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Session E4

Attachment of New Materials to Existing Structures
Utilisation de nouveaux matériaux dans les structures existantes
Anwendung neuer Baustoffe in bestehenden Bauwerken

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Connection of Old Concrete with New Concrete-Overlays

Liaison entre ancien et nouveau béton

Verbinden von altem Beton mit neuem Beton

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SUMMARY

If a new layer of concrete is applied to existing concrete with the aim of strengthening or repairing a structure, the result is a composite structure. The bond interface can crack even if the design and the work is carried out carefully. The forces must then be carried with the aid of reinforcement passing across the interface. This paper describes a simple computational model and corresponding tests with a new type of mechanical connector.

RÉSUMÉ

Si l'on ajoute unenouvelle couche de béton sur une construction en béton afin de renforcer celle-ci, il en résulte une construction mixte. Même si l'exécution est très soignée, il arrive que le joint soit fissuré. La transmission des forces à travers le joint doit se faire à l'aide d'une armature. Cet article présente un modèle de calcul simple ainsi que les essais correspondants avec des connecteurs mécaniques.

ZUSAMMENFASSUNG

Wird eine neue Betonschicht auf eine bestehende Betonkonstruktion aufgebracht, in der Absicht, das Tragwerk zu verstärken oder instandzusetzen, so entsteht ein Verbundtragwerk. Auch bei sorgfältiger Arbeit kann es geschehen, dass die Verbundfuge reisst. Die auftretenden Kräfte müssen dann mit Hilfe einer Bewehrung durch die Fuge geleitet werden. Die vorliegende Arbeit berichtet über ein einfaches Rechenmodell und entsprechende Versuche mit mechanischen Verbindungsmitteln.

1. INTRODUCTION

The demand for the strengthening of concrete structures is rapidly increasing. This is in part due to changes in the use of, or increases in the service loads for structures, but also results from the growing need for repair of damage and wear. In many cases, a loadbearing layer of new concrete (overlay) is applied over the existing concrete structure. This overlay is usually placed as cast- or pneumatically-placed concrete. It can work to augment either the flexural compression or tension zones, depending on reinforcement and placement (see Fig. 1).

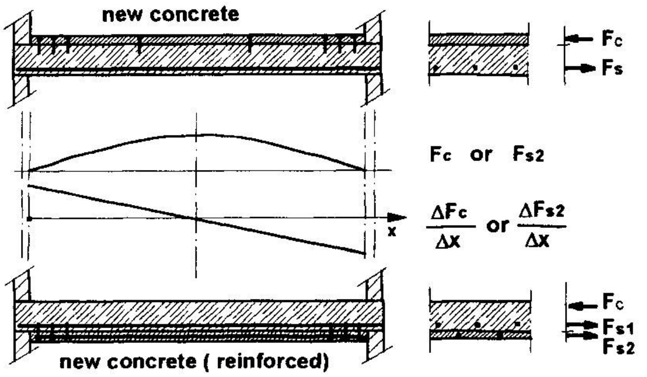

Fig. 1 Examples for the method of strengthening

2. STATUS OF COMPUTATION METHODS

Initially, the stresses in the bond interface result from a combination of external loads and internal restraint forces. Fig. 2 shows the development of internal stresses resulting from shrinkage and temperature gradients in the new concrete. Note that these stresses are typically maximum at the slab edges. The combination of external and internal

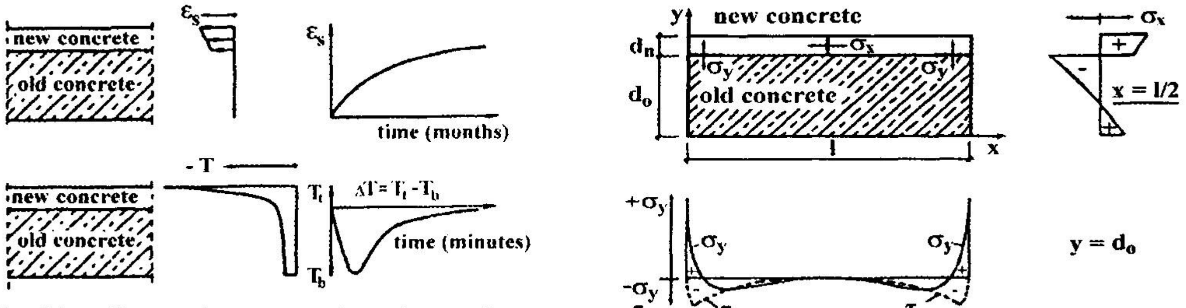

Fig. 2 Loading and stresses of new layer of concrete

stresses often exceed the capacity of the initial bond, thus requiring the designer to consider a cracked interface. This is particularly true in the case of bridge overlays which are subjected to fatigue stresses resulting from traffic loads. Furthermore, these stresses are dependent on time and bond failure can take place years after overlay placement.

If the bond interface is cracked, the stresses may be considered to result from external loads only, since the cracked interface releases the internal stresses. This assumption requires reinforcement to cross the bond interface in order for shear transfer to occur, a necessary condition for the monolithic design of the section.

Improvement in the performance of the bond between new and old concrete can be achieved by (a) careful preparation of the old surface (b) formulating a special, low-shrinkage concrete or a concrete made from low-heat cement or (c) through suitable methods of placement, such as pneumatically-placed concrete. Furthermore, chemical primers and bonding slurries can be used. It must then be borne in mind that even if the bond is made properly, i.e. the strength of the bond is the same as that of the base material, the underlying concrete may fail anyway. In many cases, failure of the bond can also be attributed to sub-standard work or unrealistic requirements for work execution.

In jobsite practice, it has been found that the stringent requirements for both planning and execution are not met in many cases. As a result, design standards usually require very low shear stresses unless transverse reinforcement is provided to facilitate transfer of the shear force through the bond layer. Such mechanical transfer is also required by

AASHTO [1] in bridge decks. The amount of transverse reinforcement required is characterized as the ratio of the cross-sectional area of reinforcement crossing the interface to the area of concrete being placed. AASHTO specifies a minimum reinforcement ratio of 0.08%, but studies [2] have shown that this amount of reinforcement is not adequate to maintain monolithic behaviour after the bond is broken. Instead, a reinforcement ratio of 0.28% is suggested. The cost of placing these connectors can be as much as 15 to 20% of the total rehabilitation cost. Another study [3] indicates that a reinforcement ratio of more than 0.1% would be uneconomical and that much smaller ratios could be used if the surface of the interface is scarified prior to placing the overlay.

Since these influencing factors are very difficult to control and quantify, the obvious procedure is to use the assumption of cracked concrete associated with reinforced concrete design and to assign the acting tensile forces to transverse reinforcement crossing the bond interface. If existing reinforcement is insufficient, additional reinforcement or mechanical connectors may be required. The working principle of shear transfer in a cracked bond interface can be compared with the problem of the transfer of shear forces across a crack in normally reinforced concrete subjected to bending and shear. This problem has been the subject of many comprehensive studies [4, 5, 6]. In a crack, however, the surface roughness differs from the roughness of an existing concrete surface resulting from various methods of surface preparation.

3. SETTING UP THE DESIGN MODEL

The model for the design is set up on the basis of a trussed framework analogy commonly used for many cases in cracked reinforced concrete. A model of this kind serves the engineer by simplifying the visualisation of various relevant influencing factors. This model assumes relatively thin connectors with diameters ≤ 20 mm. A tensile force is set up in the connector when a crack opens at the bond interface. The shear resistance of the connector is relatively small and only becomes effective if the surface of the bond interface is smooth. These relationships have been presented very clearly in the work done by Tsoukantas and Tassios [7].

Key for Fig. 3:

- V Shear force in bond
- F Tensile force in connector (dowel)
- C Compressive force in concrete
- α Angle of line of action of compression
- w Width of crack in bond
- s Slip of bond
- r Interlocking of crack surface (classified as roughness)

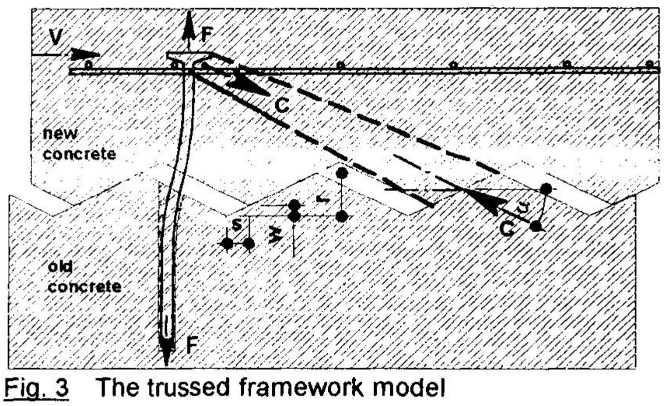

Fig. 3 The trussed framework model

From Fig. 3 the following dependencies can be derived for a composite system stressed by shear force V:

* The magnitude of tensile force F in the connector is dependent on the shear force V and the angle α of the line of action of compression C.

* The effect of the **roughness** is expressed in the model by the **angle** α of the line of action of compression C. Tests show that the roughness of the bond interface is of decisive importance for the bond shear force which can be transferred. The achieveable roughness depends on jobsite conditions and the tools, equipment, etc. available to the workmen. Since making a surface very rough can result in high construction costs, an

optimum balance between the roughness requirements and the number of connectors must be achieved. The effect of surface roughness has an upper limit which is given by fracture of the new or the old concrete adjacent to the interface zone.

* The **stiffness under tension S_t** of the connector depends on its cross-sectional area and length as well as the effectivness of its anchorage in the two layers of concrete. The stiffness is decisive for the tensile force that results from a certain **crack width w.** The crack width, in turn, is limited by the surface roughness. If serviceability requirements limit the crack width, stiffer connectors must be employed.

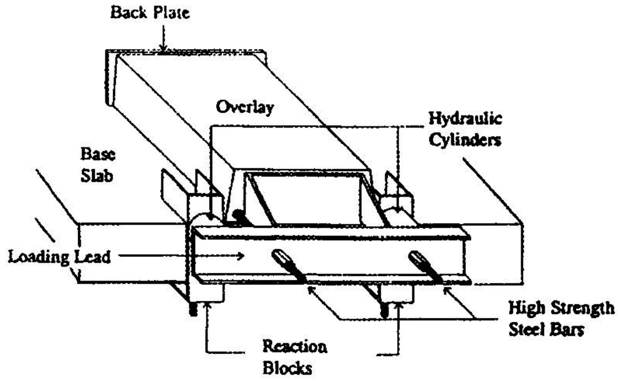

Fig. 4 Shear test (Choi [9])

For laboratory tests, the following aspects are of note: It is best for the trussed framework model to be segregated into tension and compression lines of action. The large number of influencing factors can then be sorted out and studied through simple tests.

First, simple pull-out tests of the connectors are carried out to determine the stiffness and ultimate loading capacity of the connectors. The parameters to be investigated are the concrete compressive strength f_c, the mode of anchorage in the old concrete, and the anchorage in the new concrete as determined by the anchor geometry.

The transmission of shear is then studied during pure shear tests. For these tests, the surface roughness, the normal stress in the bond and, again, the concrete compressive strength, are varied. When carrying out the shear tests, particular care must be taken to ensure that no secondary trussed frameworks are induced by the transfer of loads and that the two layers of concrete can move away from each other with parallel crack edges. Figures 4 and 5 show typical arrangements for shear tests. The overall performance is then studied by conducting building component tests.

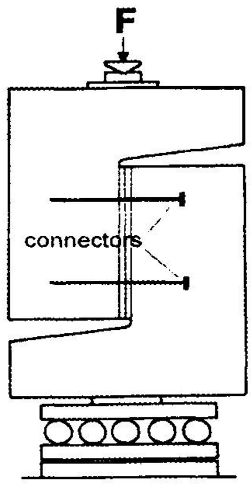

Fig. 5 Shear test (Walraven [4])

4. INVESTIGATION OF APPLICABILITY OF EXPERIMENTAL HILTI JUMBO-NAILS TO FAST-TRACK BONDED CONCRETE OVERLAYS

An investigation has been performed at the Fergusson Structural Engineering Laboratory (FSEL) at the University of Texas at Austin. The objective was to study the use of the experimental Hilti Jumbo-Nail to permit construction of fast-track bonded concrete overlays when environmental conditions endanger the development of bond between the substrate and the overlay, or as a means to limit or eliminate substrate surface preparation. The concept of the Hilti Jumbo-Nail is derived from powder-actuated fastener technology. It provides for rapid installation of dowel-like fasteners without chemical adhesives, thus reducting labor and material costs and permitting immediate placement of new concrete.

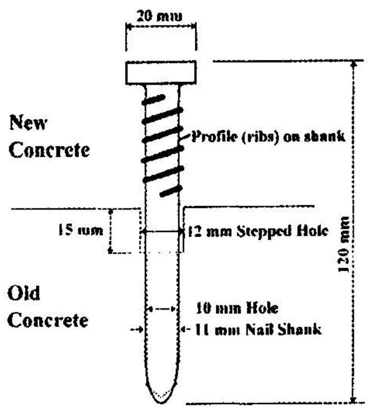

Fig. 6 Hilti Jumbo-Nail

A number of tests have been conducted to study the effectiveness of the Hilti Jumbo-Nail for controlling overlay cracking and preventing or arresting delamination [8] [9].

The test program for the pull-out tests was designed to simulate field conditions as closely as possible. Variables such as the concrete strength, the location of the nail with respect to the edge of the slab, cracks in the concrete, or other conditions that would normally be encountered during the application of the nails were studied along with as well as subtle variations from the recommended installation procedure. 336 pull-out tests were performed, and the results (Fig. 7) are documented by Colecchia [8].

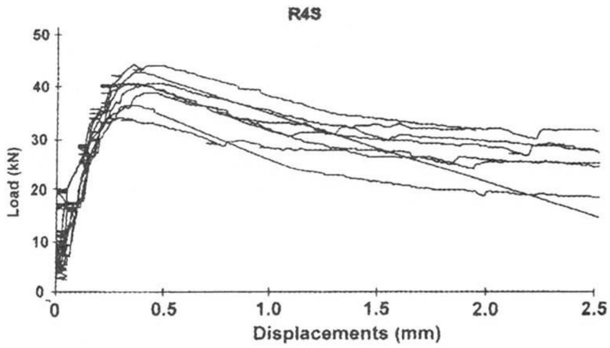

Fig. 7 Example for pull-out tests

The nail pull-out test results show that an edge distance of 215 mm is necessary for a Jumbo Nail driven into 30 MPa concrete to develop full pull-out strength. Accordingly, a Hilti Jumbo Nail shear test specimen would require overall dimensions considerably larger than those of a standard shear test. It was therefore decided to devise an „in-situ push-off test". Replicates of four concrete slabs used for the pull-out tests were used as the base slabs. The overlays were cast on top of the base slabs and cured. Up to sixteen overlays were cast on each base slab (Fig. 8). Advantages of the in-situ push-off test method are that many test specimens can be made using one set of forms and the shear test can be performed inplace as soon the overlay gains the desired strength. 116 overlays were cast and tested over a period of 12 months. The results are documented by Choi [9].

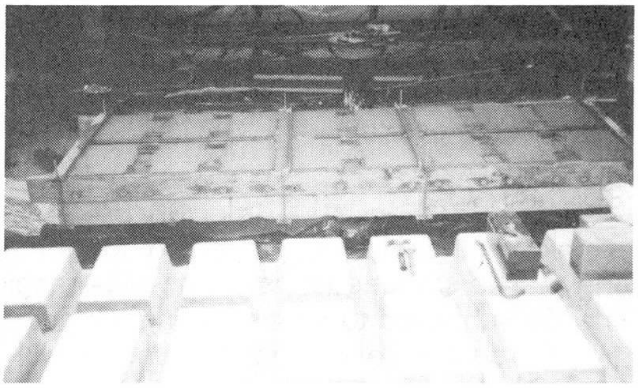

Fig. 8 In-situ shear tests of Hilti Jumbo Nails

It should be noted that all overlays were cast against the dry top surface of the base slab in this study. Since the moisture condition on a base slab can only be decided qualitatively before placement of an overlay, it was decided to keep all interfaces dry to eliminate the effect of moisture from the test results. The top surface of the base slab was roughened and the overlay cast on the roughened surface for most test specimens. For some test specimens, the overlay was cast on the relatively smooth troweled surface of the base slab. The reinforcement ratio was controlled through the use of a template and the placement of single or paired nails. The minimum reinforcement ratio was 0.13% and the maximum was 0.38%.

The test variables included: (a) the compressive strength and aggregate type of the base concrete, (b) the roughness of the interface and (c) the presence of cracks in the base slab. A number of flexural cracks was created on one face of the base slabs. In some tests, the adhesion between the base concrete and the overlay was intentionally broken using a bond breaker and some tests were performed early to analyse early-age interface shear strength development. The test results of two unbonded specimens and two bonded specimens are plotted for comparison in Fig. 9. All four test specimens had the same contact area and used paired nails (ρ = 0.38%). The shear load versus horizontal displacement plots of unbonded test specimens did not show the sharp peaks previously encountered with bonded test specimens.

Fig. 9 shows that the unbonded specimens using paired nails were effective in limiting the horizontal displacement to small values when the applied shear loads are relatively low. The magnitude of shear loads in the relatively flat plateaus in bonded specimens and unbonded specimens are similar at larger dispacements. This is important for relief of shrinkage stresses in the layer of new concrete.

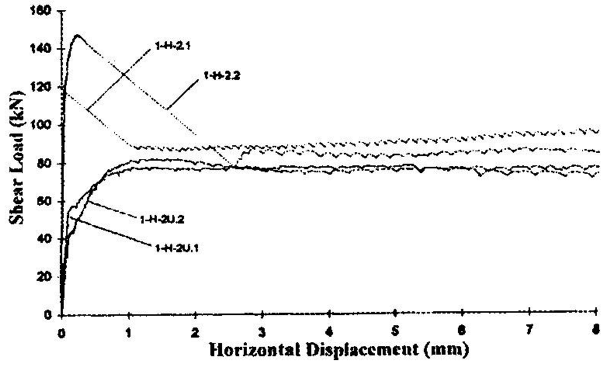

Fig. 9 Example for shear tests

5. DESIGN CONSIDERATIONS

For concrete overlays, the model shown in Fig. 3 provides a suitable design basis. The following applies fundamentally: the design resistance of the connectors and the compressive tie must be at least equal to the design value of the tensile and compressive forces calculated from the actions. The partial safety factors for resistance must be stipulated according to the mode of failure. The tensile force in the vertical component of the strut is carried by the resistance of the connector. The number of connectors per unit of area is calculated from the area over which the connectors are effective. Installing connectors over a large area can be avoided if the shear force set up by external actions is low. In such cases, however, connectors are still required in the edge zones owing to the unavoidable forces of constraint. The force to be resisted by the connectors is then calculated from the tensile force which corresponds to concrete cracking in the overlay.

6. SUMMARY

A simple methodolgy is presented for the design of shear connectors in concrete overlays. The strut model used has wide-ranging application for concrete-concrete bond interfaces subjected to shear loading. Ongoing research in this field is being conducted by Hilti Corporate Research in conjunction with its partner Universities.

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Concrete Repair under Dynamic Loads with Polymer Modified Mortars

Réparation du béton, sous charge dynamique, à l'aide de ciments et de résine époxyde

Betonreparatur unter dynamischen Lasten mit Polymer-modifiziertem Mörtel

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SUMMARY

Since the first generation of repair products using epoxy and cement technology, extensive research has been done to improve such systems in order to make them meet the highest performance requirements. These new generation of epoxy-cement products is ideally suited for extensive rehabilitation work on bridges heavily damaged by frost and deicing salts. Vibration caused by dynamic loads may have an influence on the bond and on the crack-free hardening of the repair material. Extensive testing in laboratories on samples applied under static and dynamic loads allow to optimize repair compounds so that their performance is unimpaired by dynamic loads during application.

RÉSUMÉ

Des travaux de recherche extensifs ont permis d'éliminer les désavantages des mortiers de réparation à base de ciment et de résine de la première génération. Les systèmes de la seconde génération sont parfaitement adaptés aux travaux de restauration de haute qualité, tels que ponts gravement détériorés par le gel et les sels de déverglaçage. Les vibrations, provoquées par le trafic pendant les travaux, peuvent avoir des conséquences néfastes sur l'adhérence et le durcissement des matériaux de réparation. Des essais extensifs en laboratoire sur le comportement des mortiers sous des charges statiques et dynamiques, permettent d'optimaliser les formulations des produits de réparation de façon à pouvoir garantir que leur performance ne subit pas l'influence des effets dynamiques provoqués par le trafic pendant les travaux.

ZUSAMMENFASSUNG

ECC-Mörtel vereinigen die positiven chemischen und mechanischen Eigenschaften von Epoxidharzen mit den guten physikalischen Eigenschaften von Zement. Mehrjährige intensive Forschungsarbeit war notwendig, um die baupraktischen Nachteile epoxidharzvergüteter Zementmörtel der ersten Generation zu eliminieren und die hohen Anforderungen, welche an ein grossflächiges Brückensanierungssystem gestellt werden, zu erfüllen. Durch die Nutzung von Brücken während der Instandsetzung entstehen Schwingungen, welche den Haftverbund und das rissfreie Aushärten eines Reparaturmörtels nachteilig beeinflussen können. Umfangreiche Versuche von statisch und dynamisch belasteten Verbundkörpern ermöglichen die Optimierung von Sanierungssystemen, welche eine einwandfreie Betoninstandsetzung unter Betrieb garantieren müssen.

1. RESTORATION OF REINFORCED CONCRETE BRIDGES

An enormous amount of construction work has been done in short time during the boom period of the Sixties and Seventies in Europe. Fundamental rules of construction practice, such as protecting the bridges against penetration of aggressive detrimental substances, have been disregarded. The still little known effects of de-icing salts, of increasing acidity of rainwater and of mounting aggressivity of air pollution, have been underestimated. The concrete of the bridge decks under the wear course, the shoulders and piers in the splash zone, above all show considerable corrosion damages. The repair mortars used for the restoration of reinforced concrete bridges have evolved rapidly during the last 20 years.[1]

2. REQUIREMENTS FOR CONCRETE RESTORATION WORK

The aim of concrete repair is not only to restore the original condition but also to give durable protection against further deterioration. The aggravating damages call for always better quality materials and have led to the following demanding qualification criteria for repair mortars:

- **Structural bond between concrete substrate and repair mortar.**
  - ⇒ high bond strength
  - ⇒ good water retention
- **No spallation caused by strain differentials from different thermal dilatations, shrinkage or swelling.**
  - ⇒ same thermal dilatation characteristics as the substrate
  - ⇒ low chemical and physical shrinkage
- **No overstress caused by differing material-characteristics of concrete and repair mortar.**
  - ⇒ matching E-moduli (also at low temperatures)
  - ⇒ adequate compressive strength
- **Effective protection against detrimental environmental attack(liquid/gaseous).**
  - ⇒ high degree of impermeability (CO_2 / H_2O)
  - ⇒ low w/c ratio
  - ⇒ resistance against freeze-thaw cycles and de-icing salt
  - ⇒ crack-free hardening
- **Suitable for site conditions, i.e. easy and efficient application.**
  - ⇒ long pot life and opentime
  - ⇒ non-sag
  - ⇒ suitable for machine application
- **Satisfactory esthetics of the structure after restoration.**
  - ⇒ good base for subsequent coatings
  - ⇒ suitable to match fair faced concrete

3. EPOCEM MORTARS,THE NEW PRODUCT TECHNOLOGY

The majority of the listed requirements could not be met by traditional cement mortars (above all low shrinkage, high bond strength, impermeability). Modified cement mortars and resin based mortars are therefore widely used for concrete restoration work. The products available in the market can be classified according to their binder system as follows: (Fig.1)

Binder-system	Cement		Resin	Epoxy-Cement
Designation	CC = Cement-Concrete	PCC = Polymer-Cement-Concrete	PC = Polymer-Concrete	ECC = Epoxy-Cement-Concrete
Reaction mechanism	Ordinary cement mortars	Cement mortars modified with non-reactive polymerised thermoplasts	Mortars with reactive two component resin-hardener binders	Cement mortars with a second reactive epoxy binder. Duobinder system with hydration of cement and polyaddition of the epoxy resin.
Examples		PVA, PVP, SBR, AC	EP, PMA PUR	
Positive characteristics	Similar to concrete	Bond (wet substrate) ↑ Impermeability ↑ E-Modulus ↓ Shrinkage ↓	Bond (dry substrate) ↑ Strength ↑ Chemical resistance ↑ Curing ↓	ECC = sum of the positive characteristics of cement and resin binders.

Fig.1 Classification of repair mortars according to their binder system

Water absorption, when permanently exposed to water, is the only disadvantage of PCC worthwhile mentioning. Their numerous advantages predominate by far and this type of mortar is now widely used for concrete restoration work.
PC certainly does have positive properties. However, reservations have to be made about bond to moist substrates and elasticity properties depending on temperature, which leave this type for special applications only.

ECC combines the positive chemical and mechanical properties of epoxies with the good physical properties of cement.

The hardening process of ECC, unlike PCC, comprises two separate chemical reactions

Hydratation of cement with water	+	Polyaddition of the epoxy resin/hardener component

To attain the combined effect of the two fundamentally different binder systems, it is extremely important that the epoxy resin forms inside the binder matrix a lamellar framework, into and through which the cement crystals can grow. (Fig.2) In case of cement crystal failure under load, the stresses transfer to the epoxy resin structure. The formation of such framework depends on the quantity, the type and above all on the dispersion of the epoxy resin in the mortar.

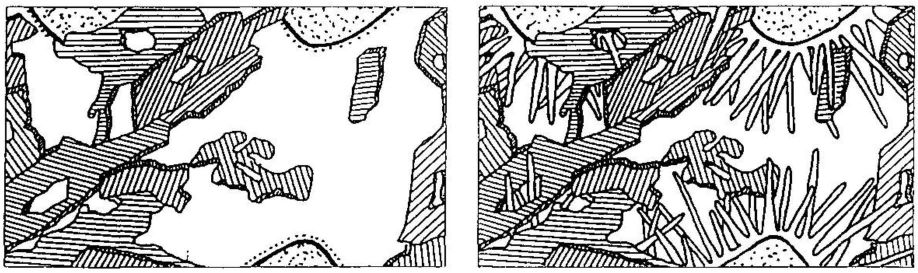

Fig.:2 Reaction mechanism in an EpoCem mortar. [2]

First generation ECC (since 1982) had considerable disadvantages such as excessively long mixing time, inhomogenious dispersion of the epoxy in the mortar, entrapped water triggering off secondary reactions causing shrinkage and unwanted stresses in the binder matrix. All this created problems on site.
Since 1987, the second generation EpoCem mortars allow easy and safe application and react without any problems.[2] These products are based on finely tuned binder systems. Selected additives, ideal grading of the aggregates for the powder component, specially developed, in water emulsifiable, high reactive polyamine hardeners and a super finely dispersed binder emulsion allow problem-free application on site.

Practical experience in the last 5 years has shown the strong points of this new product technology for restoration of reinforced concrete bridges as follows:

- **Superior bond to mineral substrates ,even when permanently moist.**
- **Low-shrinkage hardening and accelerated strength development allow easy finishing.**
- **Time saving because surfaces can be coated after 1 to 3 days.**
- **High compressive and flexural strength with E-modulus and thermal expansion similar to cement mortar.**
- **Impervious to water but pervious to water vapour.**
- **High resistance against freeze-thaw cycles and de-icing salts.**
- **Economical thanks to easy and quick application.**

Numerous successful applications are there to prove that the difficulties, met with the ECC of the first generation, have been overcome. EpoCem mortar has excellent self-levelling properties and is therefore widely used for re-bar corrosion protection or as a thin-layer overlay for bridge deck restoration work. EpoCem can be thyxotropised for vertical and overhead application.

4. RESTORATION UNDER DYNAMIC LOAD

New materials and their optimal use have led, in the recent time, to slenderer structures and wider spans which in consequence are more sensitive to deformation and vibrations. Bridge restoration work very often has to be executed under full or at least partial traffic load in order to limit the obstruction of traffic and avoid excessive costs for a temporary bridge. Repair mortars for work on the underside of a bridge have therefore to meet additional requirements.

Fig. 3a) Inertial force **F** Fig. 3b) Alternate elongation

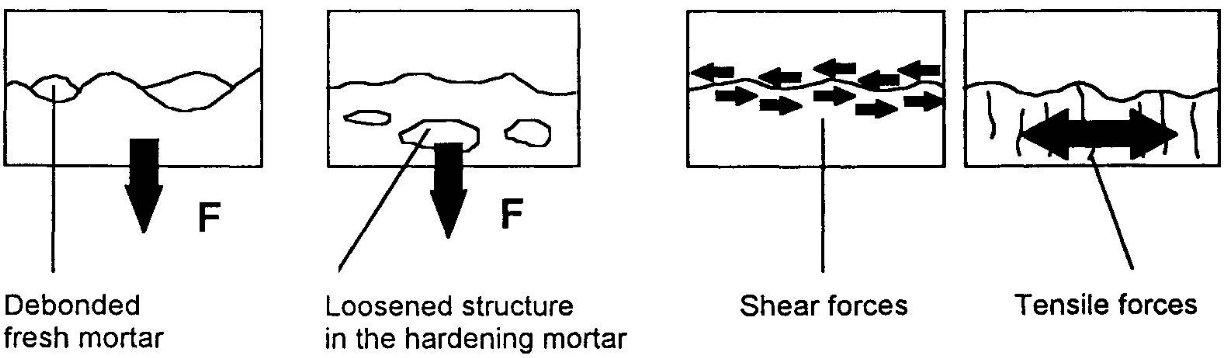

Fig.3: Effect of a dynamic load on the repair mortar. [3]

- **<u>Inertial forces,</u> resulting from vibrations, act on overhead applied repair mortar besides its proper weight, and can cause the mortar to detach from the substrate or loosen up the structure of the mortar in process of hardening.** (Fig.3a)
- **The substrates alternate deformations cause <u>shear forces,</u> which impair the bond between mortar and old concrete, as well as <u>tensile forces</u> in the mortar, which can cause cracking.** (Fig.3b)

The frequency of the oscillations is affected by the characteristic frequency of the bridge as well as by the frequency of the oscillation imposed by traffic. Records or Standard Specifications of such oscillation characteristics are practically non-existent. The test methods applied by two different test laboratories are described in chapter 5 and 6.

Basically there are two ways to apply non-sag mortar overhead:

A - <u>Hand application</u> of ready for use repair mortars for smaller patches.

B - <u>Machine application,</u> wet spray or dry spray, of ready for use mortar mixes for restoration of large areas. Both spraying methods are commonly used and the decision which one to chose is rather influenced by application criteria than by the pros and cons of different materials technologies. The main advantages are listed below:

<u>Dry spray method</u>	<u>Wet spray method</u>
• High output	• Consistent w/c ratio
• No bonding bridge necessary	• Low dust nuisance
	• Low rebound
• Thicker layers	• Easy finishing

5. DYNAMIC TESTING OF MACHINE APPLIED DRY MIXES.

The Structural Engineering Laboratory (IBAC) in Aachen, Germany uses a standardized method for the testing of repair mortars under dynamic load. The test is based on the ZTV-SIB 90 plus Annex TP BE-SPCC.[4] The loading pattern is specified as follows:
A beam (0.2 x 0.75 x 2.45 m) oscillates for 24 hours at a frequency of f=10Hz and with an oscillation velocity of v=8.5 mm/s in a way that a maximum tensile stress of $\sigma_{Z,R}$ = 1.5 N/mm^2 results in the contact zone between concrete and mortar.
The loading is controlled in a way to produce consistent elongation amplitudes in the contact zone. The testing is very comprehensive and includes different basic tests as well as quality control tests of the materials used for the repair system (i.e. compressive strength, tensile bending strength, shrinkage, E-modulus etc.). Various Standards specify the ultimate requirements depending on the function of the mortar in its intended use and on its method of application. Polymer modified mortars (SPCC) spray applied, have to meet the following specification:

- Pull off strength after oscillation: $\bar{\beta}_{HZ} \geq 2.0$ N/mm^2
  $\beta_{HZ\ min.} \geq 1.5$ N/mm^2
- Maximum crack width: ≤ 0.1 mm

Specific formulations of polymer and silica fume modified mortars for sprayed application, meeting these exacting specifications for repair under dynamic load, are available from the market since several years already. Remains open the question in how far the load pattern chosen for these tests are representative of the various shapes and characteristics of our bridges. It has nevertheless to be mentioned that all restoration work executed with mortars having passed above test, has been proven successful so far.

6. DYNAMIC TESTING OF HAND APPLIED READY FOR USE MORTARS

The Swiss Federal Research and Materials Testing Laboratory EMPA in Dübendorf, in 1994 have tested 10 different standard PCC and ECC products (some of them with fibres) available from the market on an oscillating beam.[3] Based on results gained in the field from 202 bridges, the frequency and velocity of the oscillation have been fixed at f=5Hz and v=20mm/s. The mortar on the oscillating beam has been tested for 24 hours. Reference samples have previously been applied to the static beam. High strength prestressed concrete beams (0.2 x 0.75 x 5.65m) have been used to prevent cracking of the contact surface mortar/concrete at the maximum tensile stress of $\sigma_{Z,R}$ = 1.5 N/mm^2.

The following variables have been measured:

- **Compressive and tensile bending strength of separately prepared specimens.**
- **Bond strength on 96 carrots Ø50 mm per product.**
- **Type of failure classified according to its position (in the concrete, in the mortar, in the bond line).**
- **Visual check for cracks.**

All products have been applied by their suppliers. The test results were the following:

- The statement "High tensile bending strength = High bond strength" has been found to be valid practically without exception.
- Only 2 out of 10 products reached as well on the static as on the oscillating beam a bond strength > 3 N/mm^2. Both these products showed no drop in bond strength under dynamic load.
- 5 out of 10 mortars lost up to 50% of their bond strength under dynamic load. 3 products had to be eliminated due to widely scattered results (probably due to irregular application).

<u>Conclusions:</u>

Only 2 out of 7 tested mortars could be retained for recommendation after the tests under dynamic load although all mortars had performed similarly good on separate static test specimens. Both retained mortars had suffered no loss of quality on the oscillating beams. No cracking could be detected in the mortar. Both products are of the PCC type applied on a PCC respectively a ECC bonding bridge.

In order to gain assurance that application on site can be done economically, 4 selected products have been spray applied by machine and have been tested in the same way in a second test series. First results of these tests will be available at the time of the Symposium. Within the next two years, it is planned to test in a third step the best repair mortars under different relevant load conditions.

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Concrete Composite Construction for Durability and Strength

Construction en béton composite pour améliorer la durabilité et la résistance

Betonverbundbauweise für Dauerhaftigkeit und Festigkeit

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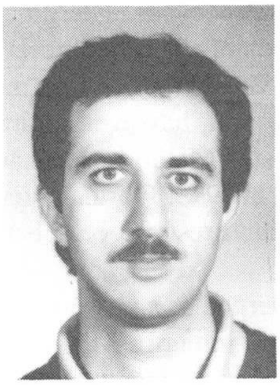

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SUMMARY

Strength, ductility and durability of concrete columns and walls could be significantly enhanced in composite construction with Fiber Reinforced Plastics (FRP). An FRP-concrete column is proposed in which the filament-wound tubular shell is the pour form, protective and confining jacket, and bi-directional reinforcement. Behavior of composite columns is studied analytically and experimentally. Results indicate higher compressive and flexural strength, as well as pseudo-ductile characteristics.

RÉSUMÉ

Il est actuellement possible d'augmenter la résistance, la dureté et la durabilité des piliers et parois en béton, en faisant appel à un mode de construction mixte à base de matières plastiques renforcées par fibres. L'auteur propose un type de pilier en béton composite, dans lequel une enveloppe de fibres tissées sert à la fois de coffrage perdu, de couche protectrice, de frettage et d'armature bidirectionnelle. L'article présente une étude analytique et expérimentale du comportement de ce type de pilier mixte. Les résultats obtenus mettent en évidence des résistances élevées à la compression et à la flexion ainsi que des propriétés de pseudo-ductilité.

ZUSAMMENFASSUNG

Die Festigkeit, Zähigkeit und Dauerhaftigkeit von Betonwänden und -stützen könnten durch eine Verbundbauweise mit faserverstärkten Kunststoffen deutlich verbessert werden. Es wird eine Verbundbetonstütze vorgeschlagen, bei der eine fasergesponnene Hülle als verlorene Schalung, Schutzschicht, Umschnürung und kreuzweise Bewehrung dient. Das Verhalten solcher Verbundstützen wird analytisch und experimentell untersucht. Die Ergebnisse deuten auf eine höhere Druck- und Biegefestigkeit sowie pseudoduktile Eigenschaften.

1. INTRODUCTION

The nation's infrastructure is plagued with two major problems; premature deterioration and structural deficiency. The average remaining life of highway bridges in the U.S. is estimated to be between 9 and 34 years depending on the bridge type [1]. Even in newer bridges, premature decay caused by service conditions has been a growing problem. On the other hand, survey of damaged structures in recent earthquakes indicates that in several cases catastrophic failure of an entire structure was triggered by the failure of columns in a chain action. Hence, it is vital to the national economy that new technologies be developed to extend service life, and to improve performance and strength of highway bridges.

Concrete members exposed to corrosive environments undergo an accelerated decay when chloride ions penetrate concrete cover and cause corrosion of embedded steel. The most important factor in long-term durability of concrete is its permeability. Current protection measures are designed to delay corrosive agents from reaching steel re-bars. They do not, however, entirely solve the corrosion problem, nor do they address the permeability of concrete. Methods such as epoxy coating have failed in severe environments such as the Florida Keys [2]. In 1991, Florida initiated expensive plans for galvanizing the corroded epoxy-coated re-bars for substructures of several bridges along its coastlines [3].

Fiber Reinforced Plastics (FRP) have emerged as a potential solution to the problems associated with the infrastructure. The most effective application of FRPs is in the form of protective jacket as well as load-carrying partner in composite construction with concrete. This provides for optimal use of materials based on mechanical properties and resistance to corrosive agents. Moreover, it results in structural members with pseudo-ductile characteristics. One such application has been demonstrated in fiber jacketing technique [4], which is now considered an effective retrofitting tool for existing columns.

2. FRP-CONCRETE COLUMN

Using the principles of fiber or steel jacketing [4,5], concrete-filled steel columns [6], and pressure vessel technology [7], a novel composite column is proposed that consists of a concrete core confined in an FRP shell (Fig. 1). The tubular shell, while an integral part of the system, is also the pour form for concrete. It may be a multi-layer FRP tube consisting of at least two plies; an inner ply of axial fibers and an outer ply of circumferential fibers (Fig. 1a). This shell type is manufactured by a continuous normal-axial winding process that generates both axial and hoop reinforcement. Axial fibers are inhibited from outward buckling by the outer

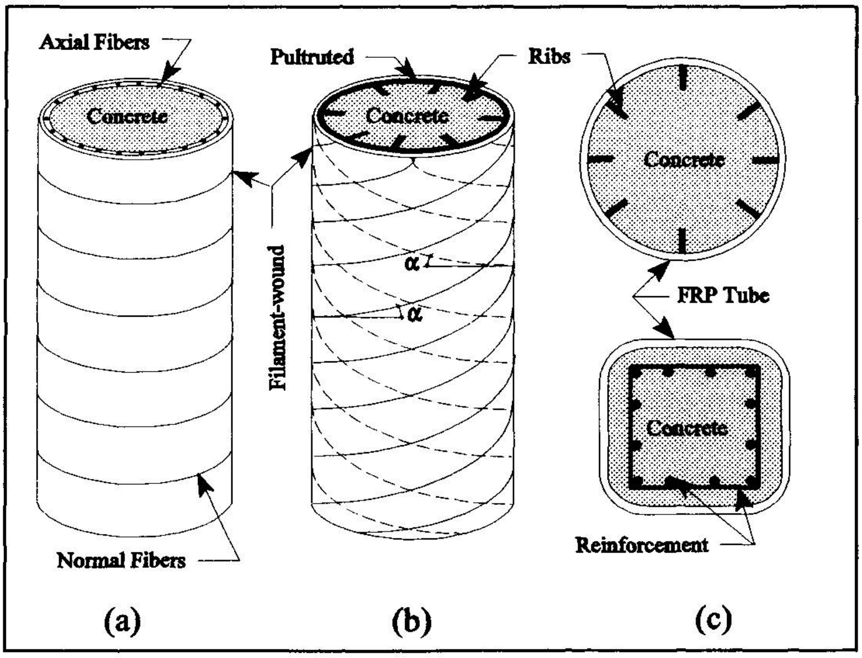

Fig. 1 Composite FRP-concrete column; (a) Normal-axial winding, (b) pultruted shape with stiffening ribs and angle-ply cover, and (c) typical sections with and without reinforcement

normal fibers, and from inward buckling by the concrete core. The author has also proposed a unique cross section that consists of a pultruted tube on which an angle-ply laminate ($\pm\alpha$) is wound. As shown in Fig. 1b, the pultruted tube may have stiffening ribs that act as axial reinforcement for concrete. In either of the proposed configurations, the jacket provides bi-directional reinforcement for concrete core; i.e., hoop and axial reinforcement. The hoop reinforcement confines concrete and prevents buckling of longitudinal fibers and ribs or bars (if any). It also increases the bond strength of reinforcing bars (if any). The longitudinal fibers improve the axial-flexural capacity of the column similar to a concrete-filled steel tube [6]. The jacket further enhances column's shear strength even more effectively than spiral reinforcement [5]. Bi-directional fiber arrangement makes it possible to remove the entire steel reinforcing cage from the column (Fig. 1c). This will significantly reduce construction cost and time, and will further improve durability of the structure in saline environments.

3. DURABILITY

Sealing and covering of a concrete column by non-corrosive FRP material increases its service life tremendously. This will protect concrete from moisture intrusion that could otherwise corrode the steel re-bars (if in existence) and potentially deteriorate the concrete itself. Sheet membrane systems have been used in the past to protect concrete bridge decks. Currently, in an effort to design and construct the first cable-stayed composite bridge in the U.S., researchers at the University of California, San Diego are investigating the use of concrete pylons covered by carbon fiber jackets. Therefore, it is appropriate to envision encasing concrete columns in FRP tubular jackets to protect concrete and the embedded steel. On the other hand, plastics have been used as main structural members as well as protective jackets. In a hot oil pipeline project that ran over the water in the Gulf of Mexico, a glass fiber jacket was chosen over aluminum and stainless steel alternatives [8]. After 11 years, reports indicate that the jacket has withstood a harsh, salty environment with over 140° F internal temperature, direct sunlight, moisture, vapor, and seawater. Another application of plastics was recently introduced as a composite plastic-steel pile in the form of a steel pipe encased in recycled plastic [9]. Army Corps of Engineers has used durable glass-flake isophthalic resin as protective coating for steel pier piling against corrosive effects of saltwater [10]. Other applications include storage tanks, composite seawalls, and liners for concrete chimneys.

4. STRUCTURAL BEHAVIOR

Confinement depends on two factors; tendency of concrete to dilate, and radial stiffness of the jacket to restrain the dilation. This will place concrete in radial compression, and the confining member in circumferential tension (Fig. 2). The proposed system creates a passive confinement, since the confining pressure is developed only after hoop elongation is imposed on the shell by expansion of concrete (Poisson's effect). On the other hand, in fiber-wrapping method [4], high-strength synthetic fibers are wrapped around the column while being tensioned, thereby producing an active confinement. While active confinement methods could potentially be used only after concrete is hardened, passive methods of confinement such as the proposed technique are suitable for new construction, since the jacket becomes the pour form for concrete. Regardless of the method, the ultimate degree of confinement is a function of the strain energy stored in the confining member. Confinement ratio (C_r) is defined as the ratio of radial pressure f_r to the 28-day compressive strength of unconfined concrete (f'_{co}). The radial pressure is in turn balanced by the hoop

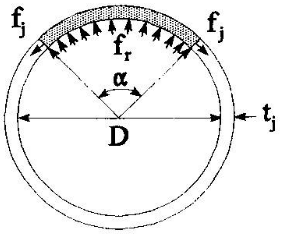

Fig. 2 Confining Action

tensile stress in the jacket (f_j) such that

$$f_r = \frac{2 f_j t_j}{D} \quad (1)$$

where, D=inside diameter of the jacket, and t_j=jacket thickness. Of the various confinement models for concrete, Mander, et al [11] offer a simple method to predict the compressive strength of confined concrete based on the confinement ratio:

$$C_a = [\, 2.254 \sqrt{1 + 7.94\, C_r} - 2\, C_r - 1.254 \,] \quad (2)$$

where $C_a = f'_{cc}/f'_{co}$ (compressive strength of confined and unconfined concrete), and C_r=confinement ratio (f_r/f'_{co}). For example, for a 1270 mm.-diameter circular column with 27.6 MPa concrete, a mere 3.18 mm. fiberglass tube with 1165 MPa hoop strength will result in a confinement ratio of 20%, which in turn doubles the compressive strength of concrete. It should be noted that the confinement developed by internal hoops or spirals only applies to an effective concrete area within the center core. This area is less than the normal core area bounded by the centerline of the perimeter tie. In fact, in the axial direction, the effective confined area is at the mid-point between the lateral ties. Hence, the confinement effectiveness is a function of core area as well as shape and spacing of transverse reinforcement. It then seems logical to conclude that externally confined columns such as the one proposed here could offer the most effective form of confinement. In this method the concrete area outside longitudinal reinforcement becomes structurally confined and its spalling will be contained. Despite its simple form, since Mander model does not take into account the constitutive model of the confining agent, it only yields upper bound values for fiber composite jackets. The author has developed a new confinement model that is applicable to both steel and fiber jackets, i.e., whether or not the jacket demonstrates a plastic behavior [12].

The proposed model is used to predict the ultimate moment and curvature at failure of beam-columns for a range from pure axial compression to pure flexure. The main assumptions in this analysis are the linear strain distribution through full depth of the cross section, and the strain compatibility of steel-concrete and concrete-FRP. Also, the confinement contribution of the interior hoops or spirals are neglected. Fig.3 shows the interaction diagrams for a typical bridge column. The curves are normalized in both directions with respect to the maximum axial and flexural capacities of the unconfined concrete. Two different confinement ratios of 0.5 and 1.0 are examined. One can easily relate the confinement ratio to the jacket parameters such as thickness and strength of the FRP shell. For example, for the same confinement ratio a thicker fiberglass shell is required as compared to a carbon fiber jacket. Two series of interaction diagrams are developed with and without contribution of the longitudinal fibers of the jacket. When contribution of axial fibers is neglected, confining effect of the jacket is more pronounced in pure compression rather than in pure flexure. For example, for confinement ratios of 0.5 and 1.0, the maximum compressive force in the section is increased by 126% and 182%, respectively, while maximum moment in pure flexure is only increased by 7.9% and 10.2%, respectively. On the other hand, when contribution of axial fibers is taken into account, the maximum compressive force in the section is increased by 161% and 278%, respectively, and the maximum moment in pure flexure is increased by 205% and 454%, respectively. The effect of actual bond strength on the interaction diagrams is reported elsewhere [13].

5. EXPERIMENTAL STUDY

A series of small-scale specimens were tested. The specimens were made of 152.5 mm. x 305 mm. concrete core with three different jacket thickness; 1.6 mm., 3.8 mm., and 6.4 mm. The jacket was made

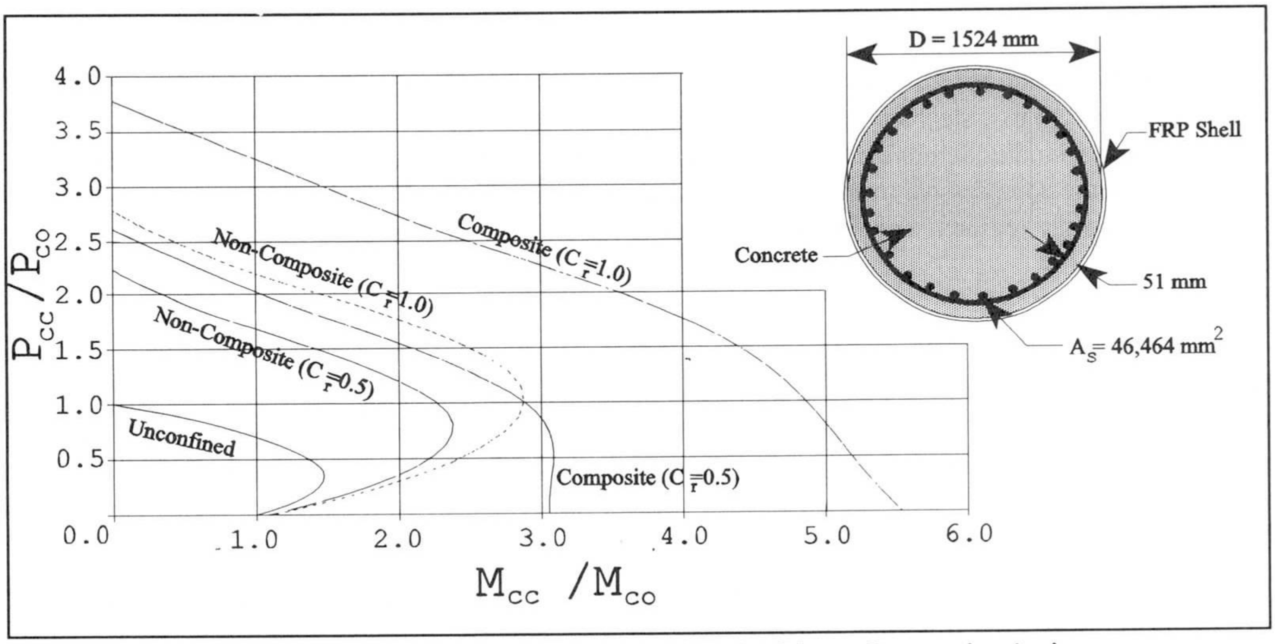

Fig.3 Column Interaction Diagrams with and without Composite Action

of an angle-ply of polyester resin and uni-directional glass fibers with ± 15° winding angle. Therefore, the jacket was primarily functional in the circumferential direction. Fig.4 shows a control specimen next to a composite specimen with 1.6 mm. jacket thickness, both after failure. The control specimen in this case failed at 378 kN (i.e., f'_{co}=20.7 MPa). The composite specimen, however, failed at 907 kN (i.e., f'_{co}=49.8 MPa), showing a 140% increase in axial strength. As shown in the figure, the control specimen failed by spalling off parts of the concrete cylinder. On the other hand, the composite specimen showed a remarkable ductility and did not fail until the first hoop fracture initiated at some point in the top of the specimen. For brevity, detailed experimental results are not reported here.

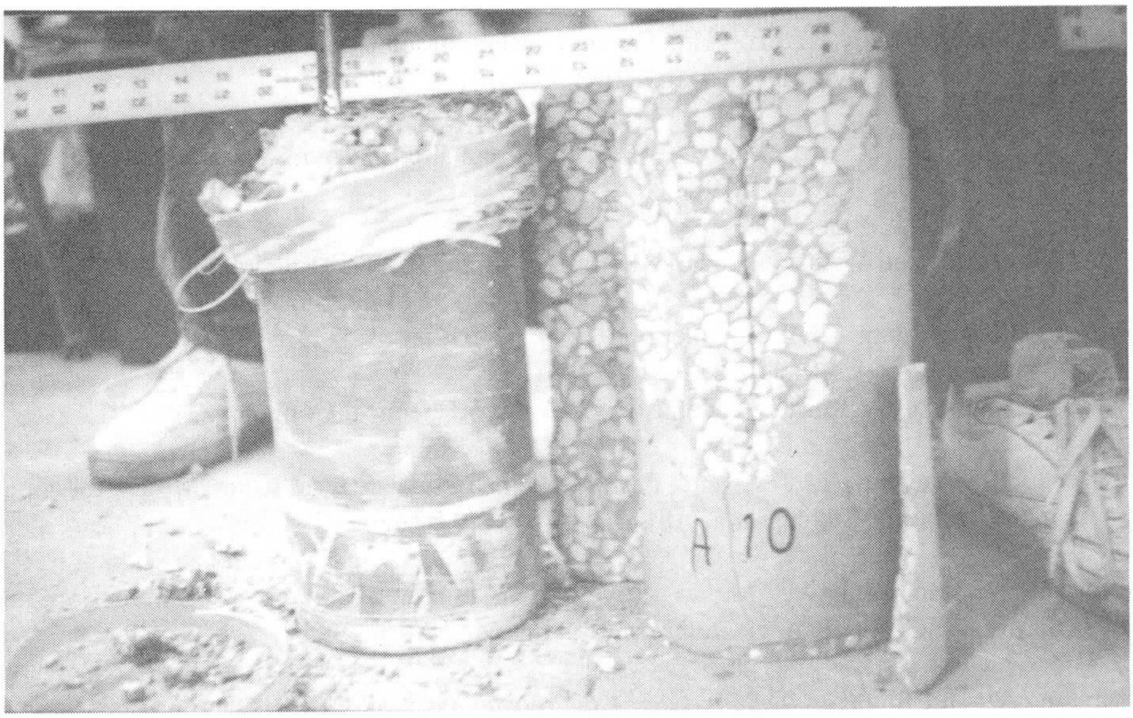

Fig.4 Control and Composite Specimens after Failure

6. CONCLUSIONS

A novel type of composite column is proposed that is similar to the classic steel-concrete composite construction, except that steel has been replaced with Fiber Reinforced Plastic (FRP) shapes. Analytical and experimental studies demonstrate the advantages of the proposed system. A mere 1.6 mm fiberglass tube increased the compressive strength of a standard concrete cylinder by 140%. The jacket is specially suitable for seismic regions, since it increases both strength and ductility of the column.

ACKNOWLEDGEMENTS

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Behaviour of Structural Members in Case of Welding under Loading

Comportement des éléments de construction lors de soudage sous charge

Verhalten von Bauteilen bei Schweissen unter Last

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SUMMARY

Refurbishment often may require welding at steel members under loading. The deformations that occur differ greatly from those of members welded without loading. Such deformations have a great influence on the ultimate load of reinforced compression bars. The temperature field caused by welding plays an important role. For this reason investigations of the temperature field, the deformations and the structural behaviour of reinforced members have been carried out.

RÉSUMÉ

Lors d'assainissement d'ouvrages, il est souvent nécessaire d'effectuer le soudage des éléments de construction sous charge. Les déformations qui en résultent sont très différentes de celles apparaissant dans des éléments de construction soudés sans charge. Les déformations de ce genre exercent une grande influence sur la charge limite des barres comprimées renforcées. Le champ de température par suite du soudage y joue un rôle important. Des études ont été réalisées sur le champ de température, les déformations qui en résultent et la capacité de charge des éléments de construction renforcés.

ZUSAMMENFASSUNG

Bei der Sanierung von Bauwerken ist es oft erforderlich, an belasteten Bauteilen zu schweissen. Die dann auftretenden Verformungen unterscheiden sich wesentlich von denen geschweisster Bauteile ohne Belastung. Derartige Verformungen haben z.B. einen grossen Einfluss auf die Traglast verstärkter Druckstäbe. Eine wichtige Rolle spielt dabei das Temperaturfeld infolge Schweissen. Aus diesem Grund wurden Untersuchungen des Temperaturfeldes, der daraus resultierenden Verformungen und des Tragverhaltens verstärkter Bauteile durchgeführt.

1. INTRODUCTION

Refurbishment often may require welding at steel members under loading. The deformations occurring differ greatly from those of members welded without loading. Such deformations have a great influence on the ultimate load of reinforced compression bars. The temperature field caused by welding [1,2] plays an important role.

Neither design codes nor the standard publications known to the authors include design rules in case of refurbishment by welding. Consequently the engineer depends on the unloading of the corresponding structural members (often being expensive and extensive).

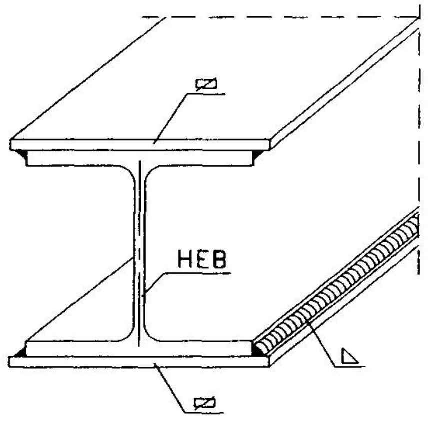
Therefore investigations of the temperature field, the deformations and the structural behaviour of reinforced members have been carried out at the Chairs of Steel Structures of the Technical Universities of Cottbus and Bochum [5].

2. TESTS

2.1 Measurements of the temperature field and deflections caused by welding

Beams and columns (HE 120B resp. HE 200B) of 3 m length were reinforced by welding (E-manual) of cover plates (90 x 10 and 150 x 10 resp. 170 x 10 and 230 x 10 mm) on the upper and/or lower flange (Fig.1). The thickness of the welds was 4 mm for girders resp. 5-6 mm for columns. Girders and columns were distinguished according to their different welding positions and speeds.

The temperature at the surface of the upper cover plate of the specimen was measured by thermovision (measuring field in Fig. 1), additional temperature measurements at selected points were carried out by thermal elements. Moreover the deflection was measured at 11 points equally arranged over the full member length. Three test specimen were available for each cross-section. Altogether 12 tests on columns and beams were carried out. The welding sequence is given in Fig. 1 (points 1, 2, ..., 24).



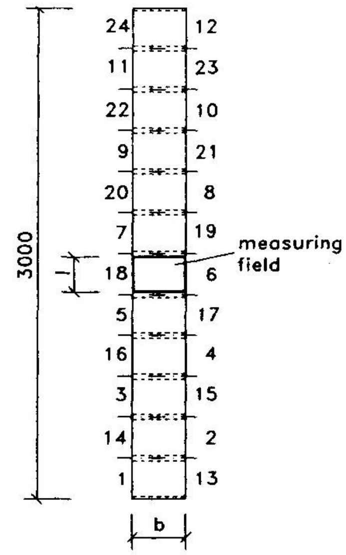


Fig.1 Test specimen. a Perspective, b Top view, measuring field and welding sequence

Fig. 2 shows a typical temperature field obtained by thermovision measurement (1min 55s after ignition of the electrode).

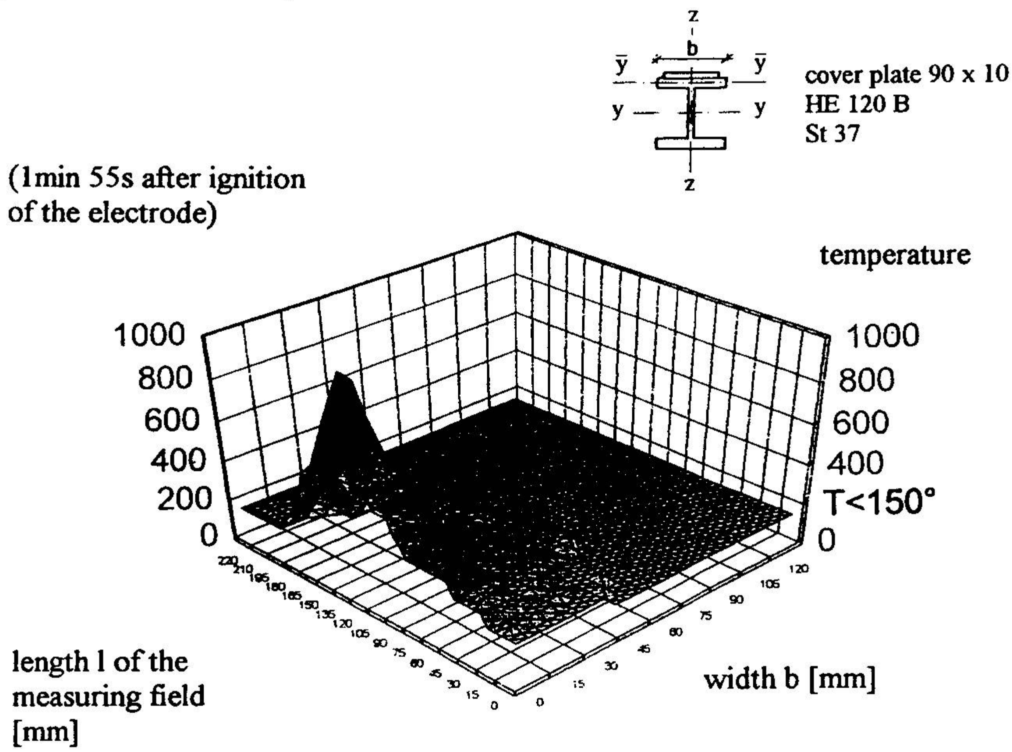


Fig. 2 Temperature field obtained by thermovision

A corresponding stress distribution over the central line of the flange at the cross section is given in Fig. 3.

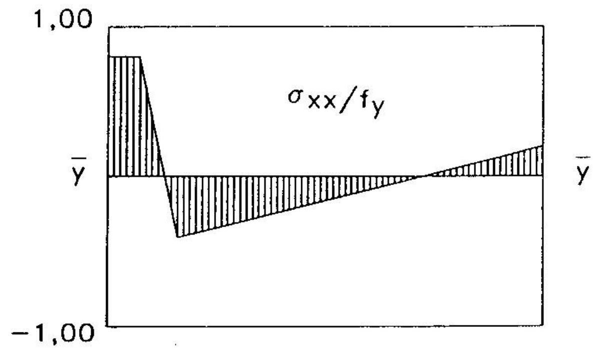
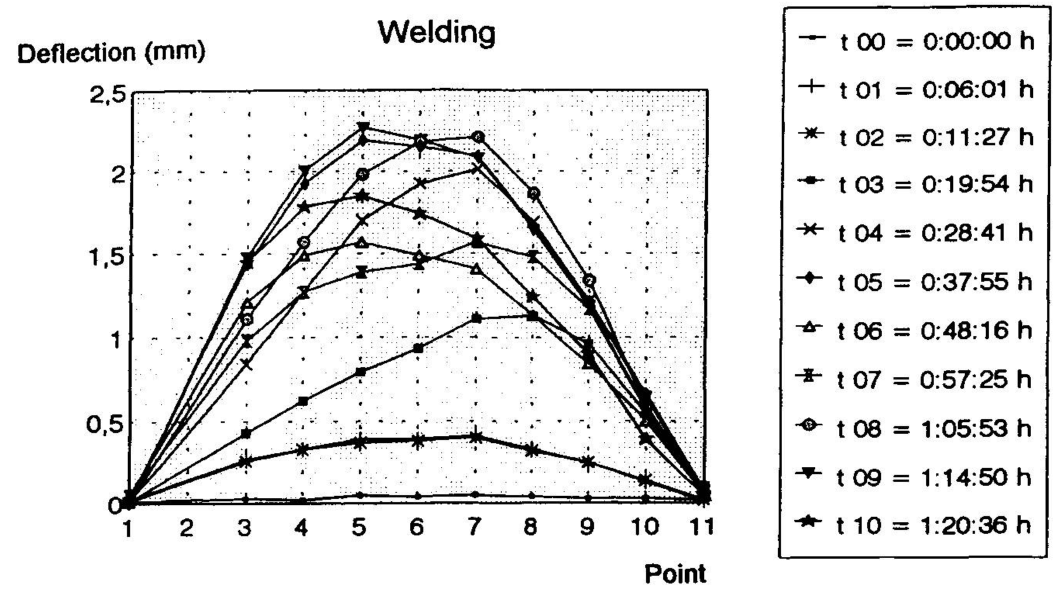


Fig.3 Typical stress distribution

Fig. 4 shows the measured time-variant deflection curves of a column (HE200B plus 1 cover plate 230 x10 mm) during welding (a) and cooling (b).

a



b

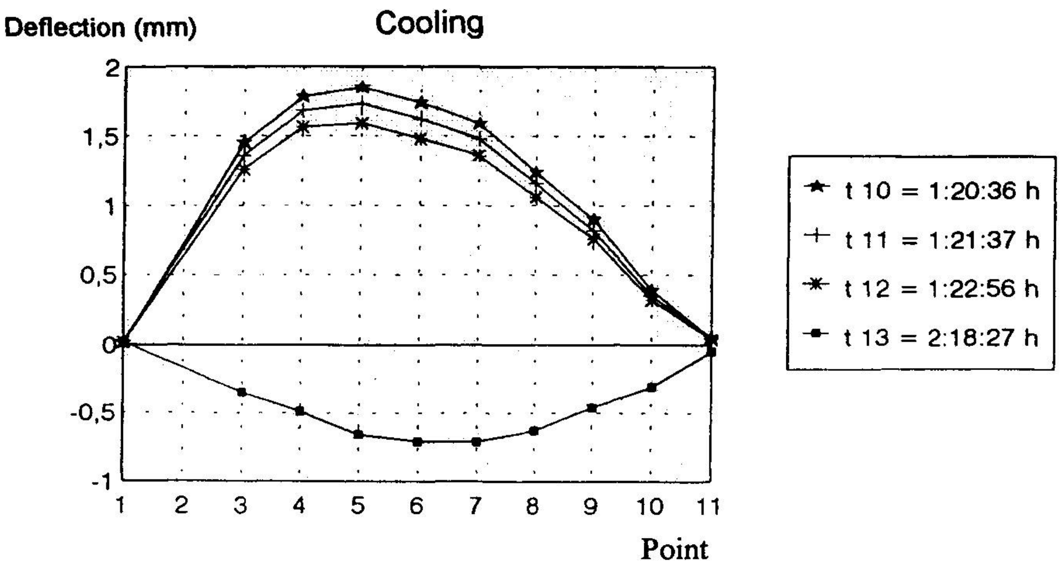


Fig. 4 Deflection curves for a column

The thermovision is a proper means for the measurement of temperature fields. The main results of these investigations are:

- the distribution of energy between the profile flange and cover plate influences greatly the temperature field emerging, the amount of energy in the web may be neglected.
- the heat flow in the cross section occurring vertically to the contact cover plate-profile is of secondary importance.
- the temperature distribution measured corresponds sufficiently with the source method [1,2] calculated quasi-stationary.

The bending lines measured and calculated correspond only partly, caused above all by the assumption of the quasi-stationary temperature field. (Notice: The curvatures calculated from the temperature field at the discrete profiles of the cross section lead via numerical integration to the bending line).

2.2 Ultimate load tests

In addition, 12 columns (HE 120B) have been investigated in ultimate load tests:

- 2 without reinforcement,
- 4 reinforced by (1 or 2) cover plates (70 x 10) but without loading before reinforcing and
- 6 reinforced by (1 or 2) cover plates and subjected to an excentric (e = 55 mm) compression force of 200 kN before reinforcing.

A typical load-deformation curve for a column consisting of HE120B plus 2 cover plates 70 x 10 mm (sequence of reinforcement: first compression flange, next tension flange) subjected to preloading is given in Fig. 5. It can be observed that welding and cooling lead to a significant increase of deflection at the level of 200 kN.

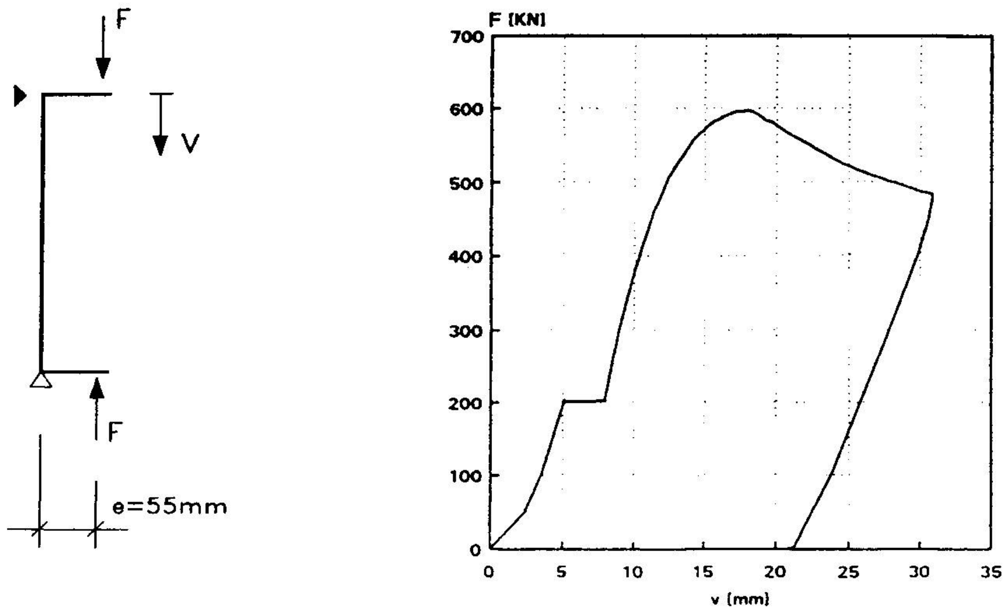


Fig. 5 Load deformation curve

Table 1 contains the load capacities according to German code DIN18800"Steel structures" (without partial safety factors and with real material properties) including the used buckling curves (a,b,c).

Furthermore, the measured load capacities for the cases with and without preloading are presented. For example, the reinforcement of a column by two cover plates leads to an increase of the load capacity from 434.9kN to 648.0 kN, i.e. 49% (without loading before reinforcing) respectively of only 35% (with loading before reinforcing).

Finally the ratios between the calculated and measured load capacities are given. The calculated load capacities of unreinforced columns and reinforced columns without preloading do not differ significantly from the measured ones. In case of preloaded reinforced columns the measured load capacities are up to 10 % below the calculated ones. Welding stresses and deformations are the reason for that. Even a change of the relevant buckling curve (from b to c) is unsufficient.

Table 1: Comparison between calculated and measured load capacities

Column	Calculated load capacity R_c (kN), buckling curve	Measured load capacities R_m(kN) and R_c/R_m ratios without preloading	with preloading
without reinforcement	426.9 (a)	434.90 1.02	
one-sided reinforced	574.0 (b) 553.5 (c)	592.40 1.03 1.07	548.20 0.95 0.99
double-sided reinforced	651.8 (b) 631.4 (c)	648.00 0.99 0.97	587.10 0.90 0.93

3 CONCLUSION

From the current point of view a structure has to be individually calculated for every reinforcing case if you either may not or do not intend to unload it. A PC-computer programme is required for practical use.

At present a computer programme for compression reinforced bars under loading has been developed in Cottbus. The ongoing numerical investigations include:
- the estimation of the temperature field unsteady caused by welding,
- the estimation of strains and residual stresses and finally
- the estimation of the load-deformation curve for the whole member.

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Repair of Structures through External Bonding of Thin Carbon Fiber Sheets

Réparation de structures au moyen de lamelles en résine époxyde renforcée de fibres de carbone

Sanierung von Tragwerken durch Aufkleben von kohlenstoffaserverstärkten Epoxidharzlamellen

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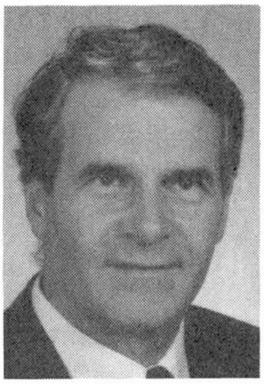

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SUMMARY

This paper seeks to demonstrate how advanced polymer matrix composite materials developed for high-performance aircraft can offer advantages for rehabilitation and retrofitting of existing civil engineering structures through external bonding of thin carbon fiber/epoxy (CFRP) sheets. Easy handling on site due to the light weight of the CFRP sheets helps to reduce labor costs. For this reason commercial use has increased in Switzerland.

RÉSUMÉ

Ce travail montre comment les matériaux composites à base de fibres à hautes performances développés pour l'industrie aéronautique et aérospatiale peuvent offrir des avantages majeurs pour l'assainissement des ouvrages d'art sous forme de minces lamelles collées en résine époxyde renforcée de fibres de carbone. Le maniement aisé de ces lamelles légères permet une économie de travail importante sur le chantier. C'est la raison pour laquelle leur utilisation commerciale a rapidement augmenté en Suisse.

ZUSAMMENFASSUNG

In dieser Arbeit wird dargestellt, wie Hochleistungsfaserverbundwerkstoffe, welche für die Luft- und Raumfahrt entwickelt wurden, neuerdings durch Aufkleben von dünnen kohlenstoffaserverstärkten Epoxidharzlamellen (CFK) zur Sanierung von Tragwerken eingesetzt werden. Die einfache Handhabung der leichten Lamellen erlaubt es, auf der Baustelle Arbeitsstunden zu sparen. Aus diesem Grund hat die kommerzielle Anwendung dieses neuen Verfahrens in der Schweiz stark zugenommen.

1. INTRODUCTION

Deterioration of reinforced and prestressed concrete bridges due to corrosion initiated by the application of deiceing salts and continual upgrading of service loads and volume of traffic on bridges has resulted in thousands of bridges the need for retrofitting. Even a much greater number of other structures require strengthening for a greater diversity of reasons. These considerations illustrate the great importance of effective renovation methods for existing structures. Thus, the method of post-strengthening structures by the bonding of steel plates or fiber-reinforced composite laminates gains in significance.

2. WHEN DO STRUCTURES HAVE TO BE POST-STRENGTHENED?

When do structures have to be post-strengthened? These measures are required when structures must take on new tasks involving increased loads. Furthermore, during the process of modernization, individual supports and walls may be removed, thus leading to a redistribution of forces and the need for local reinforcement. In addition, structural strengthening may become necessary when in the course of time damages occur due to normal usage or environmental factors.

3. CRITERIA FOR MATERIAL EVALUATION

Today in Western Europe and other parts of the world the technique of bonded steel plate strengthening is widespread and state of the art. Nevertheless, outdoor creep tests over extended time periods at the EMPA show that long-term problems concerning corrosion behavior must be expected in outdoor applications. EMPA-researcher observed "smaller traces of rust" on unprimed as well as primed steel-plate bond surfaces even after 3 years of exposure to weather. They grew larger during the course of the test. After 15 years the areas now range up to 10 mm in diameter. These tests are being continued at the EMPA and indicate a weakness in the strengthening with steel plates. However, steel plates also have other disadvantages.

During renovation work, particularly on bridges, only a limited amount of mechanized lifting machines are available on site. In the interior of box girders for example the strengthening plates have to be carried to the point of installation by hand. Due to handling limitations on site the steel plates are rarely longer than 6 to 8 m. Thus, if the strengthening work involves greater lengths, the plates must be abutted. Abutments cannot be welded together since the welding temperatures would destroy the bond. For this reason abutments of steel plates have to be formed from single-shear lap joints. On the other hand, high-strength fiber composite laminates are relatively thin and can be delivered to the construction site in rolls, in lengths of up to 300 m or more. Compared to steel plates their application is greatly simplified.

The first task was to find the best suitable fiber composite material for this application. The results shown in [1, 2] clearly show that carbon-fiber reinforced plastic laminates most closely fulfill the requirements for the post-strengthening of structures. Therefore all further discussion will be restricted to carbon-fibre reinforced plastics (CFRP).

4. MANUFACTURE OF CARBON-FIBER REINFORCED PLASTIC LAMINATES

The laminates are manufactured using a pultrusion process. The pultrusion principle is comparable with a continuous press. Normally 12k rovings (12,000 parallel filaments) are pulled through the impregnated bath, formed into laminates under heat and thereafter hardened. These laminates are unidirectional, i.e. the fibers run only in the longitudinal direction. Correspondingly the laminate strength in this direction is proportional to the fiber strength and therefore very high. In Switzerland (Stesalit AG, 4234 Zullwil, Switzerland) laminates are now made of Toray T700 fibers with a fiber strength of 4,900 MPa. With a fiber volume fraction of about 65% the resulting laminate strength amounts to about 3,000 MPa in the longitudinal direction.

5. COMPOSITE OF FIBER REINFORCED PLASTIC (CFRP) AND CONCRETE

In order to achieve an optimum composite, the preparation of the bonding surfaces of the two composite partners is very important. The CFRP laminates must be well ground on the bonding side. The outermost layer, normally matrix-rich, has to be removed to expose the fibers. Just before bonding, the bonding surface has to be carefully cleaned. This must be repeated until the washcloth no longer blackens. The concrete surface is treated by sand blasting, high pressure water jets, stoking or grinding; shortly before the bonding it is cleaned with a vacuum cleaner. The concrete must be at least 6 weeks old with a tensile strength of 1.5 MPa or higher.

Classical, highly filled epoxy resin adhesive is employed for the bonding. The adhesive is applied Λ-shaped to the CFRP laminate so that the extra adhesive is squeezed out when the laminate is pressed to the concrete structure. For a laminate width, for example, of 200 mm, the "peak height" in the middle of the laminate amounts to about 10 mm. The laminates must be pressed to the concrete until hardening using vacuum bags or other techniques. The methods developed by the EMPA are described in detail by Deuring [3].

6. CONSIDERATIONS FOR NOT PRETENSIONED, BONDED CFRP LAMINATES

In the middle of the 1980's, based on tests with middle sized beams, it was shown (Meier [4]) that post-strengthening with CFRP laminates is possible. As a result of comprehensive investigations at the EMPA Kaiser [5] came to the following conclusions:
Post-strengthening of structural components with CFRP laminates may be calculated in flexure analogously to conventional reinforced concrete. Special attention must be paid to the formation of shear cracks in the concrete. Such shear cracks lead to an offset on the strengthened surface. This generally causes a peeling-off the strengthening laminate. Thus, shear crack formation is a design criterion. Flexure cracks are spanned by the laminate and do not influence the carrying capacity. The carrying capacity can be predicted accurately in advance.

The calculation model for the anchoring of the laminates agrees with experiments over a wide range. For short anchoring lengths the model underestimates the carrying capacity of thick laminates and overestimates it for thin laminates .

Bonded CFRP laminates have a very positive influence on crack development of a reinforced concrete beam. The cracks are more finely distributed and the sum of the crack widths is greatly reduced. Even after exceeding the yielding point of the inner reinforcement the crack growth remains under control up to failure thanks to the elastic CFRP laminate.

CFRP laminates exhibit excellent fatigue behavior. Through the bonding of the CFRP laminate the inner reinforcement is relieved. Tests with very high vibration amplitudes yielded excellent results [5] over more than 10 million load cycles.

7. DESIGN FOR PRETENSIONED, BONDED CFRP LAMINATES

In EMPA Report Nr. 224 [3] Deuring shows the potentials of pretensioned laminates. Following the pre treatment of the laminate and concrete surfaces described above the CFRP laminate is tensioned to 1,000 MPa using a special tensioning device. The adhesive is applied before pressing the tensioned laminate to the structure. After hardening of the epoxy resin the force-transfer zones at the laminate ends are provided with pressure plates in order to transfer the large forces of the laminate into the concrete. As a result of a pressing force perpendicular to the laminate surface the shear strength of the concrete is increased; if horizontal micro cracks occur, the laminate remains successfully anchored to the structure owing to the effective interlocking of the laminate. Finally, the external tension is lowered and the structure is not only strengthened through a CFRP laminate, it is as well prestressed . In this way, even existing cracks can be closed.

Deuring [3] comes to the following conclusions: The calculation procedure closely predicts the load behavior of a structure post-strengthened with a pretensioned CFRP laminate. Since the CFRP laminate has no plastic deformation reserve the highest flexural resistance of a strengthened section is reached when laminate failure occurs simultaneously with the yielding of the steel and before the concrete fails. The type of failure is strongly influenced by the laminate cross-section and the tensioning force. Tension and deformation calculations may be carried out with conventional methods. Test results on realistic beams confirm the validity of the classical assumptions. Pretensioning reduces the danger of peeling off, mentioned above for non-pretensioned laminates. The total sum of the crack widths is influenced even more favorably than with non-pretensioned laminates. The excellent fatigue behavior exceeds all expectations.

8. SOME APPLICATION EXAMPLES IN BRIDGE AND BUILDING CONSTRUCTION

The Ibach Bridge [6], built in 1969 is located in Emmenbrücke, a suburb of Lucerne/Switzerland. It crosses over National Highway N2 (Basel-Gotthard-Chiasso) and the Emme and Reuss rivers. The bridge is designed as a continuous beam structure with 7 spans and a total length of 228 m. In the span, which crosses the six lane N2, a prestressed cable in a web was accidentally cut. The repair work was undertaken in the Summer of 1991. Three CFRP laminates with a total mass of 6.5 kg were bonded. In order to have obtained the same results with steel 175 kg would have been necessary. Results of loading tests show that experimentally measured elongations agree with the calculated values. This strengthening was carried out by the firm StahlTon AG, 8034 Zurich.

The historical Wooden Bridge in Sins/Switzerland [6], constructed in 1807, crosses the Reuss river and consists of two spans, each of 30.8 m length. Problems arose with the crossbeams. Under the permissible load of 200 kN per vehicle the oak beams exhibited excessive deflection. To limit this deflection the beams subjected to the highest loads were successfully stiffened in the spring of 1992 with CFRP laminates having a longitudinal modulus of elasticity of 300 GPa.

An elevator was to be subsequently installed in the City Hall of Gossau St. Gall/Switzerland [6] in 1991. Before cutting the elevator cross-section from the reinforced concrete roof the "replacement reinforcement" in the form of CFRP laminates was bonded to the "future" edges. In this example the CFRP laminates were employed for aesthetic reasons. The architect wanted to make the post-strengthening invisible; this succeeded very well as the laminates were only 1 mm thick. In this case, corrosion, mass and fatigue behavior were no criteria.

In Spring 1993 the Migros Supermarket in Uzwil/ Switzerland was expanded. In order to connect the new sales area with the old one the existing outer wall, a supporting brick construction, had to be removed over a length of 13 m. The building contractor did not want any additional supports in the new passage. The carrying capacity of the existing parking deck had to be maintained. The individual strengthening laminates were 15.5 m long. With steel plates the installation weight would have amounted to 120 kg compared to 3.5 kg with CFRP laminates.

Major projects followed in 1994: The Main Railway Station in Zurich where the load carrying capacity of a large, heavy loaded concrete slab had to be increased by a factor of 1.4 and the huge chimney of the Nuclear Power Plant Leibstadt, where the safety factor was of outstanding importance. All these cases described above were executed again by the firm StahlTon AG.

The ceiling in the tankroom of the Paper Mill in Utzensdorf (Switzerland) had to be reinforced due to the installation of a new machine unit. The reinforcement was performed in January 1995 with forty meters of CFRP sheets. The advantages of the CFRP strengthening method were decisive for the contractor due to the low working height of only 800 mm above the tanks and the small entrance recess into the tankroom. The ultimate tensile load capacity of this type of CFRP sheet is 210 kN per sheet and the dead weight is only 140 grams per meter. This CFRP sheets have been developed by Hilti in co-operation with EMPA and they were offered by Kilcher HBC, the Hilti representative for rehabilitation systems in Switzerland.

Until January 1995 there were approximately 80 applications of the method of external bonding of thin carbon fiber/epoxy (CFRP) sheets in Switzerland.

9. OUTLOOK

Based on the research and development work at the EMPA the application of CFRP laminates is already almost routine for the firms StahlTon AG in Zurich, Sika AG in Zurich and Kilcher HBC, the Hilti representative for rehabilitation systems in Switzerland, in the strengthening of existing structures. Originally it had been assumed that this technique would only be cost-efficient if there were very high requirements relative to corrosion, fatigue performance and light weight. However after further price decreases of carbon fibers this method has become also cost efficient for applications in which not all of these requirements are present.

In our opinion fiber composites have excellent chances in specific civil engineering applications as described here or for seismic strengthening [7, 8] or for stay cables [9]. However, even in the future when further decreases in price of carbon fibers can be expected they will not replace classical materials such as steel, concrete and wood but rather supplement them as called for.

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Strengthening of Structures Using Epoxy Bonded Steel- or Fibre Reinforced Plastic Plates

Renforcement de structures par collage de plaques métalliques ou en plastique renforcé de fibres

Tragwerksverstärkung mittels geklebten Stahl- oder faserhaltigen Kunststoffplatten

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SUMMARY

This paper describes tests on concrete beams strengthened for shear using bonded plates of steel or GFRP and CFRP (Glass or Carbon Fibre Reinforced Plastic) materials as outer reinforcement. The results show that it is possible to achieve a good strengthening effect with either steel or CFRP plates. In the design stage, however, the actual strain in the concrete at fracture should be considered for best results.

RÉSUMÉ

L'auteur décrit divers essais effectués sur des poutres en béton, dont le comportement au cisaillement a été amélioré par le collage de plaques soit en acier, soit en matière plas-tique armée de fibres de verre ou de carbone, appliquées comme armature extérieure. Les résultats d'essais montrent qu'il est possible de parvenir à un excellent renforcement par utilisation de l'un de ces deux types de plaques. Au cours du dimensionnement, il faudrait toutefois prendre en considération les allongements à la rupture effectifs du béton.

ZUSAMMENFASSUNG

Es werden Versuche an Betonbalken beschrieben, die zur Verbesserung des Schubtragverhaltens mit geklebten Platten aus Stahl bzw. glas- oder kohlenstoffaserverstärktem Kunststoff (CFK) als äussere Bewehrung versehen wurden. Die Versuchsergebnisse zeigen, dass mit Stahl- oder CFK-Platten eine gute Verstärkungswirkung erreicht werden kann. Bei der Bemessung jedoch sollten die tatsächlichen Betonbruchdehnungen verwendet werden.

1. INTRODUCTION

1.1 General

As most of us know concrete is a building material with a high compressive strength but poor tensile strength. A beam without any form of reinforcement will crack and fail with a relative small load. In most cases the failure happens suddenly and in a disastrous manner. The most common way to reinforce concrete structures is therefore to cast steel reinforcing bars into the fresh concrete during the building stage. The reinforcement interacts with the concrete as though they were one part and together carry the load. This implies that the amount of reinforcement has to be determined before the structure is built. Since, in normal cases, a concrete structure has a very long life expectancy, it is not unusual that the demands placed upon the structure change with time. The structure may in the future be required to bear greater loads or to fulfil new standards in the future. When such a situation arises it should be determined whether it is more economical to strengthen the exisiting structure or to replace it. In comparison with the building of a new structure is it often more complicated to strengthening an existing one.

A repair method used world-wide to strengthen existing concrete structures is to attach steel plates in the form of additional reinforcement on the outside of the structure using a cold cured epoxy adhesive. However, before joining the plates and the concrete together, certain steps must be taken. First the latiance of the concrete and the rust mill of the steel plates have to be removed by sandblasting. The surfaces need to be cleaned very carefully. The steel plates are then treated with a primer to prevent corrosion. The last step is to join the two materials together and after about one week the hardening process for the adhesive is concluded.

So far, most of the structures in the world strengthened with this method have been strengthened for bending. However, this paper presents tests performed in Sweden, at Luleå University of Technology, with concrete structures strengthened for shear using steel or FRP (Fibre Reinforced Plastics) materials. Also, design formulas for engineering purposes are presented, both for bending and shear.

1.2 Background

The project to strengthen existing concrete structures started in Sweden in 1988. The Swedish Road Administration needed to increase the load bearing capacity of existing highway bridges in order to meet the standards in the EEC (European Economic Community). From an internal investigation of existing bridges it was found that almost 1 300 of 9 000 existing concrete bridges needed to be replaced or strengthened. The method of gluing outer reinforcement could then be applied. At Luleå University of Technology extensive reserch has been performed for this strengthening methodology; practical tests, theoretical derivations, and a full scale test have been performed. The method of increasing a concrete structure's bending capacity or its stiffness with steel plates bonded to its soffits is today quite well understood. Nevertheless, strengthening a concrete member for shear is more difficult. This paper will discuss some problems in strengthening concrete structures for shear and how it is possible to perform this type of strengthening. Before the results are presented, however, a short presentation of the engineering formulas for design are discussed.

2. THEORY

2.1 Truss model

Assuming that we have full interaction between concrete, steel, and adhesive, that plane sections remain plane during loading (Bernoulli-Navier-hypothesis) and that we have a known relation between strains and stresses, then a truss model for beams will give us design criteria for plate bonded structures in the elastic domain. Let us study a beam strengthened for both bending and shear with inclined stirrups and steel plates as shown in figure 1.

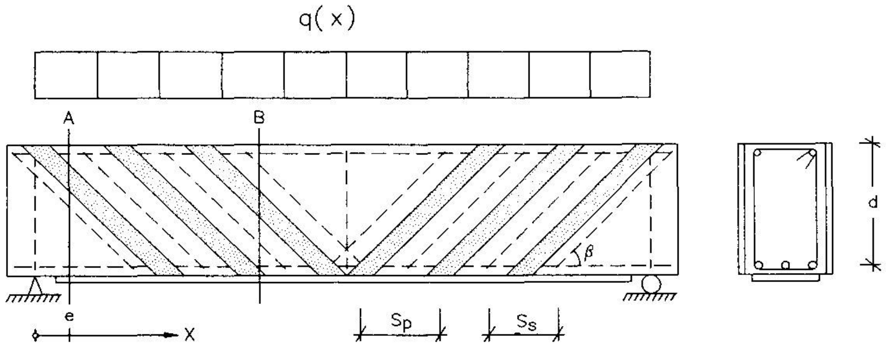

Fig. 1 Strengthened concrete beam with a distributed load q(x)

In the Swedish code for concrete structures, BBK 79, the following expressions are used for the load V_c carried by the concrete:

$$V_c = b_c d f_v \tag{1}$$

where

b_c = width
d = effective depth
f_v = formal concrete shear stress

$$f_v = \xi(1 + 50\rho)0.30 f_{ct} \tag{2}$$

where

$$\begin{aligned}
\xi &= 1.4 && \text{when the depth} && d \le 0.2\ \text{m} \\
\xi &= 1.6 - d && && 0.2\ \text{m} < d \le 0.5\ \text{m} \\
\xi &= 1.3 - 0.4d && && 0.5\ \text{m} < d \le 1.0\ \text{m} \\
\xi &= 0.9 && && 1.0\ \text{m} < d
\end{aligned} \tag{3}$$

$$\rho = \frac{A_b}{b_c d} \tag{4}$$

where A_b = bending reinforcement area in the tensile zone

If we cut out a section between point A and point B as in figure 2, simple equilibrium equations can be derived which can be stated in a very clear and descriptive way, see also [1]. From the equilibrium equations and using the notations in BBK 79, it is possible to write the following two equations for the design of strengthening with bonded plates. In the formulas below, f_{yd} denotes the yield strength, s the spacing of the shear reinforcement, and the notations s and p as exponents denote stirrups and plates, respectively.

$$\frac{A_{sv}^p}{s_p} = \frac{1}{f_{yd}^p\,0.9d}\left[V_A - b_c d f_v - \frac{f_{yd}^s A_{sv}^s\,0.9d}{s_s}\right] \tag{5}$$

$$A_p = \frac{1}{f_{ydp}}\left[\frac{M_A}{0.9d} + \frac{1}{2}V_A + \frac{1}{2}b_c d f_v - f_{ydb}A_b\right] \tag{6}$$

The first equation gives the cross section area, A_{sv}^p, for shear strengthening and the second gives the necessary cross section area, A_p, for bending strengthening. In these formulas, however, it is assumed that the load, the quality of the steel, and the distance between the plates are decided or known before strengthening takes place or else iterative calculations are needed. Nevertheless, when the concrete starts to fracture more accurate methods are needed, i.e NLFM (Non Linear Fracture Mechanics) methods may be suitable. However, these methods are not treated in this paper, but can be found in [1].

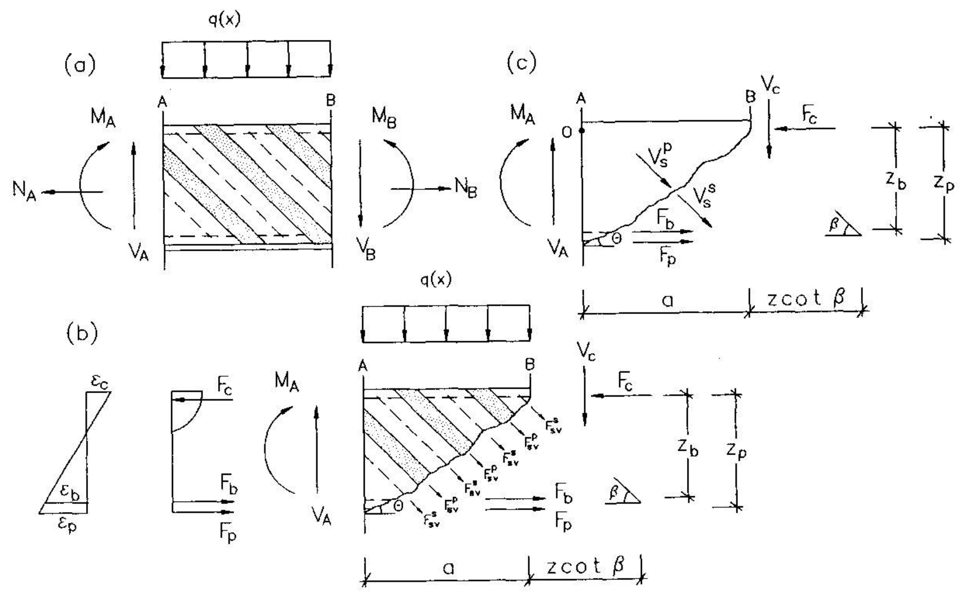

Fig. 2 Section A-B and corresponding forces acting in the studied section

3. BEAMS TESTED IN SHEAR

3.1 Preparation

The beams tested in shear, denoted series F, consisted of 6 concrete beams with the dimensions shown in figure 3. One beam, F1, was left unplated to act as a reference beam; however, even this beam was strengthened after failure. The other beams were all plated, most of them with steel plates, but also with GFRP (Glass Fibre Reinforced Plastics) and CFRP (Carbon Fibre Reinforced Plastics) plates. The sides of the beams were sandblasted and cleaned very thoroughly before the plates were attached. The beams that were strengthened with steel plates had a uniform glue layer with a thickness of 2.0 mm, whereas the beams with GFRP and CFRP plates had a glue layer that was estimated to be 0.5 mm thick.

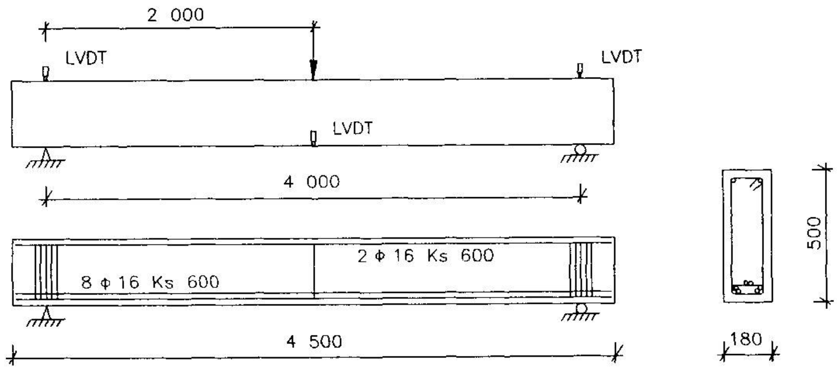

Fig. 3 Test specimens and test arrangements for series F

All of the beams were tested in three point bending, and measurements were made of the force, the mid span deflection, and the settling over the supports, which is shown in figure 3. The crack propagation during loading was also recorded. The loading was deformation controlled with a deflection rate of 0.3 mm/min. In figure 4 all of the beams are shown before loading. It should also be mentioned that in addition of beam F1, beam F2 was used in two of the tests. In table 1 the material parameters are summarised together with data for the concrete and the epoxy bonded plates. All the notations in table 1 can be found in figure 4 where E is the Young's modulus of the strengthening plates.

The low yield strength of the steel plates is readily apparent in contrast to the high yield strengths of the GFRP and especially CFRP plates.

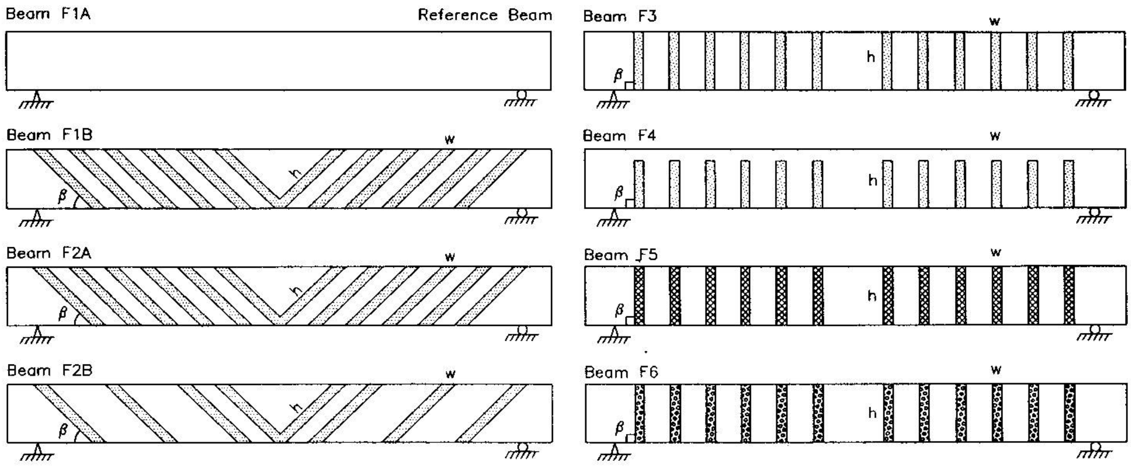

Fig. 4 Beams used in test series F, before loading

Beam	Plates type	h [mm]	w [mm]	t [mm]	s [mm]	β	f_{cc} [MPa]	f_{ct} [MPa]	f_{yd} [MPa]	f_{ud} [MPa]	E [GPa]
F1A	---	---	---	---	---	---	32.8	3.7	---	---	200
F1B	steel	$500\sqrt{2}$	80	2.9	300	45°	32.8	3.7	170	289	200
F2A	steel	$500\sqrt{2}$	80	2.9	300	45°	43.2	3.7	171	289	200
F2B	steel	$500\sqrt{2}$	80	2.9	600	45°	43.2	3.7	171	289	200
F3	steel	500	80	2.9	320	90°	56.3	4.2	161	289	200
F4	steel	400	80	2.9	320	90°	58.4	4.0	161	289	200
F5	GFRP	500	80	0.9	300	90°	48.6	4.0	564	612	22
F6	CFRP	500	50	1.2	300	90°	60.0	4.2	2497	2497	167

Table 1 Material parameters and dimensions of epoxy bonded plates in series F

Beam F1A is the reference beam and F1B the reference beam after strengthening. Beam F2A is strengthened in the same manner as beam F1B, however, the beam was not precracked before it was strengthened. Furthermore, beam F2B is the same beam with every other plate removed. Beams F3 and F4 were also strengthened with steel plates bonded to their sides. The difference between these beams and the ones already discussed was that the plates were attached vertically. However, the distance between the plates was 320 mm. On one side of beam F3, strain gauges were glued to the steel plates, see figure 5. Furthermore, the heigth of the steel plates for beam F4 was just 80 % of the plates used for F3. The aim of this test was to investigate the need for anchorage in the compressive zone. Beams F5 and F6 was strengthened with vertical GFRP and CFRP plates, respectively.

3.2 Results

In table 2 are the results from the tests summarised. Nevertheless, before the results are presented, a short explanation of the table is needed. Beam $F1B_1$ is the strengthened reference beam and beam $F1B_2$ is the same beam but with every other plate removed. Beam F5A and F6A are the beams strengthened with GFRP and CFRP, respectively, where the theoretical yield limit for the plates has been used as a design criterion. Beam F5B and F6B are the same beams, but here the actual stess level in the plates at failure has been considered. Let us now study the results from the tests. A comparison of the beams F1B and F2 reveals that despite that beam F1B was precracked, it could support the same load as beam F2. This means that it may be possible to strengthen an existing cracked structure so that it attains the same bearing capacity as an uncracked one. Beam F3, strengthened with vertical plates, almost reached the theoretical design level. If we study the curve of the measured strains, see figure 5, we can see where the steel plates start to contribute to the load bearing capacity. If we follow the strain curves over the time of loading, we see that, for up to almost 70 % of the failure load, the strains on both sides of the midsection of the beam are quite symmetrical.

Beam	Vs [kN]	Vc [kN]	Vs + Vc [kN]	Vexp [kN]	$\frac{V_{exp}}{V_c + V_s}$	$\frac{V_{exp}}{V_{ref}}$	Type of failure
F1A	---	122.5	122.5	122.5	1.00	1.0	shear
$F1B_1$	141.5	122.5	264.0	215.0	no comparison made, bending failure		
$F1B_2$	70.8	122.5	193.3	205.3	1.06	1.8	shear
F2A	142.2	122.5	264.7	212.5	no comparison made, bending failure		
F2B	71.1	122.5	193.6	202.5	1.05	1.7	shear
F3	94.3	122.5	216.8	200.0	0.92	1.6	shear
F4	75.8	122.5	198.3	160.0	0.81	1.3	shear
F5A	108.6	122.5	231.1	150.0	0.65	1.2	shear
F5B	5.0	122.5	127.5	150.0	0.85	1.2	shear
F6A	409.0	122.5	531.5	155.0	0.30	1.3	shear
F6B	27.5	122.5	150.0	155.0	0.98	1.3	shear

Table 2 Theoretically calculated and experimentally obtained values for series F

Nevertheless, the divergence at the end of the loading curves is large for the most extended gauges. At failure, gauge 8 exceeded 800 µs; this is equivalent to a stress of 165 MPa which is above the yield strength, f_{yd}, of the steel used.

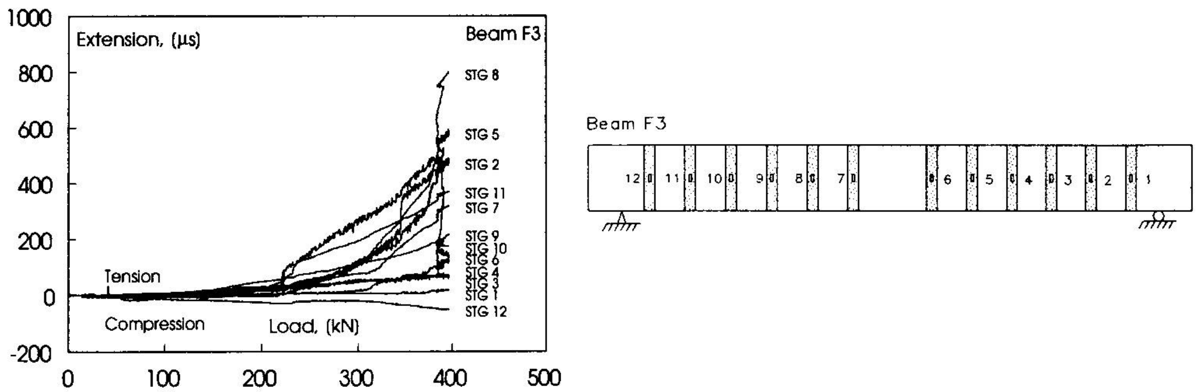

Fig.5 Placement of gauges and load-extension curves for measured strains for beam F3

The height of the steel plates for beam F4 was just 80 % of the plates used for F3. However, the load reached for beam F4 was 80 % of that reached for beam F3, which could imply that it may be possible to use, in a restricted way, a simplified model for strengthening, even if the plates can not be anchored in the compressive zone, see [1]. It is known that the concrete starts to fracture at a specific strain level; this level is in most dimensioning cases the critical design variable. Table 2 also shows that if the stress level at failure is used as a design criterion the theoretical shear level corresponds quite well to the one obtained from the tests on beams F5B and F6B. However, to keep the strain level in the strengthening plate at a moderate level two design criteria have to be fulfilled. First, the Young's modulus in the plate must be sufficently large (the larger the better) and second the cross section of the plate must have a sufficently large area. Let us study table 2. The small modulus is the reason for the limited strengthening effect of the GFRP plates. Since Young's modulus is a material parameter it can not be changed. For the CFRP plates the situation is different. Carbon fibre has a quite large Young's modulus which implies it is suitable for strengthening. However, in this particular case the cross sectional area is too small, but if the area is increased it is most likely that the shear capacity would also be increased, see also [1].

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Strengthening of Reinforced Concrete Beams by External Reinforcement

Renforcement de poutres en béton armé par précontrainte extérieure

Verstärkung von Stahlbetonbalken mit angeklebten Stahllamellen

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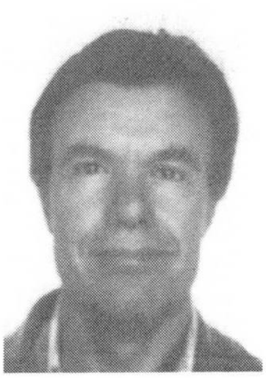

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SUMMARY

The design criteria and the methods of analysis followed in the strengthening of a reinforced concrete structure using external steel plates bonded to the existing members by means of anchor bolts and injected epoxy resin are presented. The results of the strengthening were confirmed by load tests carried out before and after the intervention. The strengthening technique is described, along with job planning and quality control procedures and data on job productivity.

RÉSUMÉ

La conception et la méthode de calcul pour le renforcement d'une structure en béton armé au moyen d'une armature extérieure en tôle d'acier attachée au béton par des boulons d'ancrage et injection d'une résine époxide sont présentés. Les résultats du renforcement ont été confirmés par des essais de charge avant et après l'intervention. La technique de renforcement, la planification et l'organisation des travaux sont présentés, ainsi que des données sur les rendements obtenus.

ZUSAMMENFASSUNG

In diesem Beitrag werden der Entwurf und die Berechnungsmethode vorgestellt, die man zur Verstärkung einer Stahlbetonkonstruktion anstellen muss, bei der eine Aussenbewehrung angewendet wird, die aus Stahlplatten besteht und durch Stahldübel und eine Epoxidharzeinspritzung im Beton fixiert wird. Die Wirksamkeit der Verstärkung wurde durch Lastproben bestätigt, die vor und nach dem Eingreifen durchgeführt wurden. Es werden auch die angewandte Technik, die Planung, Organisation und Qualitätskontrolle der Arbeit und auch einige Daten über die erreichten Ergebnisse beschrieben.

1. INTRODUCTION

In order to correct some design shortcomings, the reinforced concrete structure of the Central Post Office of Lisbon was recently subject to an important strengthening intervention.

The building has two large reinforced concrete floors consisting of a slab 0.22 m thick, supported by a grid of beams spaced 4.5 m. The main beams with a 13.5 m span are supported by circular columns. The building is subdivided by expansion joints in substructures of 40.4 m x 36.0 m (Fig. 1).

Regarding the improvement of the live load capacity and the seismic resistance of each substructure, the central columns were strengthened by jacketing and the beams by the addition of an external reinforcement consisting of bonded steel plates.

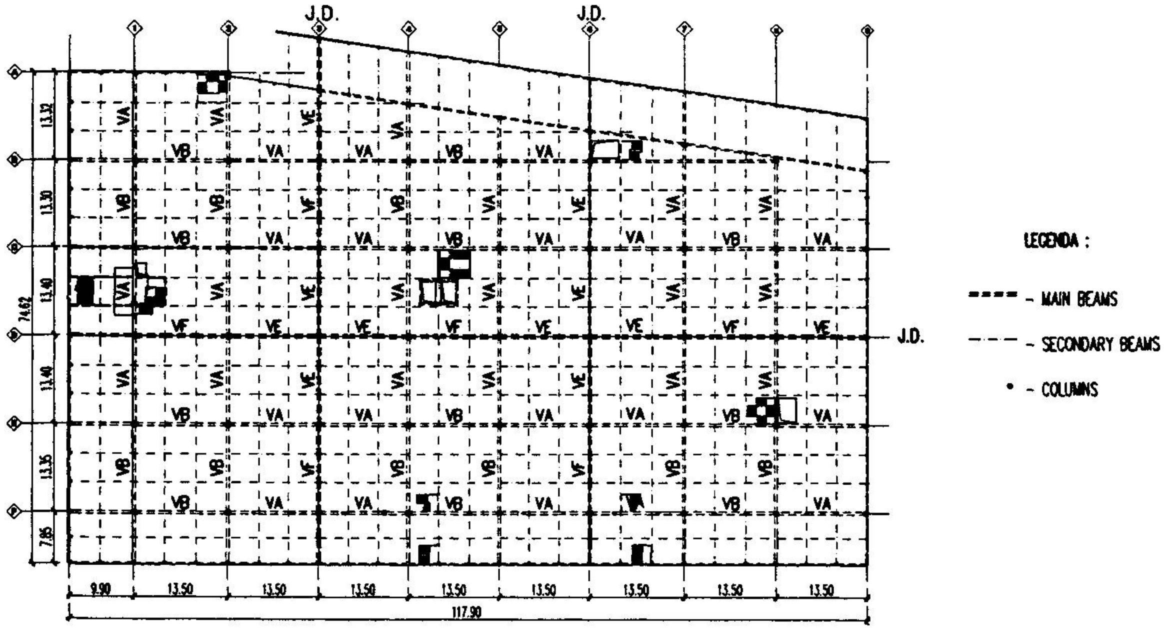

Fig. 1 - Floor level 2 - Structure

In the first phase of the project, involving the floor area of 6 600 m², over 1 500 m² of concrete beam sides were lined with 125 000 kg of steel plates and angles, around 9 m³ of epoxy resin (both for sealing mortar and injection) and around 25 000 anchor bolts were used.

2. STRUCTURAL ASSESMENT

The need to introduce new equipments in the building and the existence of systematic bending and shear cracking lead to the decision of performing an assessment of the structure safety levels.

A three dimensional bar linear elastic analysis was performed of a substructure 36 m x 40.5m with a discretisation of bar elements spaced 1.5 m. From that study, the systematic cracking was explained, as existing long term service deflections of 5 cm was estimated and the checking for the ultimate level states showed that the existing reinforcement was not enough in the critical regions of all structural elements. In some cases it was actually less than 50% of what is required to ensure the code safety levels.

The decision to strengthen the building structure both to increase its live load carrying capacity and its seismic resistance was taken by the Owner, as a result of these findings.

3. SELECTIVE STRENGTHENING AND REDESIGN

The need to maintain the strategic building in service during repairing, the difficults in increasing the beam dimensios ane the existence of a good quality concrete lead to the choice of strengthening the grid by external steel reinforcement.

The need for a significant strengthening of the columns (both the longitudinal reinforcement and stirrups needed to be incresed) lead to the choice of a jacketing solution with ordinary reinforcement and microconcrete. This paper refers only to the strengthening of the building floors.

A selective strengthening was adopted according to the following methodology, as illustrated in Fig. 2.

- The slab was assumed as a series of continuous panels 4.5 m x 4.5 m supported in the grid mesh. The bending moments obtained in this model, usual in building design, are much lower than those obtained in the FEM model where the global behaviour and different stiffness of the main and secondary beams is important. On the basis of this criteria and the acceptability of the structural model, no strengthening of the slab was required.

- For the main and secondary beams the slab load transfer was considered consistently with the slab model and to avoid the need to strength for the negative flexure resistance, redistribution was considered and the strengthening concentrated in the beam soffit.

The levels of redistribution of the linear elastic response are higher than those usually adopted for the design of new structures but are considered acceptable and supported by research which nevertheless requires deeper studies and tests.

Due to the concentration of the existing reinforcement in the beam soffit it was decided to locate the external steel in both sides of the web avoiding difficulties in the application of the mechanical bolts.

The strengthening was dimensioned by applying the monolitism coefficient technique, using the previous experience in designing and testing similar structures and the steel/resin/concrete connection.

Due to the need to restrict the extension of this paper, only bending resistance is refered to, although the grid needed also strengthening for shear.

4. EXTERNAL REINFORCEMENT STRENGTHENING TECHNOLOGY

The strengthening method used in the Lisbon Post Office job has been applied by the contractor in a large number of projects since 1983 with very satisfactory results.

It consists of an improvement of the *plate bonding* technique, allowing for a certain number of advantages in terms of ease of installation and quality of the final product (see Fig.3).

Concrete surfaces are treated using light pneumatic needle hammers, in order to remove surface laitance, loose particles and increase its the roughness.

Steel surfaces are shotblasted in shop and protected with polyethylene film for transport and handling. Protection films are peeled off immediately prior to final installation.

After surface preparation, the reinforcing steel plates are installed free of adhesive, using high strength steel bolts placed into holes drilled in the concrete member. If necessary, a steel bar detector can be used to avoid the rebars when drilling.

The epoxy bonding agent is a very low viscosity liquid which is injected continually, using two component electrical pumps, in the air gap between steel and concrete.

To allow for complete confinement of the air gap, plate boundaries and bolt heads are previously sealed with an epoxy mortar.

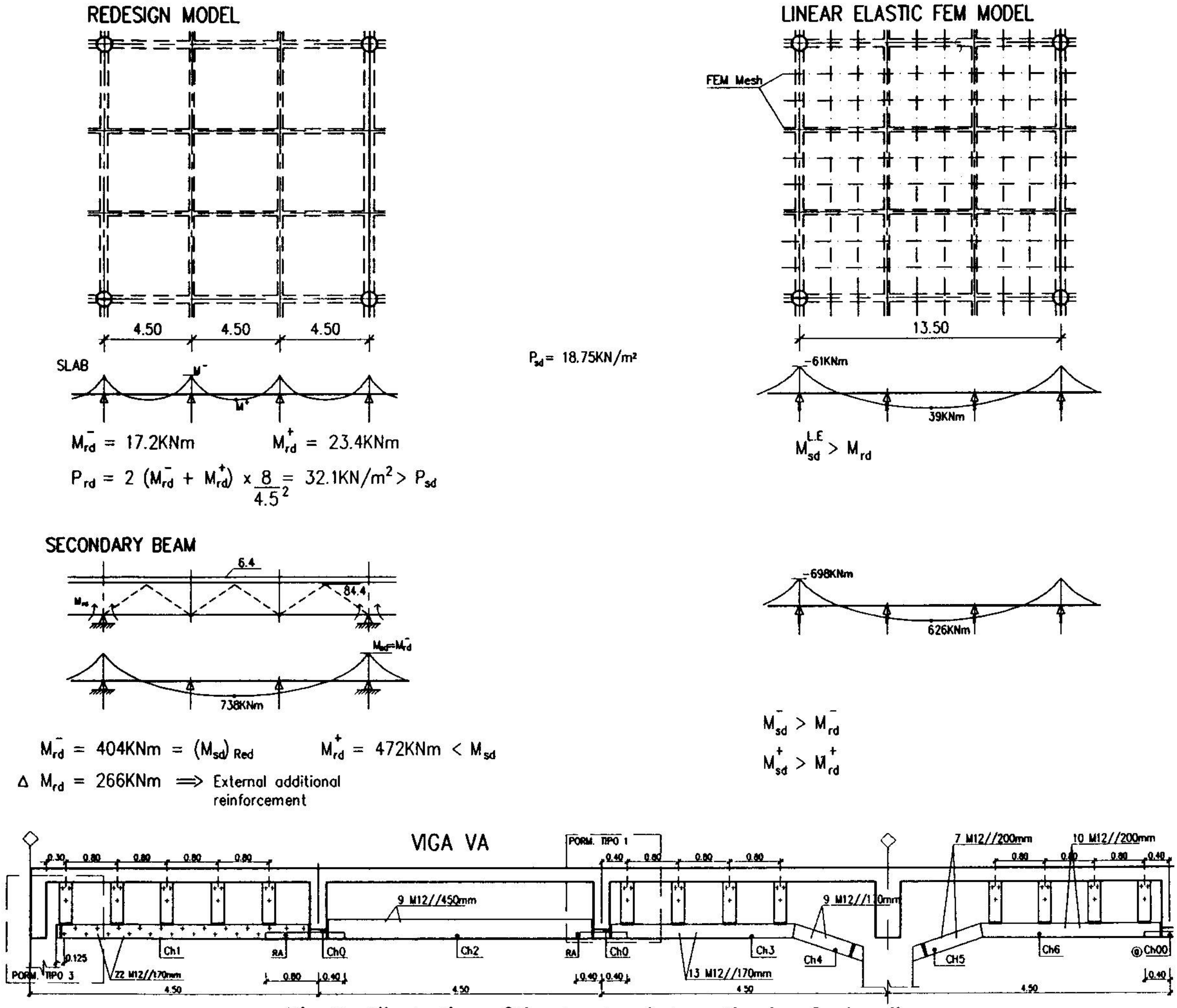

Fig. 2 - Illustration of the structural strengthening for bending

Small diameter tubes are left at chosen locations around the plate boundaries. The injection is made through the lower tubes, which work as injection ports, as the enclosed air escapes through the higher tubes, which work as vents. As soon as the resin shows in the vents flowing continuously without air bubbles, the vents are blocked to allow for some pressure build up in the liquid resin. This facilitates the penetration of resin into the small cavities and cracks in the concrete surface, allowing for a good bond.
The injected resin also penetrates at least in the outer portion of the annular space around the steel anchor bolts, enhancing its slip resistance and allowing for immediate mobilization of the plate reinforcement when the beams are loaded. Thickness of adhesive film is kept to a minimum.

A redundant shear connection is thus accomplished between steel plate and concrete: by mechanical grip (anchor bolts) and by adhesion (epoxy bonding agent).

Fire resistance is increased, as the mechanical connection acts as a back up which is not easily affected in the event of a fire.

After hardening of the resin the plastic tubes are broken off and the plates coated with the fire resistant paint for additional protection.

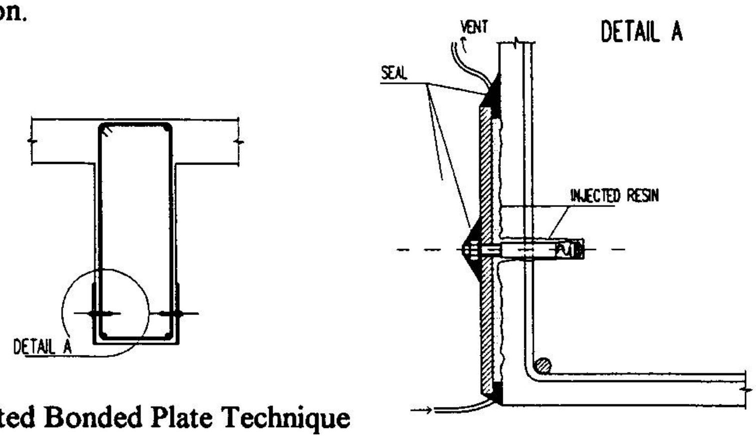

Fig. 3 - Injected Bonded Plate Technique

The low viscosity injected epoxy resin STAPOX IJ was developed in a cooperative program between STAP and LNEC. Its main mean characteristics are the following:

Yield stress in tension - 53 MPa ; Modulus of elasticity - 3570 MPa
Elongation at break - 2.0%; Yield stress in compression - 112 MPa

5. JOB PLANNING, PRODUCTIVITY AND QUALITY CONTROL

One very important constraint was imposed by the Owner on the strengthening project, as the Post Office station was to be kept in operation during the execution of the project.

Sorting machines and other postal processing equipment difficult to remove had to be protected in order to avoid damage. Work areas were sequentially made available, in accordance with the operational needs of the Owner.

Utilities had to be temporarily removed or displaced in each work area, to allow for access to the beams to be strengthened. Some cumbersome utilities as ventilation ducts difficult to remove were only loosened and lowered to allow access to reinforced concrete members.

A total of around 30 000 man hours were spent on the first phase, with the following distribution:

Task	Man hours	Productivity (h/m^2)
Concrete surface preparation	5 100	3.4
Steel plate manufacture and installation, incl. welding	15 400	10.2
Plate sealing	5 700	3.8
Resin injection	2 000	1.3
Other	1 600	1.1
Total	29 800	19.8

The 1 500 m^2 of steel plate reinforcement were completed in a delay of 5 months.

A quality control system was put in practice, involving a number of laboratory and "in situ" tests, in order to meet the high standards required by the Post Office and to ensure the reliability of the strengthening work.

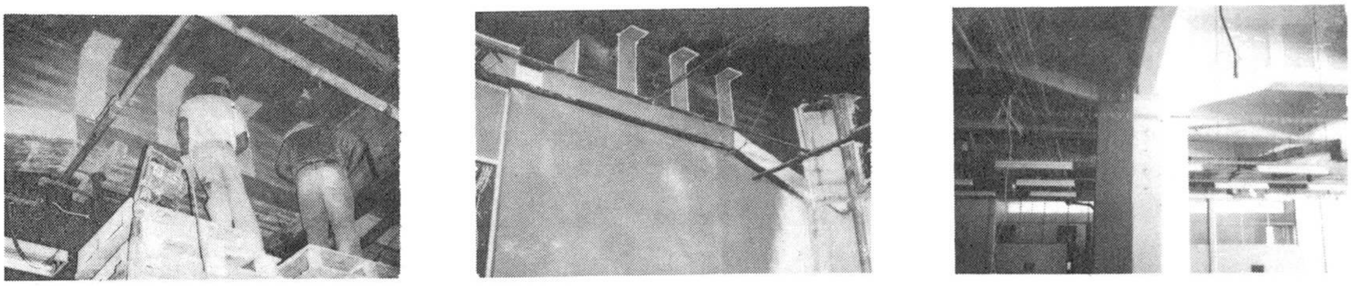

Fig. 4 - Strengthening by plate bonding - details of execution

The laboratory tests on the resin batches are standardized and don't deserve any particular mention. For the site quality control, two types of resin samples were routinely collected for testing:

a) 31 mm diameter cylinders (9 cylinders for each beam).

b) 220 x 220 x 4 (mm) plates (3 for each 200 l drum of epoxy resin).

c) Specimens consisting of three steel plates bonded by injected epoxy resin in two contact areas of 50 x 100 mm², each with a 2 mm epoxy film.

The cylinders allowed for immediate control of resin set time. After setting, its hardening was also controlled using the Barcol hardness instrument. Finally the cylinders were subject to a regular compression test, up to failure.

As for the 4 mm plates, they were used to cut out resin specimens for tensile tests, also up to failure.

The yield shear bond stress between the steel plates and the resin was 4.05 MPa (average).

In order to assess the results of the strengthening work, two load tests were also carried out on the same panel of the concrete floor, before and after the strengthening. A load of 3.5 kN/m² was applied, first in a central panel of 4.5 x 4.5 m², then over one of the main concrete beams, in a 4.5 m strip over its whole length. A reduction of the beam deflections was recorded as shown in Fig. 5.

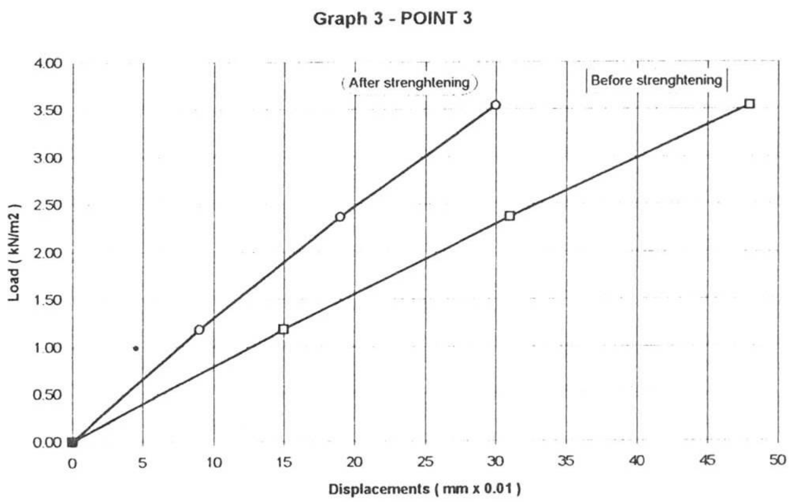

Fig. 5 - Load Test - Before and after strengthening