Collapse and Rehabilitation of Composite Trussed Structures

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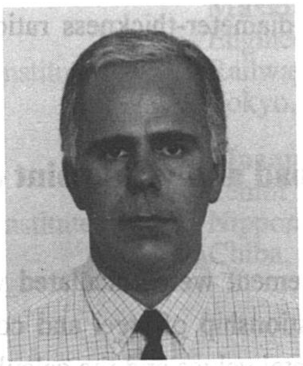

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Summary

A failure case of a 2.400 m^2 area of an industrial roofing structure is presented. The investigation leading to the cause of the disaster, the numerical and experimental study using reduced models the rehabilitation procedures for the remaining part of the structure are described. Conclusions are also drawn concerning the application of external tension reinforcements to restore the original load capacity of the structure.

1. Introduction

Composite steel-concrete trussed members are widely used in Brazil as supporting structures for roofing large industrial facility areas, and the complete lack of maintenance in such structures is for certain the main cause of serious problems of deterioration and corrosion.
The case in study concerns one of such structures, used to shelter an industrial area enclosing an environment with strong emissions of chemically active dust and vapours.
The whole building structure of 12000 m^2 is divided into four sections, constructed from 1960 up to 1987, and containing different structural models for the trusses.
Trusses of type I consist of a two span indeterminate structure of 51 m long. Trusses of type II are simply supported, 30 m long structures, covering the collapsed area. Both are constructed in reinforced concrete but have unprotected steel bars as the tensioned diagonals. The type II truss, however, has no diagonals in the central frame.
Trusses of type III and IV, more recently constructed, were in good conditions.

2. Investigation of the Collapsed Area

The collapsed structure was composed by seven type II trusses. Many nodes were ruptured and showed they were not properly reinforced as plane frame nodes.
Corrosion in the exposed bars was more evident near the lower nodes, where the bars were immersed in a thick layer of chemically active dust, accumulated over the years. One of the ruptured tendons had its cross sectional area reduced by almost seventy percent.
Four other trusses of the same type, however, were still standing. A weak connection was observed to link those two areas. Rupture at this point certainly occurred before any sufficient overload could be transferred to the neighbour trusses, and the collapsing wave was stopped.
The inspection was extended to the rest of the building and showed that trusses type I, covering the main building area, were also very deteriorated. It was further observed that some of the steel bar diagonals were completely loose, subjected to no tension.

3. Numerical Analysis

Considering the pre-cracking stiffness of the nodes, a plane frame finite element model was used for the analysis of the trussed structures.
A first analysis was made for trusses type I and II, considering the original design properties. It confirmed that two of the steel diagonals in trusses of type I had no tension whatsoever.
Further investigative analyses showed that if the continuous trusses type I were considered as two separate, simply supported spans, all member would have been well dimensioned and those diagonals properly tensioned.
All evidences pointed to the conclusion that the trusses were originally intended to be separated and, for some reason, constructed as continuous over the internal support.
A second analysis was performed for trusses type II considering the measured properties of the materials and the reduced sections of the steel diagonal bars. The result showed that the tensile force in the ruptured and deeply corroded diagonal was beyond its reduced area capacity. The remaining members were not overstressed.
Rupture was expected to have begun in that diagonal and caused failure of the whole structure.
A third analysis was made withdrawing that particular bar from the structure. The moments at the nodes of that frame raised up to extremely high values, far beyond its capacity That would certainly create a mechanism and destroy the structural symmetry, leading to a unavoidable failure mechanism of the whole structure.

4. Design of the Reinforcement

The continuous trusses type I and the remainig and trusses type II, not affected by the collapse, were completely rehabilitated and strengthened for restoring their original load capacity.
An external reinforcement was devised using epoxy glued steel plates around the nodes, also fixed with bolts, in which steel bars were welded. The reinforcement was placed parallel to the tensioned diagonals and as new crossed diagonals at the central frame of trusses type II.
Reinforcement was also positioned at the bottom cord of trusses type II and the top central cord of the continuous trusses type I, to help supporting its constructively created continuity.

5. Experimental Analysis

Reduced models have been very didactic for the learning process of engineering students.
The case in study would perfectly fit its purpose and two reduced models made of microconcrete and galvanised wire were built. One to reproduce failure of trusses type II and another, exactly alike, to test the effectiveness of the reinforcement.
Both 400 by 38 cm models were positioned in a testing frame and instrummented for the measuring of nodal displacements and strains at the bars, nodes and reinforcement.
The first model was loaded, at each nodal point, and behaved very linearly. To reproduce the corrosion rupture of the steel diagonal a thin saw was used to slowly reduce its cross section.
After cutting over half section, the bar suddenly ruptured and the moments at the nodes of the same frame increased abruptly. The very same, numerically predicted, failure mechanism was formed and the structure suffer an immediate collapse.
The second model was subjected to service load, as the first one, and behaved alike. Maintaining the load, the reinforcement was placed, as it would be done in the prototype structure.
The external loading was eventually increased by fifty percent. The model responded stiffer and tension was properly transferred to the reinforcement, as expected.
Like in the first model the diagonal bar was also cut, and now its force was adequately transferred to the reinforcement. The external load was again increased by fifty percent and the reinforced model was able to sustain it very adequately.

6. Conclusions

The strengthening procedure was considered adequate and built in the real structures. The most convenient feature of this technique consist on its economy and simplicity. The structure may be strengthened while supporting the service load and there is a minimal disruption on the production process in operation.

Use of Aluminium Alloys in Retrofitting Ancient Suspension Bridges

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Summary

The retrofitting of suspension bridges can take profit of the specific aluminium alloy characteristics for obtaining the maximum structural effectiveness, particularly if compared to classical solutions based on the use of steel. This paper also emphasizes how different structural materials can optimally co-operate in the suspension bridge scheme, thus characterizing the rehabilitated structure as a particular composite construction. The above aspects are discussed with reference to some case studies.

1. Introduction

During the seventies a rehabilitation program of ancient suspension bridges, constructed between the end of the 19th century and the beginning of the 20th century, has been developed in France. The old structures were made of wooden deck, masonry piers, steel girders and steel suspension chains. The adoption of aluminium alloy floor girders in the retrofit project of three bridges (the Montmerle and the Trevoux bridges on the Saone river; the Groslée bridge on the Rone river), allowed for conserving as much as possible some of the old structural elements, and brought to very effective solutions, both from the cost and the structural performance points of view.
In the Montmerle bridge (two 80 m bays), the use of aluminium both for the two truss beams with bolted connections, and for the deck slab, led to the possibility of increasing the weight of the road vehicles, while preserving both the existing cables and piers without significant strengthening. In the retrofit of the Groslée bridge (a single 174 m long bay) the floor structure is made of three longitudinal aluminium truss girders, connected to a light reinforced concrete slab. In this scheme a remarkable example of co-operation among different structural materials, each of them utilised in an optimum working condition, can be observed: the old masonry piers, the harmonic steel suspension cables, the stainless steel suspension ties, the aluminium alloy reticular girders with high strength steel bolted joints and the light reinforced concrete slab floor.

2. Pre-requisities of aluminium structures

The aluminium alloys can be considered as a family of materials which exhibit a wide range of mechanical properties depending on the type of alloy and the technological treatments [1]. Therefore it is possible to identify alloys which have strength comparable to the common structural steel, and, at the same time, a weight which is one third of the steel one. Due to this specific combined characteristics a high structural effectiveness can be obtained through the use of aluminium alloys. In addition, in the special case of retrofitting suspension bridges, the lightness of the material adopted for the girders allows for reducing the stress both in the steel cables and in the old masonry piers, thus allowing for the maintenance of these existing structural elements. As a further consequence of the reduced structural weight, i.e. of the dead loads, an increase of the live loads, i.e. of the maximum weight of vehicles which the bridge can sustain, can be obtained. Finally the corrosion resistance of the aluminium leads to avoid any surface protections, even in the case of bridges crossing rivers, thus reducing both the initial and the maintenance costs.

3. Advantages of aluminium versus steel

For the Montmerle and the Groslée bridges, extensive structural analyses of the actual retrofit aluminium solution and of an alternative steel solution, both subjected to several load conditions, have been carried out in [2], in order to examine the structural behaviour in the two cases and to point out the reasons which suggested the choice of the aluminium solutions instead of the steel ones. The comparison between the results obtained for the steel and aluminium solutions in both cases substantially showed that: bending stresses in the girders are less in the aluminium solution than in the steel one, axial stresses in the suspension cables are approximately equal in the two solutions and deformations in the floor girders are lightly larger in the aluminium solution, as expected due to the lower Young modulus of the material, but the values of the absolute displacement are still in an allowable range.
In addition to the numerical analyses of these particular case studies, a wide parametric analysis has been carried out in [3] on a simplified structural model of suspension bridge, accounting for second order effects. The main geometrical and mechanical parameters have been varied in a wide range, in order to point out their influence on the distribution of stresses and deformations among the different structural components, and therefore to show under which conditions the maximum benefits can be obtained through the use of aluminium structural elements. It has been observed in [3] that the detrimental effect of the larger deformability of aluminium is significantly reduced by accounting for second order effects in the structural analysis model.

4. A new proposal

A structural retrofit project of the oldest Italian suspension bridge, the "Real Ferdinando" bridge on the Garigliano river, based on the use of aluminium alloy girders, has been recently proposed in the context of a wider rehabilitation program of the zone [4]. The original bridge had a single 85 m bay scheme and the suspension system consisted of two pairs of steel chains, connected to steel ties which sustain the two steel longitudinal truss girders and the wooden transverse beams and floor slab. The proposed project, designed in order to satisfy the requirements of: (a) historic preservation, (b) stiffening of the floor structures, and (c) adoption of innovative technologies and materials, is based on the use of aluminium alloy girders, allowing for the conservation of the original geometrical configuration and appearance. The comparison between the results of structural analyses showed the following advantages of the aluminium with respect to an alternative steel solution: significant reduction of the dead loads; reduction of the required section area of the suspension cables; elimination of the costs necessary for corrosion protection treatments; reduction of repair and strengthening measures required for the masonry piers; easier and less expensive transporting and erection operations due to the lightness of the structural elements.

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Shear Strengthening of Existing Reinforced Concrete Slabs : an Experimental Investigation

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Summary

The strengthening of existing reinforced concrete slabs with vertical posttensioned steel bolts is investigated by testing six slabs. Shear strengthening increases the failure load and leads to a more ductile behavior. By prestressing or injecting the bolts, the peak-load is only slightly increased, but the slip of the bolt inside the slab is avoided and therefore, the serviceability is improved. The prediction of the peak load for slabs with both injected and non-injected bolts is discussed.

1. Description of the experiments

The punching tests (Menétrey and Brühwiler [1]) are performed on octagonal slabs of 1.2 m in diameter and 120 mm thick, supported at its extremity by RHS steel pieces (arranged around a diameter of 1.1 m) and loaded at the center through a circular steel column (diameter 120 mm) with an hydraulic jack by controlling the vertical displacement. The concrete compressive strength on cylinder after 21 days is f_c=33.4 MPa. All slabs are reinforced with horizontal orthogonal bars (steel quality S500) at the bottom and at the top. The percentage of the bottom reinforcement is ρ=0.94%. Slab 1 is not strengthened and slabs 2 to 6 are perforated and strengthened with eight high strength steel bolts placed around a radius of 140 mm equiped with force measuring device. The strengthening system is composed of a bolt type M 10 with an ultimate tensile strength: f_u= 851 MPa, yield strength at 0.2% strain: f_y = 736 MPa. The bolts of slabs 3, 5 and 6 are post-tensioned with the nut on top of the slab. The injection around the bolts are set for slabs 4, 5 and 6 with an epoxy-resin.

2. Tests results

The test results are presented with the load-displacement curve (Fig. 1a) and the mean force in the bolts versus the vertical displacement of the slab (Fig. 1b). It follows that the punching load is increased from 280 kN up to 380 kN or 37% due to the strengthening. The vertical displacement at maximum load is significantly increased by the strengthening as it has more than doubled. It is observed that slab 1 is characterized by a punching cone inclined at an angle of approximately 30^0. For all strengthened slabs, the punching cone is formed between the column diameter and the perimeter defined by the bolts. The inclination of the punching cone is approximately 70^0 (Fig. 2). This means that all the strengthened slabs exhibit a punching failure which is characterized by a punching crack that did not cross the strengthening bolts.

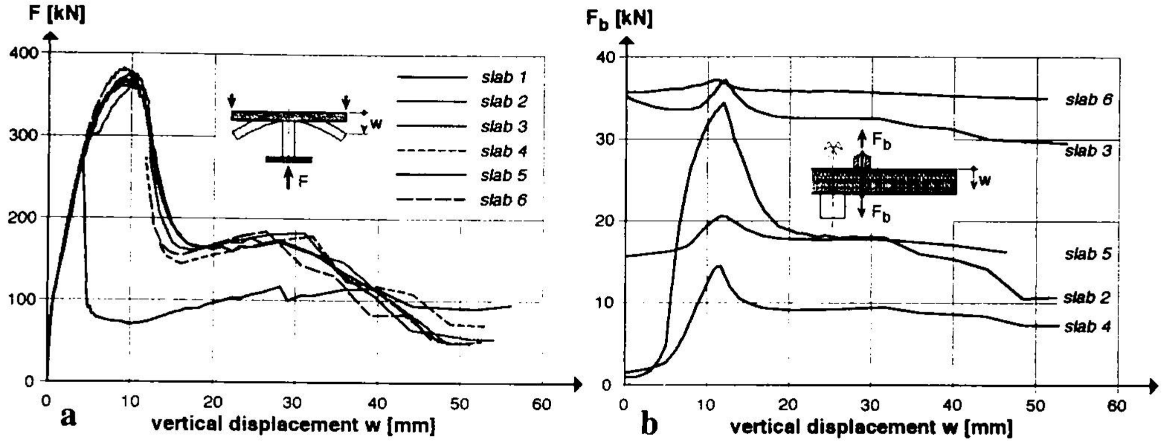

Fig. 1: Load-displacement curves and mean force in the bolts versus vertical displacement

The post-peak descending branch of the load-displacement curve is characterized by a strong reduction of the load carrying capacity with increasing displacement characterizing punching failure (Fig. 1a). After peak, the first drop of the load carrying capacity is about 200 kN for all the slabs (strengthened or not) which indicates that this decrease is due to a similar failure mechanism, that is, the concrete failure.
The present experimental results are similar to the well known characteristic of bolted joints in steel construction for which the failure load for both with and without prestressing force is the same. In addition, prestressing the bolts improves the serviceability of structures.
Injection modifies the slab mode of resistance so that it resists globally resulting in reduction of the stress level in the bolts (compare slab 2 and 4 in Fig. 1b). The injection also improves the serviceability of the slab and provides a protection against corrosion of the bolts.

3. Prediction of the punching load

The punching load for the slab without strengthening bolts is predicted with the analytical model developed by Menétrey [2] which leads to V_{pun}=250 kN. The punching load of the strengthened slabs with a punching crack inclination of 70^0 is influenced by the punching and the flexural strength (as proposed by Menétrey [3]) resulting in V_{fail}=364 kN.

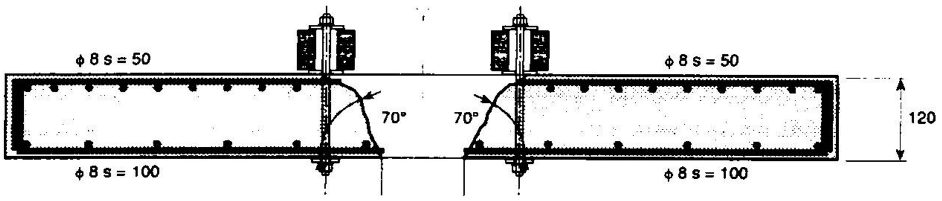

Fig. 2 : Sketch of the punching crack in the strengthened slab

The punching load of slabs strengthened with non-injected bolts is obtained by adding the dowel and the bolt strength. The punching load of slabs strengthened with injected bolts is recovered by adding the concrete, the dowel and the bolt strength.

References:

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Major Repairs to Frank Lloyd Wright's Largest House: Wingspread, Racine, WI, USA

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Summary

Frank Lloyd Wright's design for Wingspread was a brilliant conception but contained several inherent structural deficiencies. These did not manifest themselves significantly for the first 56 years of the house's life, but in 1994 large displacements occurred due to heavy snows. Repairs were made using a carbon fiber-epoxy thin shell laminated directly to the timber roof to form a composite structure. This shell was built up *in situ* from 13 layers of quad-axial carbon fiber.

Project Description

Wingspread, built in 1938 in Racine, Wisconsin, USA, the largest house ever designed by Frank Lloyd Wright (over 4,000 m^2 floor area). The roofs and walls are framed almost entirely of small dimension lumber (50 x 100 mm to 50 x 250 mm nominal), spaced closely (generally 40 cm on center). At the center of the house is the octagonal Great Hall (15 x 18 m) supported in the center by a very large brick chimney and on four of the sides by seven brick piers on each. The sloping wood roof of the Great Hall is interrupted by three concentric rings of glass skylights each stepped slightly lower than the one above. Radiating out from the Great Hall in orthogonal directions are wings containing bedrooms, kitchen, garage, etc. -- thus the name: Wingspread.

Over the years small cracks had appeared in the plaster and wood ceiling of the Great Hall as well as the East Wing. The heavy snows during the winter of 1993-94 however caused a major displacement near the skylight rings in the Great Hall with cracks larger than 40 mm opening. In addition the exterior walls of the East Wing were noticed to be considerably out of plumb due to thrust at the top of the wood stud wall caused by load from the gable roof rafters. Clearly immediate action was necessary to stabilize and repair this structure, now used as a conference center for the Johnson Foundation.

Temporary shoring was installed and the roof tiles were removed (they were not original in any case, having been replaced in 1993). The permanent repair for the great Hall sought to achieve several objectives. First it should never be visible in the completed work. Second it should minimize disturbances to the existing interior finishes. And third it should be as economical as possible while still conforming to the rules of historic preservation.

After extensive computer modeling using Finite Element Analysis, it was decided to create a shell structure out of the lower roof by coating the bare wood sheathing with 13 layers (12 mm total thickness) of quad-axial carbon fiber fabric set in a 50%-50% matrix of epoxy resin which was bonded to the wood. Prior to the first layer of carbon fiber installation the wood was thoroughly cleaned and large diameter screws were installed through the sheathing into the rafters at 15 cm on centers in oversized holes flooded with epoxy. Thus the carbon fiber was bonded to the wood sheathing which was in turn acting compositely with the rafters. Each layer of carbon was offset approximately 7-8 cm from the layer below to provide a scarf joint at every lap of fabric. Vacuum bagging was not practical for this on site installation so air bubbles were forced out using toothed rollers. The entire operation was conducted inside an environmentally controlled temporary timber framed structure built over the top of the roof. Upon completion, the composite membrane was post cured at 60° C for 24 hours.

Extensive testing was conducted prior to installation, particularly to determine the modulus of elasticity of the composite membrane. Stiffness, not strength was the chief characteristic sought after in the design. Test specimens showed a compressive modulus of E = 34 mPa.

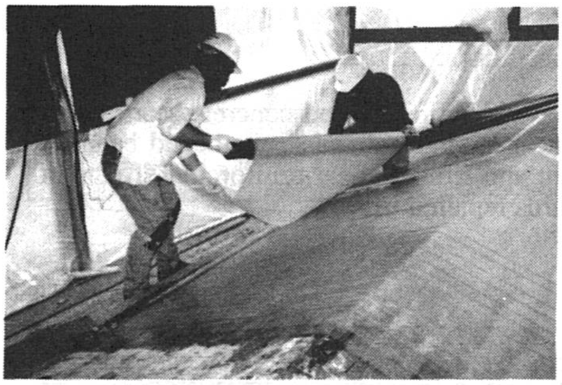

Fig. 1 Installation of Carbon Fiber Fabric

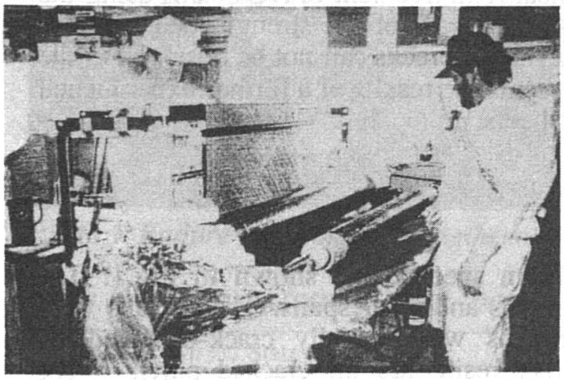

Fig. 2 Impregnating Fabric with Epoxy

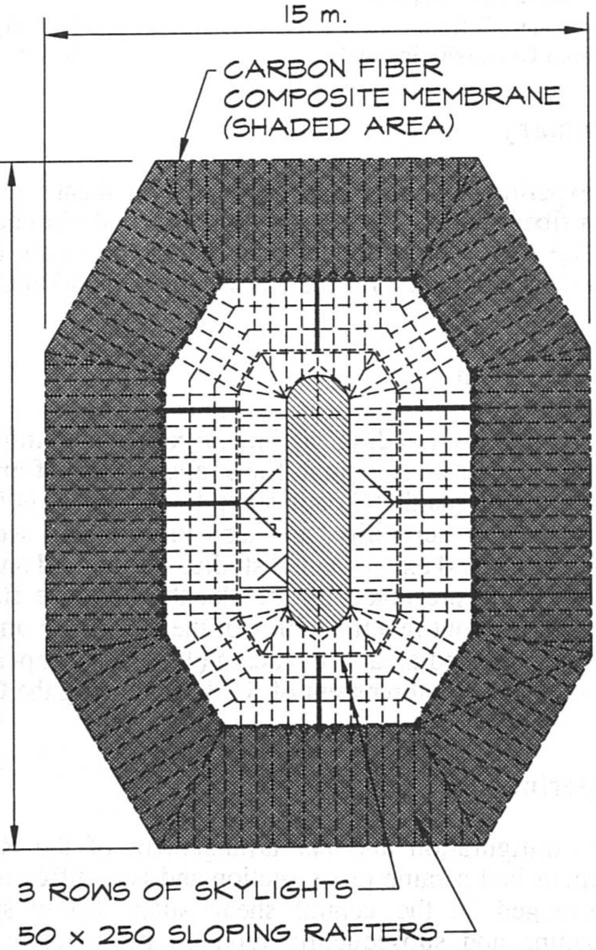

Fig. 3 Roof Framing Plan