 1):

- Local connection by dowels or tubes. The dowels are 15 mm diameter cylindrical bars. The tubes have an outside diameter of 30 mm and are 1.5 mm thick. These connectors are glued in holes prepared in the wood at a depth of 50 mm and are anchored in the concrete over a length of 30 mm;
- Expanded metal strip. The expanded metal strip is inserted into a groove in the wood to a depth of 50 mm and glued with epoxy. It is cast into the concrete to a depth of 30 mm.
- Spike plate strip. The wooden beam is cut in two lengthwise and the spike plates are nailed to each part. The two parts are then assembled and glued with epoxy and held in transverse compression prestressed threaded dowels. The penetration depths are the same as above.
- Perforated metal strip. The design is the same as above. The expanded metal is replaced with a 1.5 mm perforated metal strip.
- Gluing. The concrete is cast on the wooden beam and the forms removed once it has hardened. Gluing is carried out 7 days after casting of the concrete. A layer of epoxy glue about 1 mm thick is applied to each part to be glued. The two parts are then assembled and kept in contact by light pressure for 24 hours. The trial bodies are then placed in an over at 45℃ for 24 hours. The tests are conducted 4 days a fter gluing.

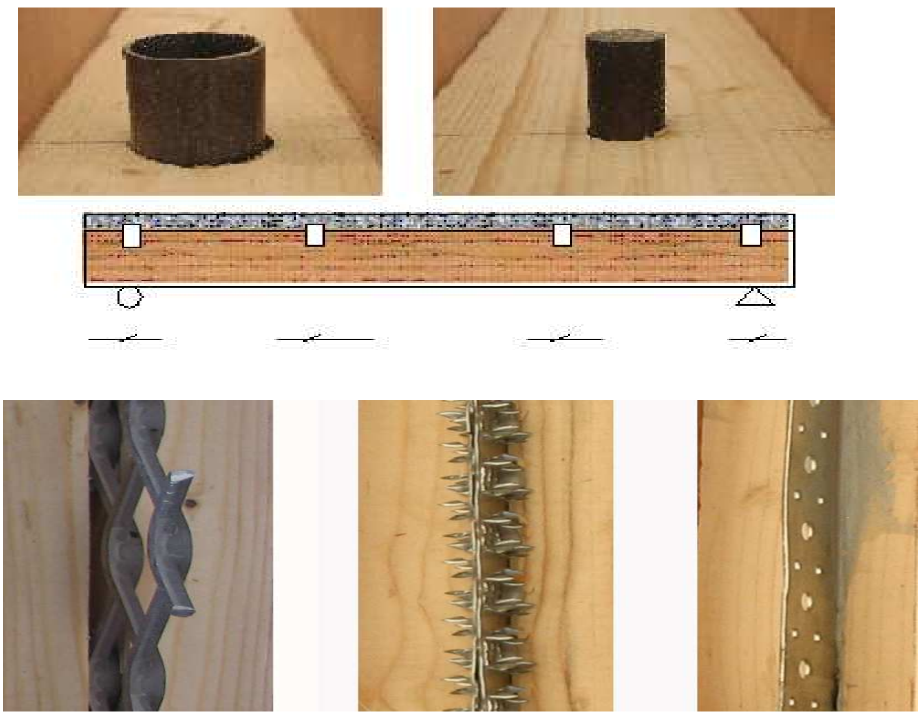


Figure 1. Examples of connectors (from left to right and top to bottom) Tube (PTu), Rod (PTi), Expanded metal (PMD), spike plate (PPAP), Perforated strip (PR).

212 Metrology

The three point bending test is carried out on the beams using a fixture installed on a LAMI press (MTS M20), equipped with a 100 kN load application device. The distance between points is 1 m. The tests are carried out during displacement. The loading speed is 2 mm/minute while the unloading speed is 3 mm/minute. The test comprises three main phases (Figure 2). The first phase consists of three load/unload cycles from 5% to 30% of the breaking load F_{rup} (value determined theoretically). It enables the stiffness of the beam to be determined while ignoring the initial cycle. The second phase consists of three load/unload cycles from 5% to 60% of the breaking load. This phase is used to check that the behaviour of the beams, and particularly the connectors, remains with the elastic field. The last phase is steadily increasing until failure takes place. It is used to determine the ultimate load to failure and the failure mode of the beams (in concrete, wood and at the interface).

Five LVDT displacement sensors are arranged to measure the deflection in the middle of the beam and the relative slipping at the interface at both ends (Figure 2). Compression of the supports, measured by the sensors, are subtracted from the deflection measured at the centre. The beams are identified according to the type of connector and a series number.

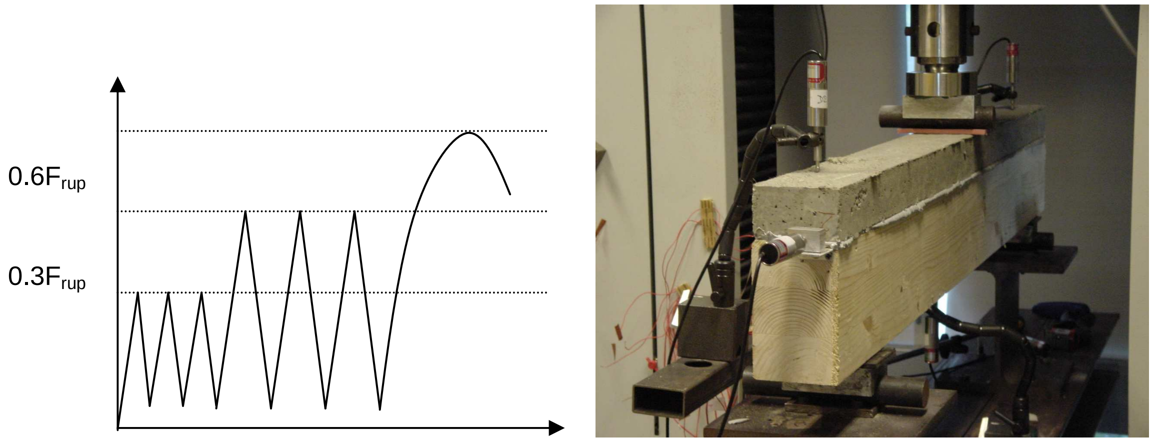


Figure 2. Loading cycles versus time with displacement control

22 Results

221 Failure modes of beams

The observations made enable four main failure modes to be determined for the beams. Failure can occur suddenly at the interface due to detachment of the connector from the wood (mode M1) or due to shearing of the wood at the interface (mode M2). The loss of adherence between the two materials in modes M1 and M2 creates tensile stress in the concrete. Failure of the concrete rapidly ensues at mid-span. Another failure mode develops as the result of a gradual reduction in the stiffness of the interface, which finally leads to failure in the connection or by tensile stress in the concrete (mode M3). Damage of the interface is due to local detachment or deformation of the metal parts. Failure of the wood due to tensile stress or shear stress (mode M4) can also be observed.

Beams	PMD31 -1	PMD31 -2	PMD43 -1	PMD43 -2	PMD51 -1	PMD51 -2	PMD86 -1	PMD86 -2	PPAP -1	PPAP -2
F failure (kN)	66	72	52	40	73	81	61	55	90	70
Failure mode	M1	M3	M1	M1	M3	M3	M1	M3	M3	M3

Beams	PR-1	PR-2	PTi-1	PTi-2	PTu-1	PTu-2	Pco-1	Pco-2	Pco-3
F failure (kN)	51	55	70	57	59	80	86	82	74
Failure mode	M3	M3	M4	M4	M4	M4	M2	M2	M2

Table 1. Failure modes

A summary of failure modes is presented above (Table 1) and, for each type of connector, a representative example of curves describing development of the load as a function of the deflection (figure 3). Failure modes M1 and M3 occurred for linear connectors, i.e. the expanded metal (PMD), metal strip (PR) and spike plate (PPAP). Failure mode M4 occurred for local connectors, i.e. the dowels (PTi) and tubes (PTu). Failure mode M2 occurred for the glued solutin (Pco). Failures can be either ductile or brittle. The deflection at failure is high for local connectors. Deformation of the interface is due to local compression of the wood, followed by bending of the dowels. This mechanism can absorb a large amount of strain (figures 3a, 3b). Failure of the glued connection is sudden (figure 3f). Once detachment begins, propagation of the detachment surface is rapid. Failure of the interfaces in the can of expanded metal can be sudden or gradual, depending on whether failure begins in the glue plane or the actual connector. The average breaking load is 80 kN. It is observed that failure takes place at the interface in the case of wood, which means that the ultimate strength of the connection is not as high as that of the shear strength of the wood.

222 Elastic behaviour of beams

The slipping effect at the interface affects the stiffness of the structure. When this effect exists, it is said that the connection of the composite structure is imperfect or partial. The first elastic theory that takes this effect into account was developed by Newmark [Newmark, 1951]. A version of this theory, included in a simplified manner in the Eurocodes, consists in calculating, under a sinusoidal load, an equivalent stiffness (El)ef. This depends on the stiffness of the interface, K, which relates the shear force to the relative displacement interface (slipping) of the two materials. The value of K is assumed to be constant along the beam, which is a very rough assumption. The solution is then extended to all the load configurations. The method is described in Eurocode 5 [Pr EN 1995-1-1].

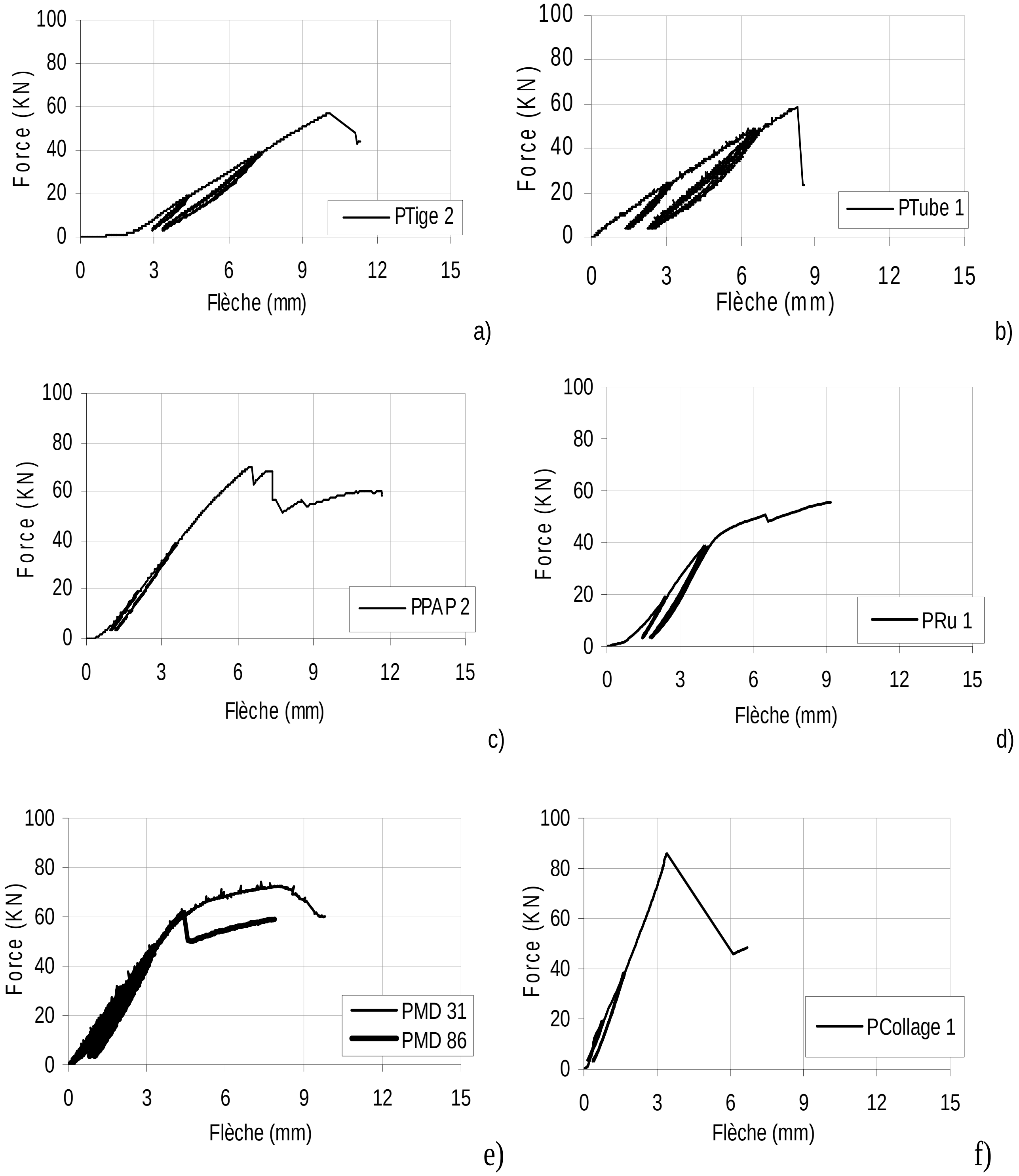


Figure 3. Examples of results during bending (Force = load, fleche = deflection)

a) Dowel b) Tube c) Spike plate d) Metal strip e) Expanded metal f) Gluing

The effective stiffness (EL)ef, of the beams in the study was calculated from the linear part of the experimental load/deflection curves, taking the shear strain into account. In the case of short wooden beams with a low shear modulus - about 700 MPa - it is essential to deduce the shear strain for the experimental deflection. The deflection is written as follows:

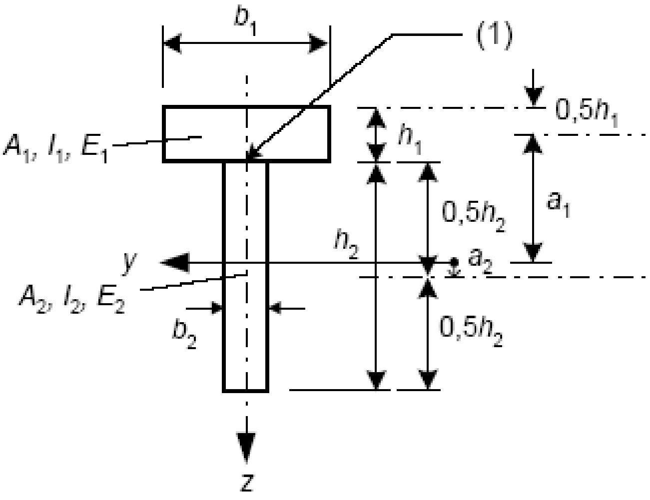
$$f = \frac{Pl^3}{48(EI)_{ef}} + \frac{Pl}{4(GS)_{ef}}$$

Where (GS)ef is the effective shear stiffness of the beam. The effective stiffness of a composite beam is considered by Batoz [Batoz, 1990] to be the sum of the stiffness of each layer. Nguyen [Nguyen el al. 2005] however shows that the effective stiffness for a perfectly adhesive bonded two-layered beam, approximated by the following:

$$(GS)_{ef} = \frac{(h_w + h_c)^2 b}{\frac{h_w}{G_w} + \frac{h_c}{G_c}}$$

Where hw, hc, b, Gw, and Gc are respectively the wooden parts, the concrete parts, the beam width, the shear modulus of the wood and that of the concrete.
The values of K are then identified using the formula proposed in standard PrEN 1995-1 (Figure 4) in which K is included in γ2.

$$(EI)_{ef} = \sum_{i=1} \left(E_i I_i + \gamma_i E_i A_i a_i^2 \right)$$



$$A_i = b_i h_i$$

$$I_i = \frac{b_i h_i^3}{12}$$

$$\gamma_1 = 1$$

$$\gamma_2 = \frac{1}{1 + \frac{\pi^2 E_2 A_2}{K l^2}}$$

$$a_1 = \frac{\gamma_2 E_2 A_2 (h_1 + h_2)}{2 \sum_{i=1} \gamma_i E_i A_i}$$

$$a_2 = (h_1 + h_2) - a_1$$

Figure 4. Magnitudes used to calculate the bending stiffness as per EN 1995-1-1

The variation of (El)ef as a function of K is given in figure 5. There is a clear distinction between the 3 groups of connectors tested. The first comprises local connectors which have an interface stiffness of less than 150 MN/m/m. This "flexibility" of the interface is known due to the way in which the connector presented above functions. The second group concerns linear connectors. The values of K range from 150 t 1200 MN/m/m. The adhesive bonded system offers between stiffness values: K > 1500 MN/m/m. The deflection is similar to that calculated for a perfect connection. The adhesive bonded connection therefore has been performance characteristics both in terms of stiffness and ultimate load.

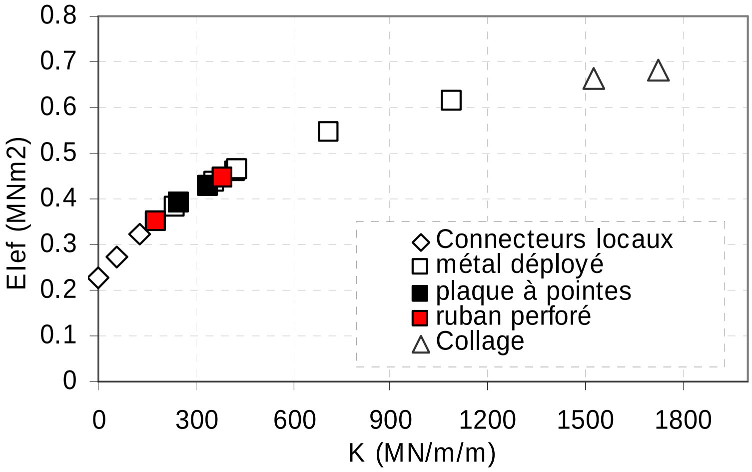


Figure 5. Stiffness (EL)ef as a function of the interface stiffness K as per the EN 1995-1-1 model for the values of (El)ef measured.

223 Interface stiffness of a set of parallel connectors

In a beam structure, the number of lines of connectors is generally limited to the number of beams. However, wooden deck bridges exist that are made of juxtaposed beams, all prestressed transversally. In this design, the longitudinal joints between the beams can be used to insert linear connectors. This was the technique explored through two tests conducted on 2 x 2 m long beams, one consisting of solid planks and the other of half-planks. These components, under which pultruded carbon flats are glued, are juxtaposed in order to produce a beam-deck component to which is connected a HPC slab [Jouslin de Noray 2005] (see § 4 below for details about the materials). One of the beams has 6 lines of connectors and the other 9 lines of the same connectors. The connection between the wood and the concrete is made by expanded metal connectors (figure 6), glued between 2 planks (or half-planks) up to a height of about 10 cm, and cast into a 5 cm thick cast-in-place compression slab up to a high of 4 cm.

The results of the two tests were studied to ascertain the effect of the number of lines of connectors on the overall stiffness of the interface. In the present case, the number of lines was varied by adjusting the thickness of the wooden components, since the expanded metal was glued between two planks or half-planks. The median value of the stiffness field calculated in the previous study on the 1 metre beams is k = 500 MN/m/m for the expanded metal geometry considered. The respective values of K' = 6 k and K" = 9k were introduced into the calculation of the stiffness of the 2 beams with 6 and 9 lines of connectors. Calculation of the stiffness of the beams takes into account the carbon by considering it to be an equivalent layer of wood whose thickness if that of the carbon multiplied by the ratio of the modulus of the two materials. The results are given in figure 7. The theoretical slopes are compared with the experimental slopes between 5 and 15 kN, i.e. in one of the elastic behaviour fields of the test bodies and eliminating the non-linear foot of the curve whose shape is due to compression of the supports. It is observed that the calculated and experimental slopes are similar, which would indicate that the overall stiffness of the expanded metal connection is proportional to the stiffness of a line of connectors and the number of lines. This result suggests how it might be possible to obtain a high stiffness interface between the wood and the concrete, which will be positive for the overall structural behaviour, using connectors with medium to low stiffness.

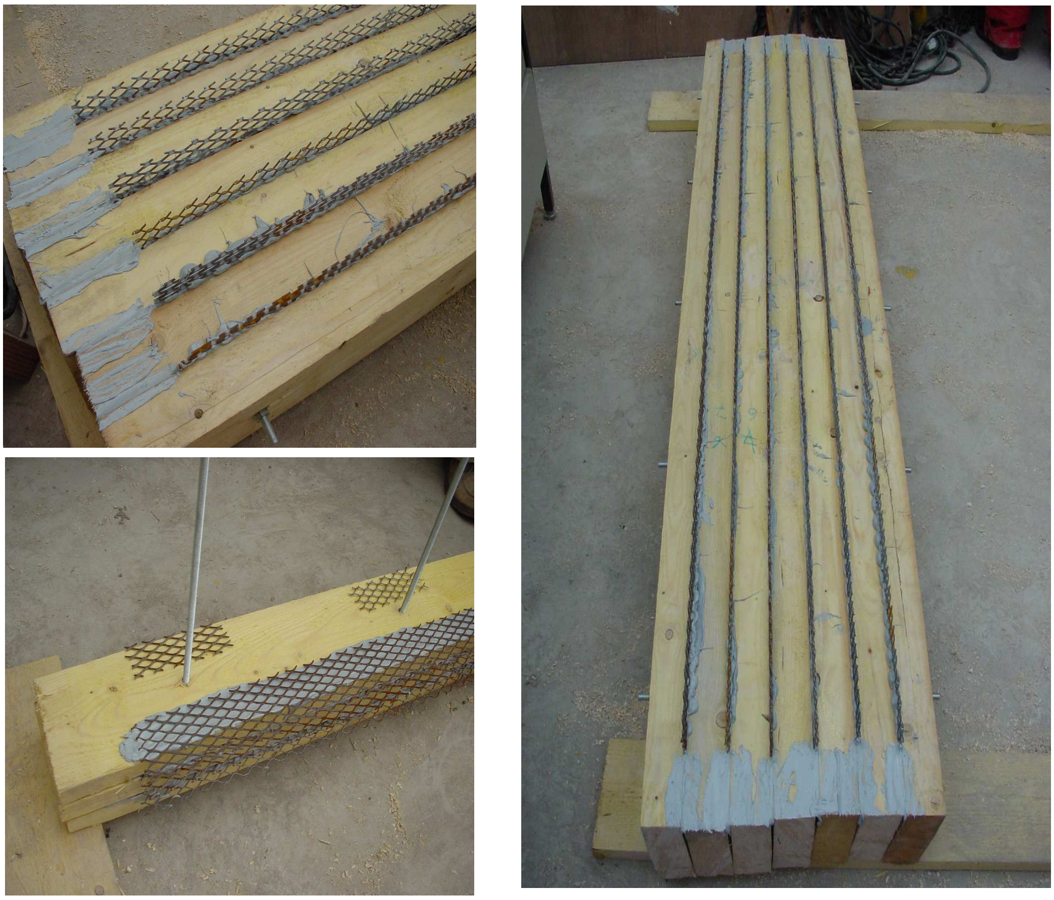


Figure 6. Preparation of connection of a composite slab

Deflection

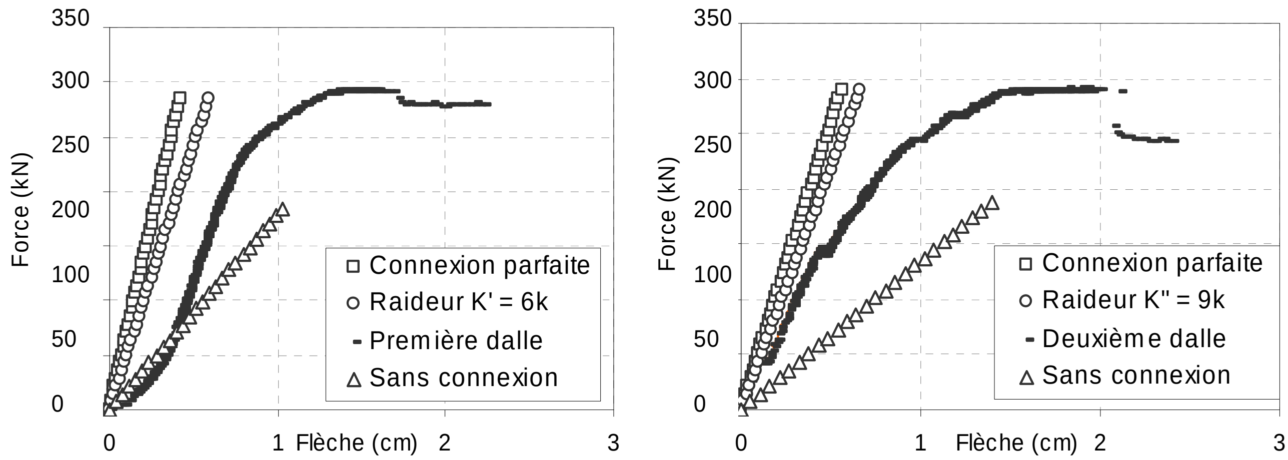


Figure 7. Results of two bending tests on two slabs (left, width 400 mm and 6 lines of connectors, right, width 300 mm and 9 lines of connectors.

3 VALIDATION OF GLUING TO UHPFRC

Having validated adhesive bonding assembly with a HPC, the use of a UHPFRC, in the present case BSI®CERACEM, required an additional study aimed at evaluating the performance of the glue according to the surface treatment of the concrete. The UHPFRC, which contains 2.5% fibres, has a characteristic strength of 165 MPa at 28 days and a characteristic tensile strength of 8.8 MPa. The glue used is Sikadur 30 epoxy glue entitled to the NF label for structural gluing. Adhesive bonding tests were carried out on additional 1 m composite beams. UHPFRC components were made by adhesive bonding in formwork, the bottom of which forms the upper horizontal surface of the wooden beam, which is protected by a plastic-coated adhesive. The adhesive gives a “glazed” finish to the concrete surface. No release agents are applied to the formwork in order to avoid greasy products from coming into contact with the glue. Before gluing, the surfaces are cleaned with a solvent-soaked rag. Double gluing, i.e. application of the glue to both surfaces to be glued, is practised. A first series of tests consisted in gluing the UHPFRC components directly to the wood without treating the concrete surface, 7 days after casting of the concrete. This series was not conclusive since the concrete rapidly started to come away at the end of the beams. This is due to drying shrinkage of the UHPFRC. Although it is considered to be very low - in the order of 10-4 for a relative humidity of 50% according to Le Roy [Le Roy et al. 1999], it nevertheless resulted in the creation of an upward curve, which the adhesive bonded connection to a small "glazed" surface was not able to prevent.

A second series of test were conducted after polishing the surface of the concrete with a diamond disk and carrying out adhesive bonding for 28-day old concrete so that any shrinkage after gluing would be very low. The tests were conclusive because the connection withstood a load of 100 kN corresponding to a shear stress of 4.7 MPa at the wood-concrete interface.

A third series of tests is scheduled with sand-blasting of the UHPFRC surface.

4 REINFORCEMENT OF WOOD WITH CARBON FIBRES

Here we will examine the carbon fibre reinforcements used for certain beams in the previous study. With the aim once again of using wood to design bridges, the possibility is also studied of reinforcing wood by gluing on composite and especially a carbon-fibre based composite.

Carbon fibres are available commercially in the form of pultruded strips of unidirectional (UD) or bi-directional (BD) fabrics. The pultruded strip has the advantage of having a high fibre content of nearly 70%. Its modulus is 160 GPa. It is generally used to reinforce concrete (or steel).The glues are usually of the epoxy type. Wood can be reinforced using similar techniques. Another method already tested consists in inserting carbon-fibre flat sheeting between the pieces of wood while compressed in a standard glulam production assembly.

The UD fabric is used by superposing layers glued with an epoxy resin, with gluing of each layer. The composite module obtained is about 100 GPa. To increase the fibre density, pressure should be exerted on the composite during hardening, by means of a vacuum. The sheeting used to create the vacuum is flattened against the reinforcement layer and exerts a uniform pressure on its entire surface. This technique, tested at ENPC, is the subject of current research (Figure 8).

Figure 8. Example of a glulam beam reinforced with vacuum-glued fabric (ENPC)

41 Experimentation

The test bodies are presented in paragraph 2.2.3 above. The geometrical details are given in table 2 and the effective properties of the materials in table 3. The tests are 3-point bending tests. The distance between supports is 1.70 m. LVDT displacement probes are placed in the middle of the slab to measure the deflection and at the ends to measure slipping between the wood and concrete.

Test body	Wood (mm)	Carbon (mm)	Concrete (mm)	Connectors
Beam 1	7 planks 150x56	7 flats 1.7 x 52	Slab 50x400	6 rows of expanded metal
Beam 2	10 half-planks 150x30	6 flats 1.7 x 52	Slab 50x300	9 rows of expanded metal

Table 2. Geometry and components of the 2 test bodies

Material	Bending strength (MPa)	Compressive strength (MPa)	Tensile stress (MPa)	Young's modulus (MPa)	Shear modulus (MPa)
Solid wood	22	20	13	10 000	630
Concrete		90	5	51100	21000
Carbon			2500	150 000	

Table 3. Properties of materials used for the calculations

Strength of beams

Failure occurred at a load of 292 kN for beam 1 and 287 kN for beam 2, giving very similar results, although there is a difference in the width of the 2 beams. The narrower of the two was made up for by the presence of 3 additional rows of connectors, which stiffened the connection between the wood and the concrete, leading to a reduction in bending stress. It can be seen (Figure 7) that behaviour is linear up to a load of 150 kN. The non-linearity of the behaviour which ensued is indicative of gradually damage which stems from the non-linearity of slipping of the interface with the load. Failure occurred when the concrete (reinforced only by the expanded metal) was sheared longitudinally along the connectors, causing tensile failure and detachment of the carbon at mid-span (Figure 9). An ultra high performance fibre-reinforced concrete would no doubt have delayed failure, due to the higher shear strength. Toutlemonde [Toutlemonde et al. 2007) evaluates the shear strength of commercial ultra high performance concretes at 8 to 10 MPa, based on punching tests.

This breaking load of nearly 300 kN can be compared to the theoretical breaking load of a reinforced concrete beam with the same geometry, which gives an idea of the performance of the structure tested. Thus, by considering that a 400 mm wide 200 mm high reinforced concrete beam, whose reinforcements correspond to 1% of the cross-section of the concrete, breaks as a result of plastic deformation of the reinforcements during an identical test, a breaking load of about 160 kN is obtained, that is, nearly half that of the composite beams tested.

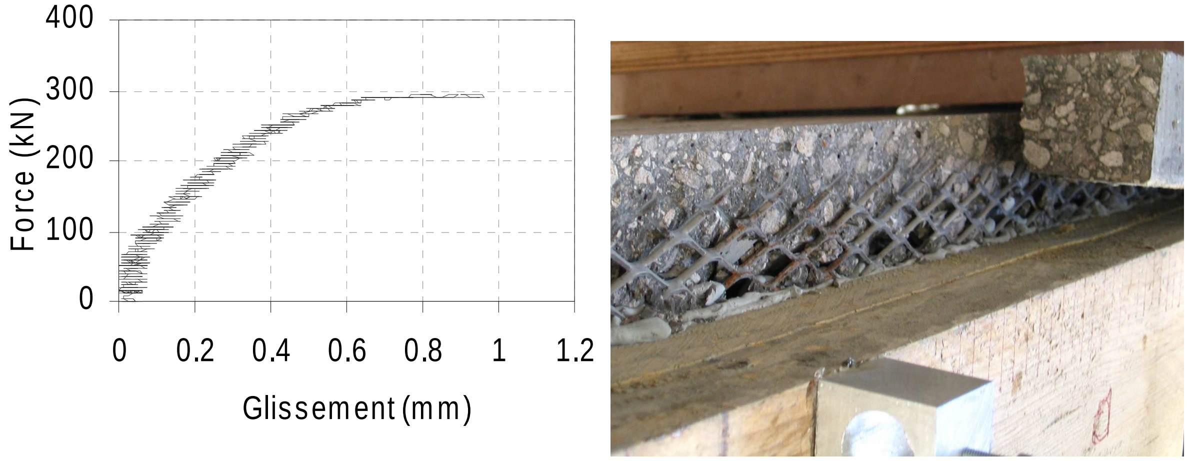


Figure 9. Load versus slipping at the interface on end and shear failure in the concrete.

42 Field of mechanical interest of carbon-fibre reinforcement

It may seem difficult to justify the use of carbon fibres to reduce the span to depth ratio of bridge decks. An order of magnitude separates the longitudinal module of the carbon from that of wood. A thickness of 5 mm of carbon is therefore equivalent in terms of stiffness to about 50 mm of wood, i.e. 1 to 2 additional pieces of wood, which does not significantly change the span to depth ratio.

However, in relation to service life, a comparison can be made between two structures with similar inertia, the first a wood-UHPFRC composite beam and the second a UHPFRC-wood-carbon composite beam. The reinforcement leads to a reduction in the tensile stress in the wood (Figure 10), which is more or less proportional to the relative gain in height. This reduction in the tensile stress can be useful in terms of fatigue.

With respect to ultimate limit states, it is understood that the carbon fibres will be useful if the failure mechanism begins in the wood during bending, which supposes that the interface between the wood and the concrete is highly resistant and that the shear strength of the concrete is high. In this case, failure will begin in the wood. The difference in strength between the reinforced beam and a non-reinforced beam with the same inertia would then be due to the difference in tensile stress in the wood (Figure 10), but not the shear stress. The reinforcement, which leads to a reduction in tensile stress in the wood of 5 to 7 MPa in the present case, will then result in an increase in the breaking load. To this quantifiable effect is added that of the probable increase in the strength of the wood through the presence of carbon. In other words, the failure in the reinforced wood could occur at much higher yield strengths than those of wood alone. This aspect is obviously to be confirmed by tests.

In conclusion, for the use of a carbon-fibre reinforcement to be mechanically advantageous, a wood-concrete connection system is need with very high strength and stiffness. If that is not the case, it will probably be more economical to try to improve the connection before using composite materials.

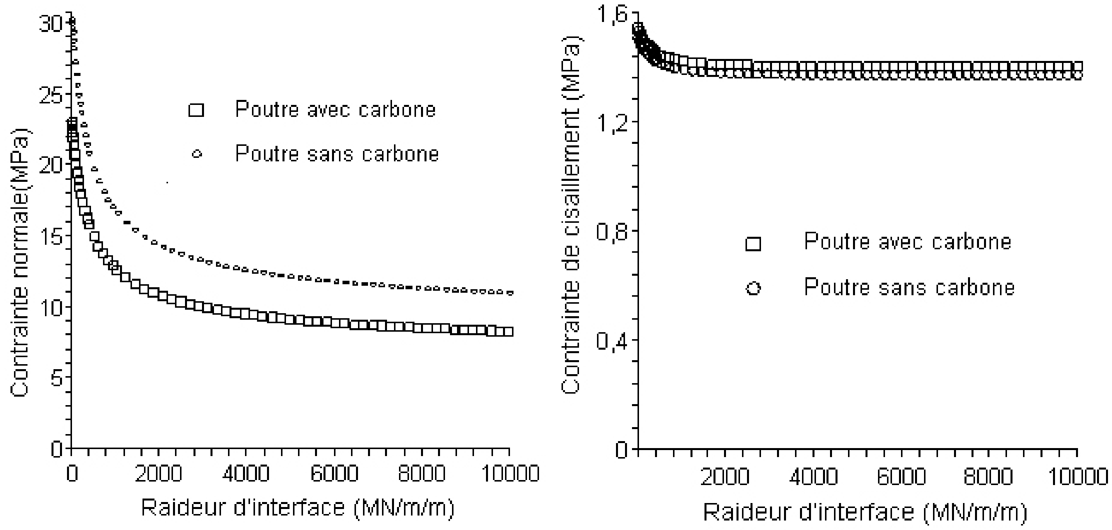


Figure 10. Development of normal stresses (on the left) and maximum shear (on the right) in the wood as a function of the stiffness of the wood-concrete interface. Big square points are relative to reinforced beam with carbon, small diamond shaped points are relative to non reinforced beam.

5 CONCLUSIONS

A set of HPC-wood connection systems for composite decks has been presented in this article. Composite beams have been designed and produced and their performance in terms of ultimate static load and bending stiffness have been measured. Four different failure mechanisms have been highlighted. These mechanisms are generally initiated inside the connector or the wood. The system of direct gluing of the concrete to the wood distinguishes itself from the others by a high level of stiffness, Very close to the perfect connection, and by an ultimate load which is also greater than the others by at least 20%. The gluing technique was then validated by an ultra high performance fibre-reinforced concrete and the type of surface treatment for the concrete to be glued was validated.

The equivalent stiffness of the different test bodies was estimated using an analytical modelling taking slipping at the interface into account. This model enables the connection system to be classified by means of a stiffness coefficient. It is thus possible to classify the systems tested into three different groups.

The article presents the results of two bending tests on a concrete-wood-carbon beam. The stiffness of the component's interface was found to be proportional to the number of rows of connectors at the interface. Calculation of the tensile and shear stresses shows that reinforcement of the wood using carbon-fibre strips has a positive effect on the ultimate limit states in the case of a wood-concrete interface which has a good level of stiffness and strength.

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