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Durability of Masonry Strengthening by Injection Techniques

Durabilité de maçonneries consolidées par des injections

Dauerhaftigkeit von durch Injektionen versteiftem Mauerwerk

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SUMMARY

Brick-masonry prisms, built with different types of mortar, were subjected to compression tests up to failure. The cracked prisms were injected with epoxy resin or cement-polymer grout according to a widely used repair technique. The mechanical improvement from the cracked situation, the penetration and diffusion of grout and the durability with respect to salt crystallization and thermal cycles was examined by means of destructive and non-destructive tests.

RÉSUMÉ

Des prismes de maçonnerie composés de briques liés avec différents mortiers ont été soumis à des essais de compression jusqu'à la ruine. Les prismes fissurés ont été injectés avec des résines époxydes ou des coulis de ciment modifiés avec des polymères. L'amélioration mécanique obtenue après l'injection, la pénétration et la diffusion du coulis ainsi que la durabilité à la cristallisation des sels et aux cycles thermiques ont été vérifiés par des essais destructifs, et non destructifs.

ZUSAMMENFASSUNG

Prismen von einem Ziegelmauerwerk, das mit verschiedenen Mörtelarten aufgebaut worden war, wurden den Druckprüfungen bis zum Bruch unterzogen. Die beschädigten Prismen wurden entsprechend einer sehr verbreiteten Reparaturtechnik entweder mit Epoxy-Harz oder mit Polymer-Zement eingespritzt. Zerstörende und zerstörungsfreie Versuche wurden angewandt, um die Verbesserung der mechanischen Eigenschaften, die Durchdringung und Ausbreitung der injizierten Stoffe und den Widerstand gegen Salzkristallbildung zu untersuchen und zu bestimmen.



1. INTRODUCTION

Grouting by injection of resins and cement-polymer grouts is a strengthening and repair technique widely used for masonry in Italy and in most European countries.

A systematic experimental approach is needed to study the effectiveness of grouting for what concerns: the improvement of masonry strength, the degree of penetration and diffusion of the injected material and the durability of repairs to external agents such as temperature variation, frost-defrost action, etc. Furthermore a study of the interface between the grout and the existing materials is necessary in order to detect their bond strength.

Brick-masonry prisms were built with different types of mortar; some of them were submitted to a salt crystallization procedure set up by the authors which lead their external surfaces to decay. The strength and deformability of the prisms were evaluated by compression tests; care was taken in order to avoid a total destruction of the specimens. The damaged prisms were successively strengthened by injection of epoxy resin or cement-polymer grout and subjected again to the compression test.

Some of the repaired prisms were instead submitted to the crystallization test in order to detect the durability of grouting when masonries are exposed to aggressive environments. The influence of temperature variation on the mechanical characteristics (compressive strength, deformability, bond-strength) of small specimens repaired by epoxy resin and subjected to thermal cycles was also measured.

When dealing with in service masonry structures, a way to detect the actual conditions of the materials and to evaluate the efficacy of the injection technique could be drilling of cores and inner inspection. However this checking is of destructive nature with significant sampling difficulties. Ultrasonic pulse transmission tests were used as a mean to check penetration and diffusion of grout and measure the degree of strengthening in masonries repaired by injection. The aim was to verify the effectiveness of this non destructive technique. The results obtained were controlled by cutting some prisms into slices and by verifying the quantity and position of voids after injection.

2. EXPERIMENTAL AND GROUTING PROCEDURE

Twenty five prisms (25x52x60 cm) were prepared with one type of solid brick and three types of mortar: pozzolana-lime (M1), cement-lime (M2) and high strength cement modified with acrylic resins (M3). Detailed information on the mortar composition and on brick properties are given in [1]. Some of the prisms were then subjected to a crystallization test in order to simulate the masonry decay due to external aggressive environments [2]. The deteriorated prisms made with mortar M1, M2, M3 were respectively named MU1Ti, MU2Ti, MU3Ti where $i = 1, \dots, n1$ ($n2, n3$), represents the number of the prism. The reinig undamaged prisms were named MU1i, MU2i, MU3i (where $i = 1, \dots, n1$ ($n2, n3$)).

All the prisms were subjected to compression tests. The tests were performed using an MTS hydraulic servocontrolled machine at a constant rate of displacements of 0,00095 m/s. The testing recommendations [3] were adopted. The prisms which had not reached a total collapse were then repaired by injection, some with an epoxy resin, some with a grout composed of cement and a 10% of the same epoxy resin. The resin was a two component one, mixed before injection. Its pot life is 60 to 40 min, the setting time 6 to 4h between 15 and 22°C, the compressive strength 110 MPa, the tensile strength 70 MPa and the elastic modulus 3,000 MPa. The injections were carried out after the external surfaces of the cracked prisms were impermeabilized with a thick epoxy paint. Details of the operation are described in [4]. After injection and appropriate curing, the prisms were again subjected to the compression tests above mentioned.

Three prisms repaired with epoxy resin were cut into slices, in order to check the degree of diffusion and penetration of the resin.

Three prisms injected with epoxy resin were subjected to the crystallization test described in [5] in order to study the behaviour of the repaired masonry to aggressive environment. At every cycle the specimens, after a 4h immersion in a saturated solution of Na_2SO_4 , were exposed for 7 days at 20°C and 50% R.H.

Small stack-bond prisms repaired with epoxy resin were subjected to thermal cycles in order to detect the effect of the temperature variation on the strength and deformability of masonry injected with polymeric materials (8h at 60°C and 70% R.H., 16h at 20°C and 50% R.H. or 16h at -15°C and 8h at 20°C and 50% R.H.); a detailed description of the test and results is given in [5].

During every phase of the experimental work (before the compression test, after the compression test, after injection and finally after testing again the prisms up to collapse) some of the prisms were controlled by ultrasonic pulse transmission tests. Two 90 kHz-wave transducers were used and the pulse transmission data (velocity and waveform) were monitored along horizontal path both in longitudinal and transverse directions. More detailed information are found in [1].

3. RESULTS AND DISCUSSION

The results of the different experimental procedures adopted to detect the effectiveness and durability of repair by injection technique are reported in the following.

3.1 Mechanical tests

Detailed description of the compression tests carried on MU1i, MU2i, MU3i and MU1Ti, MU2Ti, MU3Ti prisms are reported in [1], [5] where tables and figures are used also to show the correlation of the compressive strength of injected prisms, to the quantity of injected resin or cement-polymer grout and to the number of injections. Apparently no correlation exists between the compressive strength and the injection pressure, which for some prisms reached 0.55 MPa, but which is recommended to be taken as low as possible (in some recommendations no more than 0.05 MPa).

In fig. 1 an example is given of the most representative stress-strain curves of prisms after injection with epoxy resin together with their original curves. The following comments can be made:

- The strength $\bar{\sigma}_r$ (corresponding to the maximum load carrying capacity) of the injected prisms is always higher than the value σ^* at which the previous compression test was stopped (Fig.1).
- The strength $\bar{\sigma}_r$ of the injected prisms reaches an average of 85% of the strength σ_r originally attained by prisms.
- The stress-strain curves of the injected prisms tend to remain similar to those of the prisms under the first compression test (Fig.1). In other words the injections of epoxy resin apparently do not give a totally new situation in terms of deformability; in fact the influence of the mortar used for the masonry is definitely prevailing even after injection. Nevertheless a tendency to an increasing in brittleness is detected.

The same experimental investigation was carried on four prisms injected with cement-polymer grout. At presently only some comments on the first results can be made:

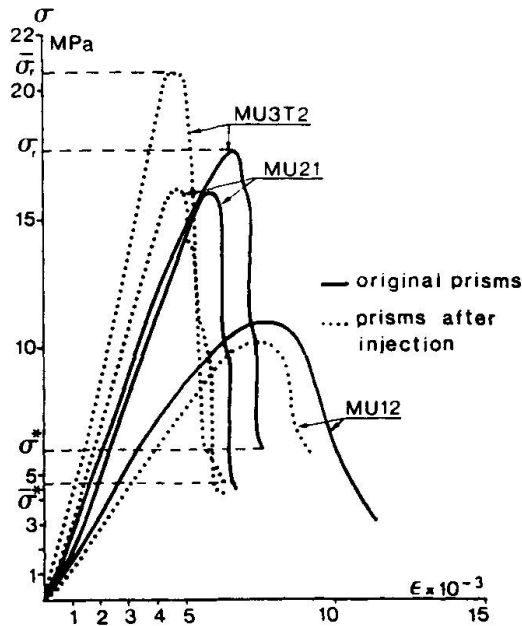


Fig. 1 Stress-strain curves for prisms injected with resin

- The strength $\bar{\sigma}_r$ of the injected prisms is still higher than the value σ^* at which the previous compression test was stopped, but it does not attain more than the 40% of the value σ_r originally reached by the prisms.
- The slope of the $\sigma - \epsilon$ curves of the injected prisms tends to be lower than the one given during the first compression test. Nevertheless after the value $\bar{\sigma}_r$ is reached a tendency is shown to a reduction in brittleness, if comparison is made to the original situation.

3.2 Ultrasonic pulse transmission tests

Based on all ultrasonic velocity measures obtained on the prisms at the several phases considered (undamaged, damaged, injected, collapsed), a rough guide for the evaluation of the masonry condition can be stated, slightly different from that one proposed by other authors [6]:

- velocities of less than 1400 m/s indicate highly cracked masonry with very poor mechanical properties (less than 3,5 MPa compressive strength);
- velocities between 1400 and 2400 m/s

indicate a lower quantity of cracks and an appreciable compressive strength (between 3,5 and 10,5 MPa);

- velocities over 2400 m/s indicate masonry in a good normal condition. The average values measured at the various phases of the experimental work are reported in Fig.2 for all the masonry prisms: the positive grouting effects are clearly visible, particularly for the prisms injected with epoxy resin. The high value of velocity measured on MUjTi prisms is due to their content in sodium sulphate. Moreover a relationship, even if quite rough, has been noticed between

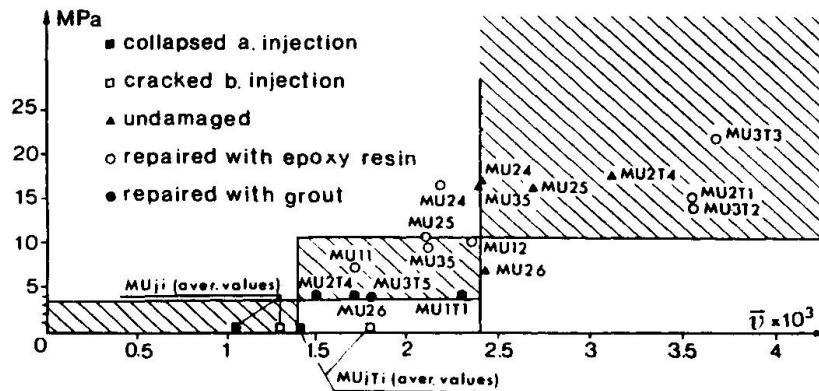


Fig. 2 Rough guide for quality masonry

velocities and strength. The ratio between the masonry strengths in the undamaged condition and after injection (0.85 and 0.40 respectively for epoxy resin and resin modified mortar grouting) was found similar to the ratio between

the relative average velocities. If confirmed by further experimental data, the pulse velocities could then be considered as reliable to define the global change of the material characteristics after repair by injection. Finally, taking into account also the waveforms of the ultrasonic pulses and comparing their amplitudes and frequencies in the different phases considered, more detailed local information on the grouting penetration and diffusion have been obtained. In fact in some cases locally increased velocities did not correspond, in the waveform, to increased values of amplitude and to a general increase in the signal components with higher frequency. Thanks to inner observation (see Sec. 3.3) this situation could easily be connected with the presence of cracks and voids that the injected material has only partially filled.

3.3 Penetration and diffusion of the grout

In order to control the information given by the sonic tests and to know the local state of penetration and diffusion of the injected grouts, the three prisms, one for every type of mortar, cut into slices were carefully investigated. A map of the crack distribution was drawn; in several cases a high percentage of voids was detected.

The inspection of the cut surfaces and of the failure mode of the prisms shows that the external crack distribution does very seldom correspond to the internal one so that external cracks do not connected with internal.

Two different situations were observed:

a) Injection of epoxy resin:

the resin filled better wide open cracks ($5 \div 7$ mm) than narrow cracks ($0.2 \div 2.5$ mm) and cracks in mortar joints. In several cases the resin was absorbed by the brick (5 to 6 mm of impregnation) while the cracks remained empty (Fig.3). This impregnation, beside changing the colour of the injected brick (Fig.4), decreases the brick porosity and increases its mechanical strength.

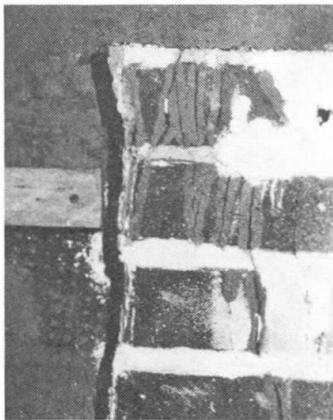


Fig. 3 Penetration of salts inside cracks and voids

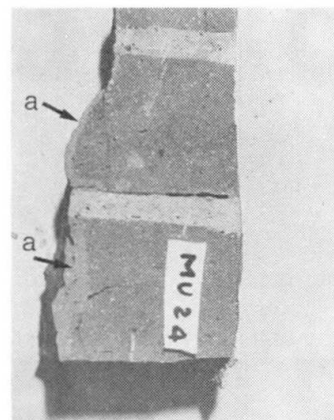


Fig. 4 Fracture along the boundary of the impregnated zone (a)

b) Injection of cement-polymer grout:

the grout had better filled the cracks than the resin, but shrinkage phenomena took place in various cases and also the formation of bubbles was observed when air was trapped inside cracks [5].

3.4 Durability tests

Three prisms, one for each type of mortar (M1, M2, M3), were subjected to the described crystallization test for 16 cycles.

The sodium sulphate was transferred and deposited by capillary rise inside voids and unfilled cracks and, sometime, caused a propagation of cracks. Injections cannot prevent the penetration and diffusion of salt solution inside the masonry if they do not fill all voids (Fig.3).



Small specimens (3 or 4 brick stack-bond prisms) repaired with epoxy resin were subjected to thermal cycles. The mechanical tests carried on the prisms after 63 cycles allowed to state that temperature variations influence the behaviour of masonries repaired with resin: freeze tends to increase the material stiffness and the brittleness; while thaw, decreases its stiffness and strength [5].

CONCLUSIONS

Up to now, some conclusions can be drawn concerning particularly the repairs made with epoxy resin (which on the other hand are not so frequently used in practical cases without a filler).

An improvement of the mechanical strength of cracked masonry is the consequence of the injection of resin. Resin does not influence the stiffness of the masonry apart from a tendency to a more evident brittleness in the case when the brittle behaviour is typical of the original masonry.

Uniform penetration and diffusion of resin into cracks and voids is seldom realized if external cracks do not connect with internal ones. Water and salts can easily penetrate into small unfilled cracks and deteriorate the material.

Temperature variations can influence the behaviour of the repaired masonry and produce internal states of stresses caused by the variation of properties of the resin thermal cycles.

Based on the results obtained in this experimental work the technique of the ultrasonic pulse velocities measurements can be a practical tool in the assessment of the masonry conditions and a reliable way in the evaluation of the grouting efficiency. The waveform analysis of the ultrasonic pulse can give useful information on the grout penetration and diffusion into the masonry, as increased velocity alone can not assure a complete filling of voids and cracks. However a large presence of such discontinuities generally represents an obstacle in the interpretation of ultrasonic measurements.

The next step of the research, which is partly supported by a EEC Stimula Research Contract, will be to study the behaviour of prisms injected with other types of epoxy resin, the variation of bond strength when the technique is applied in the presence of moisture inside masonry and to continue the research on the effectiveness of cement-polymer grouts.

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Ausbesserung beschädigter Oberflächen von Betonkonstruktionen
Repair of Deteriorated Surfaces of Concrete Members
Réparation de surfaces détériorées de constructions en béton

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ZUSAMMENFASSUNG

Eine dauerhafte Instandsetzung beschädigter Betonoberflächen ist u.a. abhängig von den Eigenschaften der Reparaturwerkstoffe und von den Verfahrenstechniken. In diesem Beitrag werden die Ergebnisse einer Reihe von Untersuchungen zu diesem Thema vorgestellt. Anforderungen an Reparatursysteme werden definiert und der Einfluss unterschiedlicher Umweltbedingungen auf die Verbundfestigkeit wird aufgezeigt. Mit der Realkalisierung karbonatisierter Betonrandzonen durch Auftrag eines hydraulischen Reparatursystems wird eine wirksame Möglichkeit zur Wiederherstellung des Korrosionsschutzes der Bewehrung im Beton aufgezeigt.

SUMMARY

A durable repair of deteriorated concrete surfaces depends on the properties of the repair materials and the techniques of application. In this paper the results of a number of investigations related to these problems are presented. Requirements to be satisfied by the repair systems are defined and the influence of local conditions on the bond strength is shown. It is shown that realkalinization of carbonated concrete sections can be achieved by means of a cement based repair system applied to the concrete surface. This is an effective measure to reestablish corrosion protection of the reinforcement.

RÉSUMÉ

Une réparation durable de surfaces détériorées de constructions en béton dépend des matériaux et des techniques de réparation. Dans cet article, les résultats d'études à ce sujet sont présentés. Les propriétés qui sont exigées des systèmes de réparation sont définies et l'influence des conditions locales sur l'adhérence est mise en évidence. La réalkalisation des zones extrêmes du béton par l'application d'un système de réparation hydraulique présente une possibilité efficace pour rétablir la protection contre la corrosion de l'armature dans le béton.



1. PROBLEMSTELLUNG

Betonkonstruktionen sind häufig Umweltbedingungen ausgesetzt, die zur Korrosion des Bewehrungsstahles und des Betons führen. Solchermaßen geschädigte Betonbereiche müssen durch geeignete Reparaturmaßnahmen instandgesetzt werden.

Als Reparaturwerkstoffe werden organische, rein hydraulische oder organisch modifizierte hydraulische Werkstoffe eingesetzt. Im allgemeinen wird bei solchen Maßnahmen zwischen Altbeton und Reparaturmörtel eine organische oder hydraulische Haftbrücke aufgebracht. Bei großflächigen Instandsetzungsmaßnahmen werden meist hydraulische Reparatursysteme verwendet. Neben Kostenvorteilen besitzen hydraulische Systeme im Vergleich zu organischen Systemen eine größere mechanische und physikalische Ähnlichkeit mit dem Altbeton.

Um eine dauerhafte Instandsetzungsmaßnahme mit hydraulischen Reparatursystemen zu gewährleisten, müssen bestimmte Randbedingungen erfüllt sein. Hierzu erforderliche Materialkenngrößen der Reparaturwerkstoffe, die Auswahl geeigneter Haftbrücken und zweckmäßige Arbeitsverfahren wurden in theoretischen und experimentellen Arbeiten untersucht. Im folgenden werden die Ergebnisse dieser Untersuchungen zusammengefaßt [1, 2, 3].

2. ANFORDERUNGEN AN DIE REPARATURWERKSTOFFE

Reparaturschichten auf Betonbauteilen werden aus der Umwelt, dem Betonuntergrund und der Reparaturschicht selbst beansprucht. Dabei lassen sich Beanspruchungen unterscheiden, die zu Spannungen und Verformungen führen und solche, die die Eigenschaften des Reparatursystems verändern oder korrosiv wirken. Zu den letztgenannten zählen insbesondere die Karbonatisierung der hydraulischen Reparaturschicht mit der damit verbundenen Verminderung des Korrosionsschutzes der Bewehrung, Feuchte- und Frosttausalzeinflüsse sowie der Verschleiß infolge Verkehrsbeanspruchung. Anforderungen an Reparaturwerkstoffe beziehen sich demzufolge auf Karbonatisierungswiderstand, Witterungsbeständigkeit und Verschleißfestigkeit.

Als wesentliche mechanische Beanspruchung des Systems bestehend aus Reparaturschicht und Altbeton stellen sich jene Lastfälle heraus, die Zugspannungen in Reparaturschichten hervorrufen, insbesondere Schwinden der Reparaturschicht und schockartige Abkühlung des Bauteils, z.B. durch einen Gewitterregen auf eine erhitzte Oberfläche (s. Fig. 1).

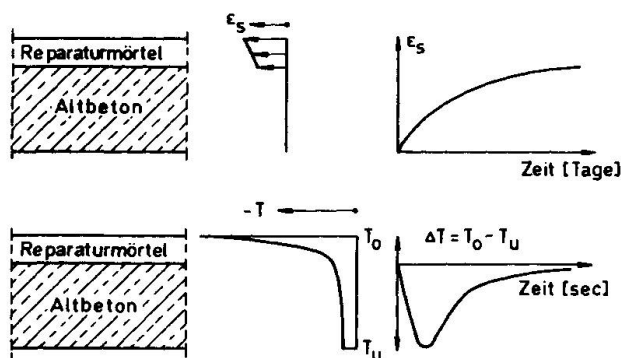
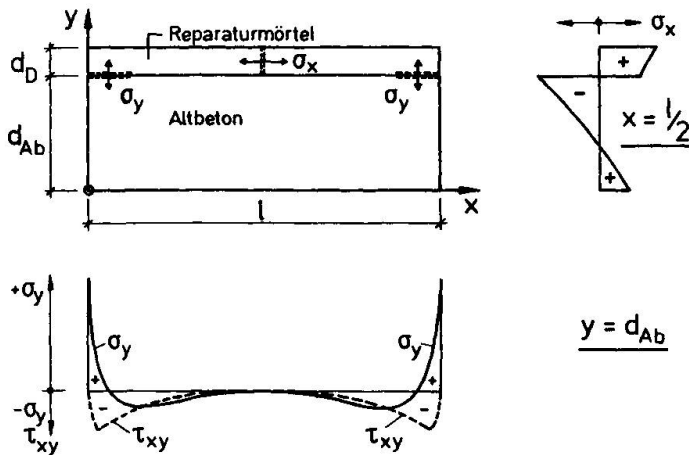


Fig. 1 Beanspruchung von Reparaturschichten

Diese Lastfälle wurden mit Hilfe der FEM unter Berücksichtigung wirklichkeitsnaher Stoffgesetze für Reparaturschicht und Unterbeton analysiert. Infolge der behinderten Verformung kommt es zu horizontalen Zugspannungen in der Reparaturschicht, die zu vertikalen Rissen führen können. In der Verbundzone treten neben horizontalen Schubspannungen vertikale Spannungskomponenten auf, die an freien Rändern als Zugspannungen in jedem Fall die Zugfestigkeit der angrenzenden Schichten oder

die Adhäsionsfestigkeit überschreiten und so zu horizontalen Ablösungen führen (Fig. 2). Daß es nicht zu einem Versagen der Reparaturmaßnahme kommt, liegt am spezifischen Verformungsverhalten zementgebundener Werkstoffe. Mit zunehmender



Dehnung können über die Zugbruchdehnung hinaus noch Zugspannungen aufgenommen werden [4]. Es kommt demnach nicht zu einem reißverschlußartigen Ablösen größerer Bereiche, sondern zur Ausbildung sogenannter geschwächter Zonen über eine Länge von einigen Zentimetern an den freien Rändern der Systeme und zwar normalerweise im weniger festen Unterbeton (Fig. 3). Solchermaßen geschwächte Ränder konnten in experimentellen Untersuchungen nachgewiesen werden.

Fig. 2 Spannungen in Reparaturschichten

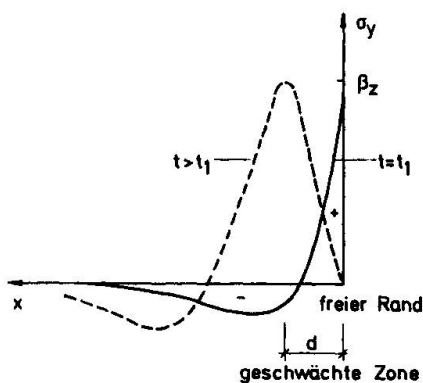


Fig. 3 Geschwächte Zone im Randbereich von Reparaturschichten

Schwinden ($t = 90$ Tage)	$\epsilon_s < 0,8 \text{ ‰}$
Therm. Dehnung	$\alpha_{tRep} = \alpha_{tBeton}$
E-Modul	$E_{Rep} < E_{Beton}$
Kriech- und Bruchdehnung	ϵ_k, ϵ_u mgl. groß
Verbundfestigkeit	mgl. groß $\beta_{HZ} > 2,0 \text{ N/mm}^2$
Reparaturschichtdicke	$d_{Rep} \geq 10 \text{ mm}$

Tab. 1: Anforderungen an Reparaturwerkstoffe

In Tab. 1 sind als Ergebnis der Spannungsanalyse Anforderungen an Eigenschaften von Reparaturwerkstoffen zusammengefaßt, die für eine dauerhafte Instandsetzungsmaßnahme eingehalten werden sollten.

Neben den Anforderungen aus korrosiven und mechanischen Beanspruchungen muß die Reparaturschicht dauerhaft gewährleisten, daß keine schädlichen Substanzen, und hier in erster Linie Chloride, an den Bewehrungsstahl vordringen und dort zu Korrosion führen können.

3. PRÜFVERFAHREN ZUR BEURTEILUNG DES VERBUNDES

Um die Wirksamkeit und Dauerhaftigkeit des Verbundes zwischen Reparatursystem und Altbeton zu bestimmen, bedient man sich unterschiedlicher Prüfverfahren, die allgemein den Nachteil haben, daß der Bruch selten in der Verbundebene erfolgt und die Streuung der Ergebnisse groß ist.

Aufschluß über die Zugfestigkeit der Verbundzone kann der modifizierte Spaltzugversuch geben. Dabei wird ein Bohrkern, bestehend aus Unterbeton und Reparaturschicht in Höhe der Verbundzone über Lastmanschetten so beansprucht, daß Zugspannungen senkrecht zur Verbundzone zum Bruch der Probe in der Verbundzone führen (Fig. 4).

Der Spannungszustand dieses Versuchsaufbaues wurde mit der FEM analysiert und



es konnte eine Beziehung für die mod. Spaltzugfestigkeit hergeleitet werden:

$$\beta_{MS} = K \cdot F_U / (\pi \cdot d^2/4)$$

wobei: F_U = Bruchlast

d = Probendurchmesser

K = Beiwert nach Abb. 5

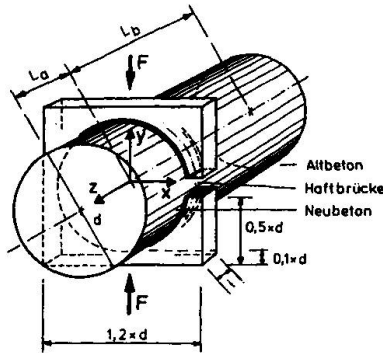


Fig. 4 Der modifizierte Spaltzugversuch

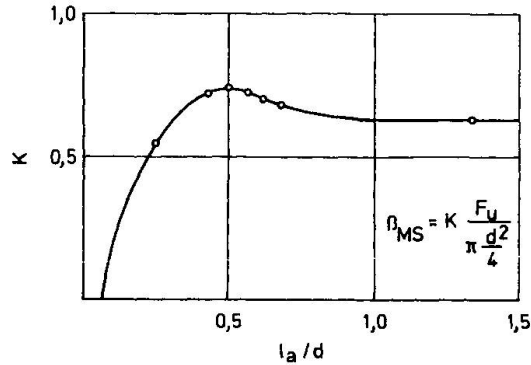


Fig. 5 Die modifizierte Spaltzugfestigkeit

4. EXPERIMENTELLE UNTERSUCHUNGEN AN REPARATURSYSTEMEN

In experimentellen Untersuchungen wurde der Einfluß unterschiedlicher Lagerungsbedingungen auf die Verbundfestigkeit hydraulischer Reparatursysteme und dabei insbesondere der Einfluß unterschiedlicher Haftbrücken untersucht [1, 2].

Im Rahmen von Laboruntersuchungen wurden hydraulische Reparatursysteme auf vorbereitete Betonplatten 300x300x100 mm aufgebracht. Als Reparatursysteme wurden rein hydraulische Mörtel mit Zementleimhaftbrücken (CC), thermoplastmodifizierte Mörtel mit thermoplastmodifizierten Zementleimhaftbrücken (TCC) und epoxidharzmodifizierte Mörtel mit Epoxidharzhaftbrücken (ECC) angewandt. Die Lagerung der Probekörper erfolgte bei 20 °C/65 % r.F. unter Wasser oder bei Frosttausalzbeanspruchung. Die Wirkung von Gewitterregen auf erhitze Betonoberflächen wurde durch Temperaturwechsellagerung simuliert. Zur Bestimmung der Verbundfestigkeit wurden u.a. modifizierte Spaltzugversuche durchgeführt. Die Ergebnisse sind auszugsweise in Fig. 6 dargestellt.

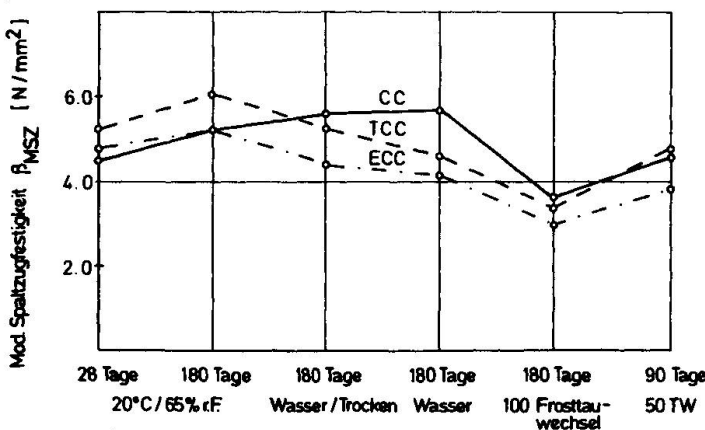


Fig. 6 Verbundfestigkeit von Reparatursystemen bei unterschiedlicher Lagerung

Die rein zementgebundenen Systeme (CC) erreichen hohe Verbundfestigkeiten und stellen sich als ausgesprochen feuchtigkeitsstabil dar. Durch die Zugabe von Kunststoffen kann die Verbundfestigkeit im Trockenzustand gesteigert werden, Feuchteinflüsse können jedoch zum Quellen und Redispersieren von Kunststoffpartikeln führen. Dagegen stellen Verseifungsreaktionen bei den meisten heute angewandten Modifizierungsmitteln kein Gefährdungspotential mehr dar. Die Frosttauschädigung des Verbundes wird hauptsächlich durch

in die Verbundzone eingedrungenes und dort gefrierendes Wasser ausgelöst. Damit ist der Frosttauwiderstand des Verbundes abhängig vom Frosttauwiderstand der Mörtel und evtl. Vorschädigungen durch die Lastfälle Schwinden, Temperatur oder Verkehrslast.

Im Rahmen von Feldversuchen wurden hydraulische Reparatursystem mit unterschiedlichen Haftbrücken zur Instandsetzung frosttausalzgeschädigter Betonfahrbahnplatten auf einer Bundesautobahn eingesetzt. Nach einer Oberflächenvorbereitung wurden rein hydraulische, organisch modifizierte hydraulische Haftbrücken sowie eine Epoxidharzhaftbrücke zusammen mit einem rein hydraulischen Ausbesserungsbeton ($w/z = 0,4$, Größtkorn 8 mm) aufgebracht. Im Alter von 7, 180, 900 und 1200 Tagen erfolgten u.a. modifizierte Spaltzugprüfungen an Bohrkernen im Labor. Die Ergebnisse sind auszugsweise in Fig. 7 dargestellt.

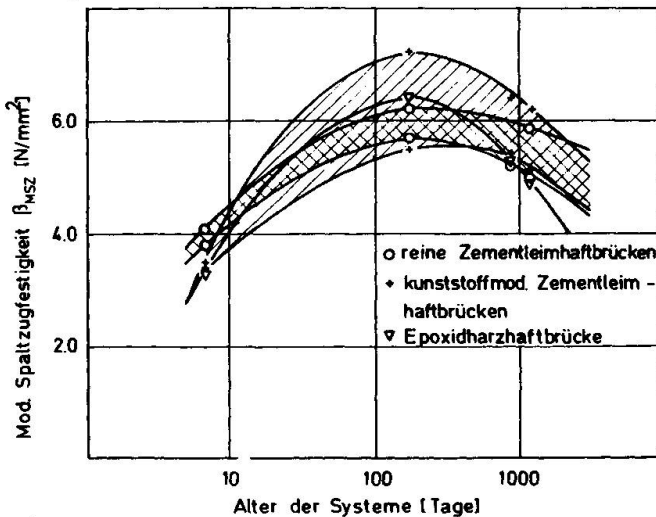


Fig. 7 Verbundfestigkeit der Reparaturssysteme in Abhängigkeit vom Alter

Die von den untersuchten Reparatursystemen erreichten Verbundfestigkeiten liegen weit über den Anforderungen an die Systeme. Es zeigte sich, daß rein hydraulische Systeme in der Lage sind, einen dauerhaften Verbund zu gewährleisten. Der vielfach beschriebene Vorteil der kunststoffhaltigen Haftbrücken konnte in Bezug auf die Dauerhaftigkeit nicht nachgewiesen werden. Von besonderer Bedeutung für die Dauerhaftigkeit der Reparaturmaßnahme ist die Qualität der Durchführung.

5. KORROSIONSSCHUTZ DER BEWEHRUNG DURCH REALKALISIERUNG KARBONATISierter BETONRANDZONEN

Die Wiederherstellung des Korrosionsschutzes der Bewehrung erfolgt in der Praxis meist durch Entfernung des geschädigten, karbonatisierten oder verseuchten Betons, vollständiges Entrosten des Stahles, Anstrich des Stahles und Wiederherstellung des alkalischen Milieus in der Umgebung der Bewehrung durch ein zementgebundenes Reparatursystem. Dieses Verfahren ist insbesondere bei großflächigen Schäden sehr aufwendig. Bei einer Beschichtung der Stähle kann es zudem zu einer Makroelementbildung aufgrund unterschiedlicher Belüftung und zu erneuter, verstärkter Korrosion kommen [5].

Ist der Korrosionsschutz des Bewehrungsstahles durch Karbonatisierung des Betons verlorengegangen, aber noch keine Schädigung des Betons aufgetreten, können durch großflächigen Auftrag hydraulischer Reparatursysteme in ausreichender Dicke die karbonatisierten Betonrandzonen realkalisiert und eine weitergehende Korrosion verhindert werden, wenn keine Chloridverseuchung des Betons vorliegt.

In experimentellen Untersuchungen wurden hydraulische Reparatursysteme auf karbonatisierte Oberflächen unterschiedlicher Betone aufgetragen. Nach 14, 28 und 90 Tagen wurde die Realkalisierungstiefe mit Phenolphthalein als Indikator gemessen. In Fig. 8 sind die Ergebnisse auszugsweise dargestellt.

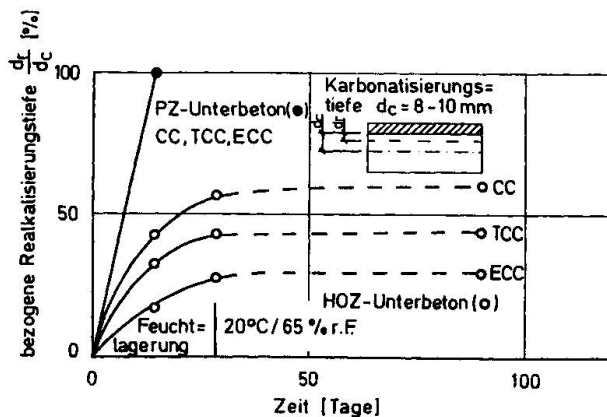


Fig. 8 Realkalisierungsfortschritt für unterschiedliche Reparatursysteme, Unterbetone und Lagerungsbedingungen

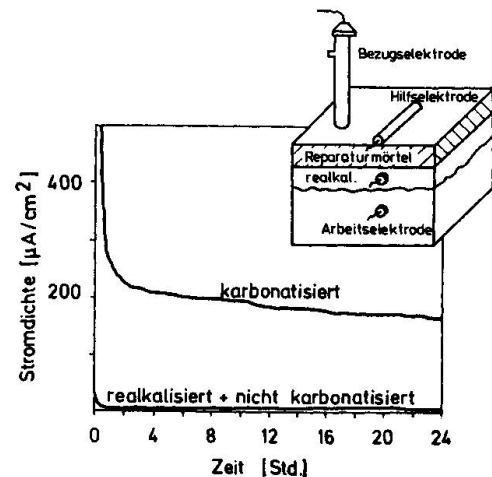


Fig. 9 Potentiostatischer Halteversuch an Stählen im karbonatisierten, nicht karbonatisierten und realkalisierten Beton

Es zeigt sich, daß durch eine Feuchtlagerung die Realkalisierung beschleunigt wird, aber daneben von der Art des Unterbetons und von diffusionsbehindernden Kunststoffbestandteilen der Haftbrücke oder der Reparaturmörtel abhängt.

Um zu überprüfen, ob mit der Realkalisierung karbonatisierter Betonrandzonen auch eine Repassivierung von Stählen verbunden ist, die im karbonatisierten Beton eingebettet sind, wurden in [3] potentiostatische Halteversuche in Anlehnung an die Prüfung von Betonzusatzmitteln auf stahlaggressive Substanzen durchgeführt (s. Fig. 9). Für den Stahl im karbonatisierten Bereich wurde nach 24 Std. eine Stromdichte von ca. 150 $\mu\text{A}/\text{cm}^2$ beobachtet; der Stahl korrodierte. Die Stromdichte-Zeitfunktionen für Stähle im nicht karbonatisierten bzw. realkalisierten Bereich zeigten keinen signifikanten Unterschied. Die Realkalisierung hat damit eine Repassivierung eingebetteter Stähle zur Folge.

Vor einem verbreiteten Einsatz dieses Verfahrens sind jedoch noch eine Reihe von baupraktischen Randbedingungen abzuklären und hierbei insbesondere Fragen der Verfahrenstechnik und erforderliche Nachbehandlungsdauern zu lösen. Die Probleme werden z.Z. im Rahmen eines Forschungsvorhabens bearbeitet.

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Renforcements de structures par la technique des tôles collées
Verstärkung von Tragwerken durch geklebte Stahllamellen
Checking Bonded-Plate Structural Reinforced Reinforcements

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RÉSUMÉ

Une méthode de renforcement des structures en béton a connu depuis près de vingt-cinq ans un développement important. Il s'agit de la technique des tôles collées qui consiste à pallier le manque d'armatures passives par des tôles collées en surface du béton. Bien que s'apparentant au béton armé, le renforcement des structures par tôles collées présente des différences importantes de comportement. Cet article présente les résultats des études sur trois aspects particuliers et les recommandations qui en ont été déduites pour la vérification des renforcements réalisés par cette technique.

ZUSAMMENFASSUNG

Eine Verstärkungsmethode für Betonbauelemente hat seit fast zwanzig Jahren erheblich an Bedeutung gewonnen. Es handelt sich dabei um das auf geklebten Stahllamellen beruhende Verfahren, bei dem ein Mangel an schlaffer Bewehrung durch oberflächlich auf den Beton geklebte Bleche ausgeglichen wird. Obwohl mit dem Stahlbeton verwandt, weichen Elemente mit Verstärkungen durch geklebte Stahllamellen in ihrem Verhalten stark davon ab. Dieser Artikel beschreibt die Ergebnisse der Untersuchungen zu drei speziellen Fragen und stellt die sich daraus ergebenden Empfehlungen für die Ueberprüfung der mit dieser Technik ausgeführten Verstärkungen vor.

SUMMARY

For nearly twenty-five years, the bonded-plate technique, one of the many ways of reinforcing concrete structures, has undergone considerable development. Plates bonded to the surface of the concrete are used instead of passive reinforcements inside. The technique is related to reinforced concrete, but there are significant differences in behaviour. This article describes the results of researches on these three particular points and the resulting recommendations for the checking of reinforcements using this technique.



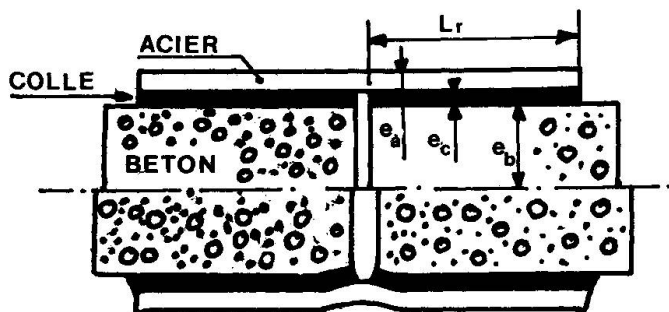
INTRODUCTION

Devant une structure en béton qui présente des insuffisances vis-à-vis des charges d'exploitation diverses solutions de renforcement peuvent être envisagées. On peut, par exemple, ajouter ou rajouter de la précontrainte si les conditions et l'état global le permettent. C'est une solution coûteuse qui n'est mise en oeuvre que pour le renforcement global d'une structure. Dans bien des cas on préférera s'en tenir à un renforcement passif qui consiste à augmenter l'inertie des sections "faibles". Cette augmentation peut se faire de deux façons : par ajout de matière dans la partie comprimée ou par ajout d'armatures dans la partie tendue. Dans ce dernier cas les armatures, sous forme de barres, sont noyées dans du béton banché ou du béton projeté ou bien lorsqu'elles se présentent sous forme de tôles, elles sont collées sur la surface du béton.

Cette dernière forme de renforcement, connue depuis près de vingt-cinq ans [1.à.3], a vu son emploi connaître un développement particulièrement important durant ces dernières années surtout dans le domaine du bâtiment. Par suite de certains déboires survenus en 1978, sur le viaduc d'accès au Pont de Gennevilliers, renforcé par des tôles dans les zones de couplage, le Laboratoire Central des Ponts et Chaussées a étudié cette technique sur le plan du fonctionnement. Les travaux entrepris ont mis en évidence trois problèmes de fonctionnement qui forment l'une des spécificités de cette technique.

Après avoir présenté les résultats des études menées sur chacun de ces trois problèmes, on présentera dans la quatrième partie la traduction qui en a été faite dans l'annexe I du document du STRRES, fascicule n° 6 "Tôles collées".

FLEXION LOCALE



Qualitativement, la flexion locale des tôles a été mise en évidence à l'aide des mesures réalisées sur le renforcement du viaduc d'accès au Pont de Gennevilliers. Pour en quantifier les effets on peut utiliser un modèle simple qui, depuis la fin de la seconde guerre mondiale, a fait l'objet de nombreuses études. Il s'agit du joint à double recouvrement (cf. figure n° 1).

Figure 1 - Joint à double recouvrement

L'étude d'une telle structure est complexe si l'on s'intéresse aux valeurs exactes du champ des contraintes dans chacun des constituants du modèle. Par contre, comme l'on fait les auteurs qui se sont préoccupés des collages, des hypothèses simplificatrices permettent d'obtenir une bonne approximation du champ.

Les hypothèses retenues ici sont les suivantes :

- les matériaux constitutifs de l'assemblage (acier-colle-béton) présentent un comportement élastique linéaire;
- le comportement de la tôle dans la tôle sera assimilé à celui d'une poutre si le système est étudié en contraintes plane ou à celui d'une plaque si on l'étudie en déformation plane [4] ;
- la couche de colle ne reprend pas de contraintes longitudinales [4] et [5] ;
- les déformations longitudinales sont constantes dans l'épaisseur du béton.

Avec ces hypothèses on peut écrire les équations d'équilibre, de comportement et de compatibilité d'une tranche dx de l'assemblage. En résolvant ces équations par rapport au cisaillement dans le joint de colle (τ_c) et au déplacement vertical de la fibre moyenne de la tôle (w_a) on obtient le système suivant :

$$\frac{E_a e_a^3}{12} \cdot w''''_a + \frac{1}{e_c + e_b} w_a = \frac{1}{2} \cdot (e_a + e_c) \cdot \tau'_c$$

$$\tau''''_c = \frac{G_c}{e_c} \cdot \left[\frac{1}{E_a e_b} + \frac{1}{E_a e_b} \right] \cdot \tau_c + \frac{G_c e_a}{2 e_c} \cdot w''_c$$

où e_a , e_b , e_c désignent les épaisseurs d'acier, de béton et de colle.

La résolution de ce système d'équation permet de calculer le moment fléchissant qui est appliqué à la tôle au voisinage de la fissure. L'analyse de ces résultats montre que l'on obtiendrait la plastification de la tôle pour des valeurs de 30 % inférieures à celles que l'on obtient si la tôle était soumise à la traction simple. Ce résultat a été déterminé pour des tôles dont l'épaisseur est comprise entre 3 et 5 mm et pour des colles de type époxydique dont l'épaisseur est voisine du millimètre. Les essais réalisés sur des poutres renforcées [6]-[7] ont confirmé ce résultat.

On peut également démontrer, à partir de ces résultats, que l'intensité du champ des contraintes au voisinage de la fissure ne dépend pas de la longueur de recouvrement (L_r) à condition que celle-ci soit au moins égale à deux fois l'épaisseur du béton (e_b).

Deux possibilités existent quand il est nécessaire de mettre en oeuvre des sections de tôles importantes. Une tôle de forte épaisseur peut être utilisée ou bien plusieurs tôles de faible épaisseur peuvent être collées l'une sur l'autre. Le tableau suivant indique les efforts repris par chacune des tôles dans les cas de superposition de 2 et 3 tôles (les tôles ont une même section).

	$F = F_1 + F_2$	$F = F_1 + F_2 + F_3$
$F_1 = 0,66 \cdot F$ $F_2 = 0,34 \cdot F$		
	$F_1 = 0,5 \cdot F$ $F_2 = 0,25 \cdot F$ $F_3 = 0,25 \cdot F$	

Toutes les tôles d'un assemblage de ce type subissent la même flexion locale.



REPARTITION DES DEFORMATIONS

La tenue des barres d'acier dans du béton se fait par "adhérence" c'est-à-dire en fait par mise en compression de petites bielles de béton s'arc-boutant sur les crénelures des barres. La résistance des barres de béton d'acier ne peut donc se faire qu'avec un certain déplacement de l'extrémité libre. Les tôles par contre ont un mode de liaison avec le béton tout à fait différent, le transfert des efforts se fait par l'intermédiaire de la colle qui, si elle est de bonne qualité, assure la continuité des déplacements.

Si l'on examine ce qui se passe au voisinage d'une fissure d'une section renforcée les suppositions suivantes peuvent être faites : la tôle et les aciers subissent les mêmes déplacements dans leur partie libre, les déformations de chacun des constituants dépendra du mode de liaison. Ainsi l'hypothèse couramment admise des sections planes se trouve mise en défaut.

Une étude sur modèle numérique, puis sur modèle physique [7] a permis de chiffrer cet écart. Le tableau suivant résume les résultats obtenus sur les sept poutres testées durant cette étude.

Tôle de 3 mm		$\frac{\epsilon_t}{\epsilon_T}$	Tôle de 3 mm	
	1,4		Tract.	Compr.
	1,4		$\frac{\epsilon_t}{\epsilon_T}$	$\frac{\epsilon_t}{\epsilon_T}$
	1,5		1,06	1,05
			1,05	1,03
			1,11	0,99
			1,21	1,22
Tôle de 5 mm			1,10	

Ces résultats montrent que la déformation moyenne de la tôle est supérieure à la déformation qu'il subirait dans l'hypothèse des déformations planes, l'écart étant compris entre 0 et 50 %.

RUPTURE PAR DECOLLEMENT

La partie sensible d'un renforcement de poutres en béton par ajout de matière est la partie de béton située au voisinage immédiat du renforcement. Dans le cas de renforcement par tôles collées la ruine se produit la plupart du temps par propagation d'une fissure dans le béton parallèlement au plan de collage. Cette fissuration est initiée depuis l'une des extrémités du collage.

L'utilisation des concepts de la mécanique de la rupture [8]-[9] permet de déterminer, à partir de quelles sollicitations appliquées à l'un quelconque des constituants de l'assemblage, quand une fissure d'interface progresse.

Dans sa progression la fissure sépare l'ensemble poutre-tôle, une certaine quantité d'énergie est libérée.

Si l'on désigne par G l'énergie libérée par unité de surface on a alors :

$$G = \frac{\partial}{\partial a} (W_{ext} - W_{el}) Q = cst$$

W_{ext} : travail des forces extérieures
 W_{el} : énergie élastique
 a : aire de la fissure
 A : charge extérieure

Le tableau suivant donne les valeurs de G pour un tirant et pour une poutre rectangulaire renforcée sur sa face inférieure.

	$G = \frac{F^2}{2b^2} \left[\frac{1}{E_b e_b} - \frac{1}{E_a e_a + E_b e_b} \right]$
	$G = \frac{1}{2b} \left(M_u^2 \left[\frac{1}{E_b I_b} - \frac{1}{(EI)^*} \right] + T_u^2 \left[\frac{k_1}{G_b A_b} - \frac{k_2}{(GA)^*} \right] \right)$

* Valeurs calculées sur les sections homogénéisées

La matière dans laquelle se propage la fissure oppose une certaine résistance. Celle-ci est traduite par la quantité R que l'on appelle le taux de restitution critique. Ainsi la fissure se propagera si $G > R$ et ne se propagera pas dans le cas contraire. La quantité R est une propriété intrinsèque du matériau dans lequel se propage la fissure, sa valeur n'est accessible que par l'expérience. Dans le cas du béton au voisinage de l'interface cette valeur est de 50 J/m^2 .

JUSTIFICATION D'UN RENFORCEMENT

Le fonctionnement particulier des tôles collées a nécessité l'introduction de nouvelles règles de vérification d'un renforcement. Ces règles portent sur les points suivants :

- section d'acier à prendre en compte aux états limites de service : la répartition des déformations n'étant pas plane dans une section on applique, suivant que les aciers internes se déforment plus que les tôles ou le contraire, un coefficient minorateur aux sections d'aciers externes (K_e) ou aux sections d'aciers internes (K_i). Ces coefficients sont donnés par les formules :

$$\begin{aligned} K_e &= 1,2 - 0,08 e_a & e_a & \text{épaisseur de la tôle} \\ K_i &= 0,46 + 0,08 e_a & & \text{exprimée en millimètres} \\ & & & (3 < e_a < 5) \end{aligned}$$

Deux calculs sont nécessaires.

Ces formules sont déduites des expériences et de la modélisation numérique faite pour interpréter les expériences.

A l'état limite ultime aucun coefficient n'est à appliquer aux sections d'acier.

- Contrainte limite des aciers de renforcement aux états limites de service. Après avoir déterminé les efforts appliqués aux aciers d'après ce qui précède et d'après la règle de répartition énoncée au paragraphe 2 dans le cas d'empilement de tôle, on calcule les contraintes moyennes dans les tôles.

Ces contraintes sont comparées à la contrainte limite σ_{et} donné par la formule :

$$\sigma_{et} = 0,7 \times \frac{2}{3} f_e ;$$

le coefficient 0,7 résulte de la réduction de résistance des tôles du renfort qui est due à la flexion locale au voisinage des fissures.

- A l'état limite ultime, outre les vérifications classiques on s'assurera que la condition de non décollement est vérifiée. C'est-à-dire que dans les sections d'arrêt des tôles on calcule le taux de restitution de l'énergie (G) suivant la méthode précédemment exposée et l'on vérifie que :

$$G < 50 \text{ J/m}^2$$



CONCLUSIONS

L'étude systématique de cette technique de renforcement a mis en évidence certains écarts de comportement avec ce qui est habituellement admis pour le comportement du béton armé. Les règles qui en ont été déduites permettent de prendre en compte les phénomènes physiques particuliers propres à cette technique dès l'élaboration du projet.

L'utilisation de ces règles dans des cas réels de renforcement n'ont pas conduit à des écarts très importants avec les méthodes classiques telles que préconisées par M L'HERMITE et M BRESSON, elles conduisent, en général, à augmenter de 10 à 20 % les sections d'acier de renfort.

NOTATIONS

E : module d'élasticité
V : coefficient de Poisson
G : module de cisaillement - taux de restitution de l'énergie
 σ : contrainte normale
 : contrainte de cisaillement
R : taux de restitution critique
e : épaisseur
b : largeur de la tôle
w : déplacement vertical de la fibre moyenne de la tôle

a, b, c en indice font référence à l'acier, le béton et la colle.

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Beams Strengthened by Epoxy Bonded Steel Plates

Poutres en béton armé renforcées par des plaques en acier collées à l'époxy

Verstärkung von Stahlbetonbalken mit epoxygeklebten Stahllamellen

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SUMMARY

This paper describes the initial stages of a long-term investigation of the durability of reinforced concrete beams strengthened by epoxy bonded steel plates. Details of accelerated test programmes, using freeze-thaw and salt spray environments, and long term exposure tests are given. Some initial results are presented.

RÉSUMÉ

Cet article décrit les premières étapes d'une recherche à long terme sur la durabilité de poutres en béton armé renforcées par des plaques en acier collées à l'époxy. Le détail des programmes des essais accélérés utilisant les méthodes de gel-dégel et de vaporisation salée, et des essais d'exposition des poutres aux conditions atmosphériques à long terme sont présentés. Des résultats préliminaires sont donnés.

ZUSAMMENFASSUNG

Dieser Artikel beschreibt das Anfangsstadium einer langfristigen Untersuchung über die Widerstandsfähigkeit armierter Betonbalken, die mit epoxygeklebten Stahlplatten verstärkt sind. Die Einzelheiten der beschleunigten Testprogramme, mit Gefrier- und Auftauzyklen und Salzspray-Umgebung, sind dargelegt. Die Langzeit-Bewitterungsversuche an Balken unter atmosphärischen Bedingungen sind ebenfalls gegeben. Erste Resultate werden vorgestellt.



1. INTRODUCTION

1.1 Background

The use of externally bonded steel plates is now established as a useful and economical method of stiffening and strengthening existing concrete structures. The plates are normally bonded to the concrete members by means of an epoxy resin adhesive. A range of applications of the technique to buildings and bridges has been summarised by Mays [1]. A considerable amount of research has been carried out in order to investigate the behaviour of plated beams under short term loading and to establish design criteria. The anchorage zone at the ends of the plates has been shown to play a critical role in determining the failure load. This has been investigated by Swamy and Jones et al [2, 3].

1.2 Durability Considerations

Only a relatively small amount of research work has been directed at determining the long term behaviour of strengthened beams. An on-going programme of long term exposure tests on beams of various sizes has been described by Calder [4]. It was reported that there was evidence of corrosion and some loss of bond area after ten years of exposure in an industrial environment but the load capacity of the beams was not significantly affected. This may be because the design of the specimens and their loading arrangement is such that the plate anchorage zones are restrained by the beam supports.

The long term performance and durability of epoxy bonded plated beams may be investigated at three levels:-

- (i) the material level
- (ii) the composite joint level
- (iii) the engineering level i.e. considering the full composite structural system.

Mays, Vardy and Hutchinson [1, 5] have considered the requirements for adhesives in civil engineering applications and have devised tests at level (i). The investigation described herein is concerned with levels (ii) and (iii). Bridges and other exposed structures may be subjected to freeze-thaw cycles and the action of salt spray. The investigation therefore includes accelerated durability testing using small specimens in a freeze-thaw cabinet and a salt spray cabinet. In addition a programme of long term exposure tests on both small and large specimens is being carried out to assess the creep behaviour under long term loading and the effects of natural weathering. This paper describes the test arrangements and presents some initial results from the programme, which is still in progress.

2. TEST SPECIMENS

2.1 External Exposure Tests

Two sizes of beams are being used, as shown in Figure 1(a) and (b). The beams have been designed so that anchorage zone failure, rather than yielding of the steel plate, is likely to be critical. Two different epoxy resins have been used to bond the plates. Most of the beams were plated in the unloaded condition but two pairs of the 2500mm long and two pairs of the 1000mm long beams were plated whilst they were under a sustained load equal to 50% of the load capacity of the unplated beams. This meant that the concrete in the tension zones of these beams was cracked when the plates were applied. Unplated beams of each size are being used to provide a basis for comparison.

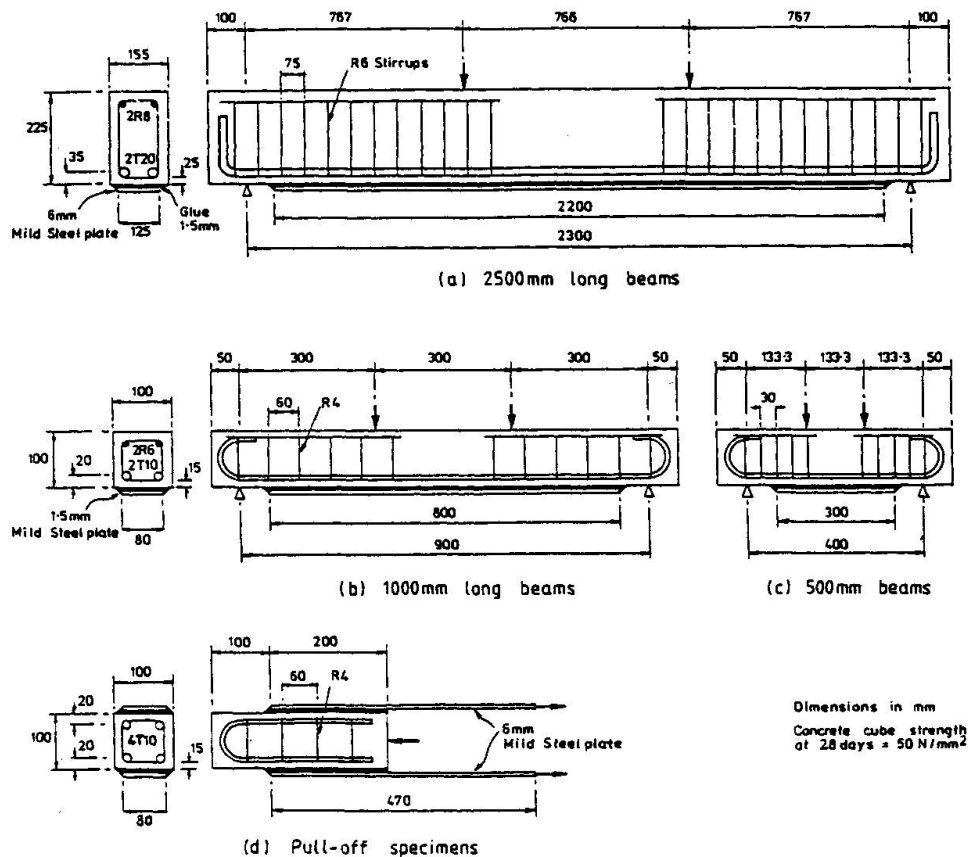


Fig. 1 Details of Test Specimens

2.2 Accelerated Tests

Smaller beams are being used for these tests so that they can be accommodated in the freeze-thaw and salt spray cabinets. These are shown in Figure 1(c). In addition, the pull-off specimens shown in Figure 1(d) have been designed to simulate the conditions in the anchorage zones of the plated beams. The same two epoxy resins have been used in these specimens as were used in the larger beams in the external exposure programme. In addition, the influence of different forms of preparation of the steel plates is being investigated by including specimens in which the plates were primed before the resin was applied and in which zinc grit blasting was used as an alternative to the normal grit blasting surface preparation.

3. TEST PROGRAMMES

3.1 External Exposure Tests

These beam tests are summarised in Table 1. Duplicate sets of long term beams are being used so that the effects of two exposure periods may be investigated. It is initially planned to use periods of 5 years and 10 years, but these may be revised as data is accumulated. One set of beams will be returned to the laboratory and load tested at the end of each period. The long term control beams are being permanently stored in the laboratory.



BEAM TYPE		LONG TERM EXTERNAL EXPOSURE				LONG TERM CONTROL		SHORT TERM	
		LOADED		UNLOADED		LOADED	UNLOADED	CONTROL	
		2500mm	1000mm	2500mm	1000mm	1000mm	1000mm	2500mm	1000mm
Plated	Resin 1	2 No.	2 No.	2 No.	2 No.	2 No.	2 No.	1 No.	1 No.
	Resin 2	2 No.	2 No.	2 No.	2 No.	2 No.	2 No.	1 No.	1 No.
Cracked and Plated	Resin 1	2 No.	2 No.	-	-	-	-	1 No.	1 No.
	Resin 2	2 No.	2 No.	-	-	-	-	1 No.	1 No.
Unplated		2 No.	2 No.	2 No.	2 No.	2 No.	2 No.	1 No.	1 No.

Table 1 External Exposure Test Programme

The loaded beams are supported in pairs in purpose designed loading rigs and the unloaded beams are stored on racks. The overall layout of the exposure site is shown in Figure 2. The loaded beams were loaded to 50% of the load capacity of an unplated beam, as determined from the short term control tests. Strains at the midspan cross-sections of all beams are monitored at intervals using demountable mechanical gauges. The ambient temperature at which readings are taken is also recorded. At approximately 8 to 9 months after the initial loading the loads on the beams were checked, and were found to have reduced by 15% to 20%. The rigs were re-loaded to their original loads and this checking and re-loading procedure will be repeated at intervals throughout the test programme.

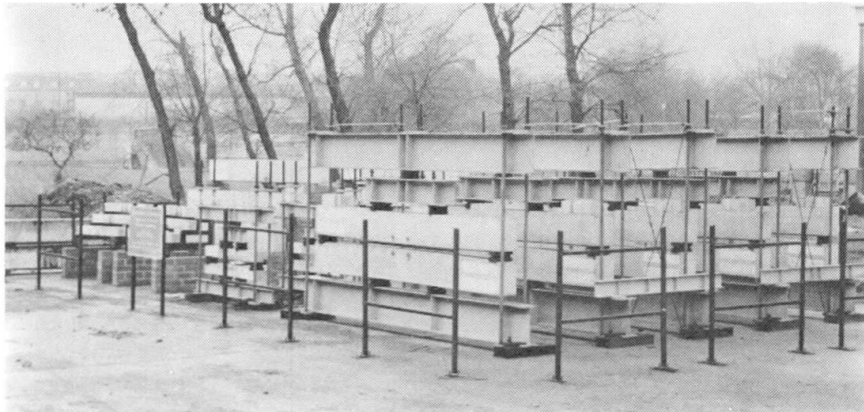


Fig. 2 External Exposure Site

3.2 Accelerated Tests

The numbers of specimens used in this test programme are shown in Table 2. Again, duplicate sets of specimens have been constructed to enable two exposure durations to be used for each type of test. The salt spray specimens will be subjected to 500 and 1000 cycles and the freeze-thaw specimens to 100 and 200 cycles.

Demountable mechanical gauges are used to monitor permanent changes in strain and relative movement between the plates and the concrete in the freeze-thaw specimens. Additional dummy specimens with thermocouples embedded at various



BEAM TYPE		ACCELERATED EXPOSURE				SHORT TERM	
		FREEZE/THAW		SALT SPRAY		CONTROL	
		500mm	P.O.S.	500mm	P.O.S.	500mm	P.O.S.
Plated	Resin 1	2 No.	2 No.	2 No.	2 No.	1 No.	1 No.
	Resin 2	2 No.	2 No.	2 No.	2 No.	1 No.	1 No.
	Resin 2 + ZINC grit	2 No.	2 No.	2 No.	2 No.	1 No.	1 No.
	Resin 2 + primer	2 No.	2 No.	2 No.	2 No.	1 No.	1 No.
Unplated		2 No.	-	2 No.	-	1 No.	-

P.O.S. = Pull-off specimen

Table 2 Accelerated Test Programme

depths below the surface are being used to monitor the time-temperature relations and the temperature gradients within the concrete. At the end of each accelerated exposure period, the specimens will be removed from the cabinets, visually examined and load tested. Short term control specimens are used to give a basis for assessing the effect of the exposure type and period on the structural performance of the specimens.

4. RESULTS TO DATE

4.1 External Exposure Tests

Typical strain data for the compression face of a loaded plated beam are presented in Figure 3. The readings taken on the companion unloaded beam are used to determine the strain due to shrinkage and moisture movement. All the data is corrected to a standard temperature of 15°C. This calculated thermal strain and the corrected shrinkage and moisture movement strain are plotted in the figure.

A comparison between the strain data for an unplated beam, a beam that was plated in its unloaded condition and a beam that was plated in the loaded, pre-cracked condition is shown in Figure 4. The data will be corrected to allow

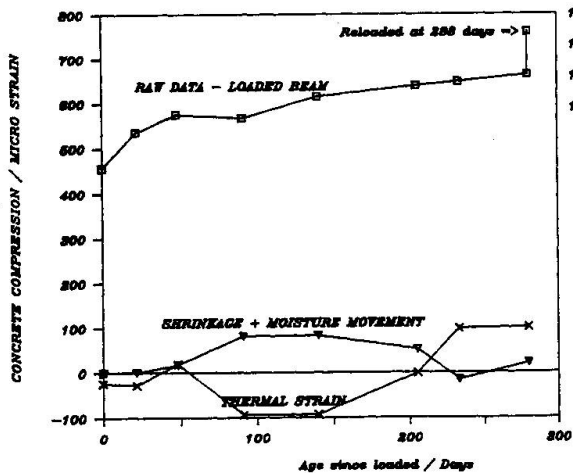


Fig. 3 Strain Data for Plated Beam

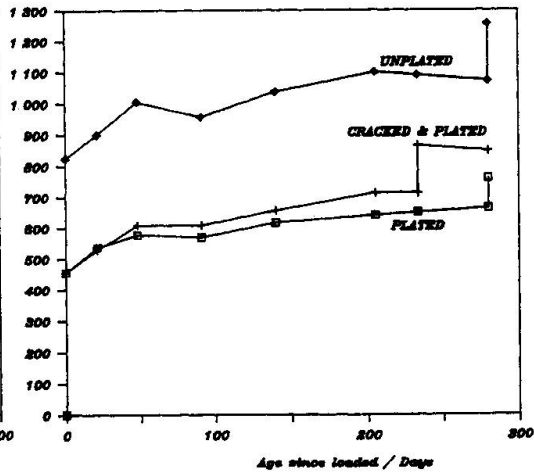


Fig. 4 Comparison of Three Beams



for the loss in load with time once long term strains after further re-loading to the original load level are available. The lower initial elastic strains on loading of the plated beams reflect their increased stiffness when compared with the unplated beam. The additional creep strains are, however, similar up to this stage.

4.2 Accelerated Tests

The initial periods of testing in the salt spray and freeze-thaw environments have yet to be completed, so no load test data is yet available. Visual inspection of the salt-spray specimens after 450 cycles revealed some evidence of surface deterioration of the steel plates, producing rust staining of the concrete. Strains in the freeze thaw specimens after 75 cycles indicate a net expansion of less than 50 microstrain. There is no measurable differential movement between the plates and the concrete.

5. CONCLUSIONS

These initial results indicate that the creep behaviour of plated beams is similar to that of unplated beams. This tentative conclusion remains to be confirmed as longer term data is obtained. The accelerated test programmes indicate that plated beams are able to survive 450 cycles of salt spray and 75 cycles of freeze-thaw action without serious apparent deterioration. The actual effect on anchorage bond strength remains to be established.

6. ACKNOWLEDGEMENTS

The test programmes described have been financed by a grant from the Science and Engineering Research Council. The epoxy resins have been supplied by Messrs Formulated Resins Ltd and Messrs Sika Construction Products.

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Strengthening of Reinforced Concrete Beams

Renforcement de poutres en béton armé

Verstärkung von Stahlbetonträgern

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SUMMARY

Results of laboratory tests on bending strengthened reinforced concrete beams are presented. The laboratory tests threw light in the efficacy of different techniques of bending strenghtening.

RÉSUMÉ

Les résultats d'essais sur des poutres en béton armé renforcées sont présentés. Les essais exécutés ont permis de connaître et de comparer l'efficacité des différentes techniques de renforcement.

ZUSAMMENFASSUNG

Es wurden verschiedene Methoden zur Versteifung von Stahlbetonträgern untersucht. Die durchgeführten Versuche lassen die Wirkung der verschiedenen angewandten Verstärkungstechniken auf die Durchbiegung der Balken erkennen.



1. INTRODUCTION

Repair and structural strengthening are matter of great importance today, for they increase safety and durability of structures. Premature deterioration (corrosion, cracking, deformation, etc), design or construction faults, increase in live loads, accidents, etc, are some of the most frequent causes.

Papers and books describing strengthening or repair executed upon existing structures, sometimes with load tests, are frequently obtained (1), (2), as well as papers dealing with laboratory tests on models strengthened using a certain technique (1), (3). It is not easy, however, to find papers comparing different strengthening techniques, which motivated the present research.

Reinforced concrete beams, strengthened with additional bending reinforcement were tested, and efficacy compared among different techniques. Later, other reinforced concrete beams were repaired with grout, substituting concrete crushed by bending. These beams were once again loaded, a few days after repair, and the results were analysed. The description of the laboratory tests and the outcome are described as follow.

2. STRENGTHENING OF BENDING REINFORCEMENT

Three different bending strengthening techniques of reinforced concrete beams were compared: shotcrete, glued mortar and glued steel plates.

2.1 Beams and Type of Strengthening

Twelve equal beams, 2.0 m in length each, were poured (fig. 1). Three of them were not strengthened (beams SB), and were used as a lower-bound limit. Three beams were strengthened with shotcrete (beams SH, fig. 2), and other three with glued mortar (beams MB, fig. 2). The remaining three were strengthened with epoxy-glued steel plates. For the beam in fig. 3.a (beam SP1), only epoxy was used to connect plate to concrete. For beam SP2 in fig. 3.c, U plates were used to connect steel plate to the beam, and for beam SP3 (fig. 3.b) bolts were used to assure a better connection between plate and concrete.

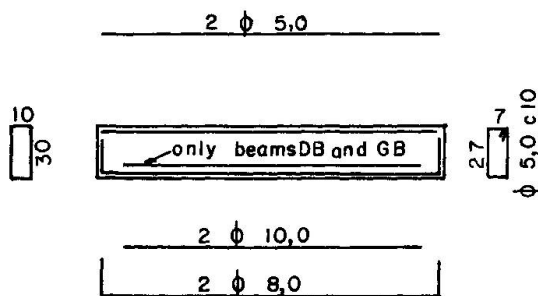


Fig. 1 Beam reinforcement

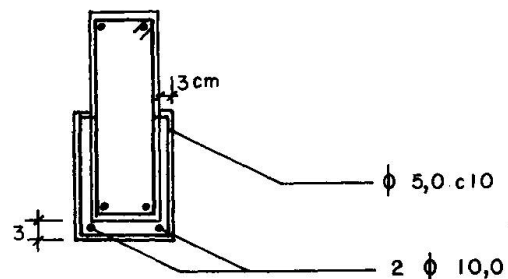


Fig. 2 Shotcrete and glued mortar

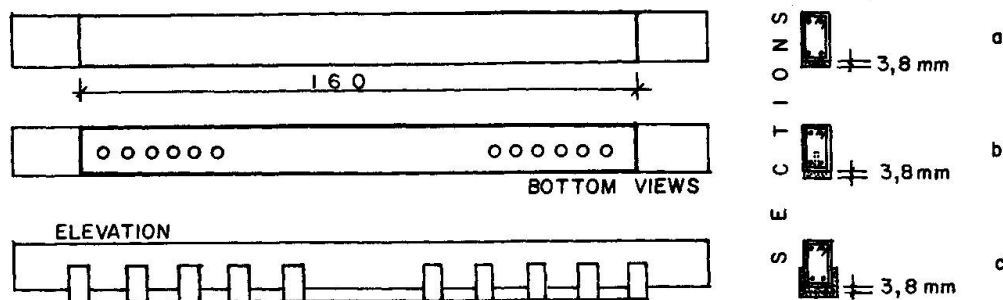


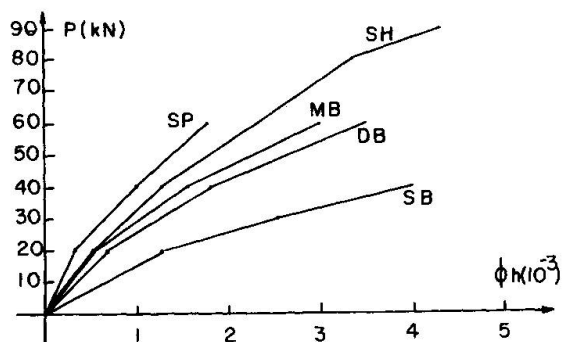
Fig. 3 Epoxy-glued steel plate strengthening

Three other beams received a second reinforcement layer before pouring (beams DB in fig. 1), and were used as an upper-bound limit.

2.2 Executed Tests

All beams were tested for bending, with a 1.85 m span and two equal concentrated loads distant 0.30 m between them. During the tests, vertical displacements, bending and shear crack widths, and specific strains along depth at mid-span, were measured.

Fig. 4 displays "load x curvature" diagrams, and table 1 shows the design loads (P_d), the theoretical (P_u) and the experimental ($P_{u,exp}$) collapse loads (4). The listed values are total loads applied to the beam (sum of two concentrated loads).



BEAM	P_d	P_u	$P_{u,exp}$
SB 1/2/3	33,6	46,5	51,5
SH 1/2	71,3	115,0	119,5
SH 3	71,3	115,0	101,5
MB 1/2/3	71,3	115,0	91,3
SP 1	51,0	77,0	70,0
SP 2/3	76,0	90,6	99,0
DB	60,0	98,3	106,0

Fig. 4 Load x Curvature diagram

Table 1 Theoretical and experimental loads (kN)

2.3 Tests Results Analysis

The following items were noticeable during the tests:

- All tested beams exceeded the design loads;
- Two beams with glued steel plates and special devices to assure connection to the beam, SP2 and SP3, exceeded theoretical collapse load by 9%; collapse load for beam SP1 (fig. 5) was lower than the theoretical one by 10% (5); all beams have collapsed showing a disconnection of the steel plate from the beam (although this was expected only for beam SP1), and none of them have showed rupture of the steel plate.
- Two shotcrete beams surpassed theoretical collapse load by 4%; the other one failed with 88% of expected load, probably because of an old concrete area with



- out rugous surface (fig. 6);
- Beams strengthened with glued mortar failed with 79% of the expected load;
- Beams strengthened with glued steel plates were the stiffest, followed by shotcrete strengthened beams;
- Beams strengthened with glued steel plates showed the best behaviour for crack width, with no cracks in strengthened region until the design loads were reached; nevertheless, beam SP1 collapsed soon after the first horizontal crack in the concrete cover occurred (fig. 5).

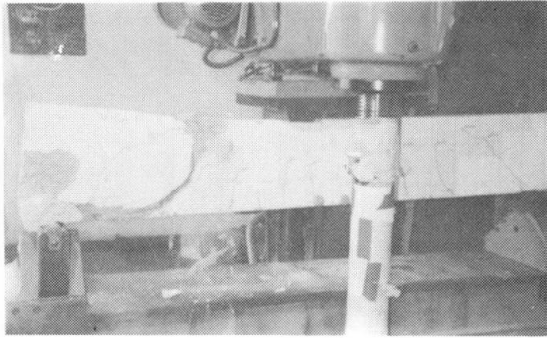


Fig. 5 Steel plate beam

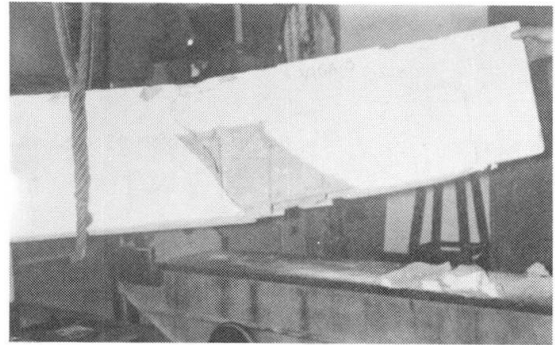


Fig. 6 Shotcrete beam

3.GROUT REPAIR

3.1 Beams and Repair Execution

During the second stage three beams were built (identical to DB beams), and named GB beams. These beams were subjected to test loads until failure, which always occurred crushing compressed concrete without yielding of the tension reinforcement.

These beams were repaired, by pouring grout (high strength expansive mortar) to substitute the damaged concrete.

3.2 Results Analysis

Beams were once more loaded, and fig. 7 shows "load x curvature" diagrams for tests before and after repair. Collapse loads before and after repair are presented at table 2, and a repaired beam before and after failure is showed at figures 8 and 9.

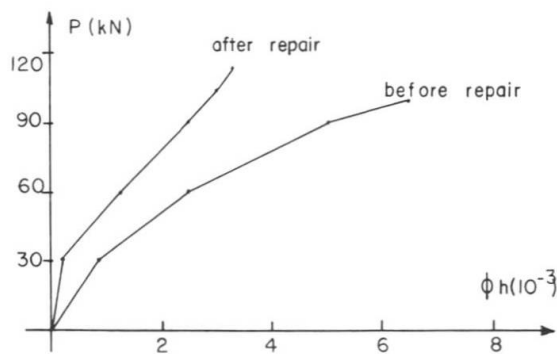


Fig. 7 Load x Curvature diagram

BEAM	$P_{u,exp}$ before	$P_{u,exp}$ after
GB 1	109,0	124,5
GB 2	108,5	130,0
GB 3	106,0	88,0

Table 2 Failure loads before and after repair (kN)

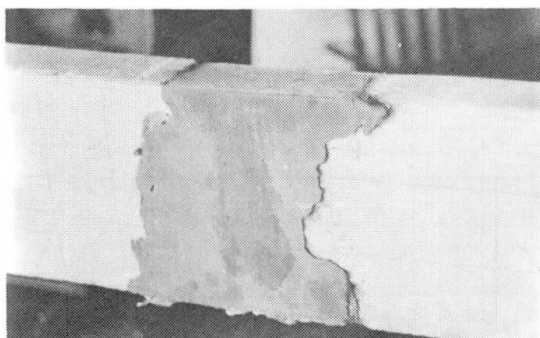


Fig. 8 Grout repaired beam

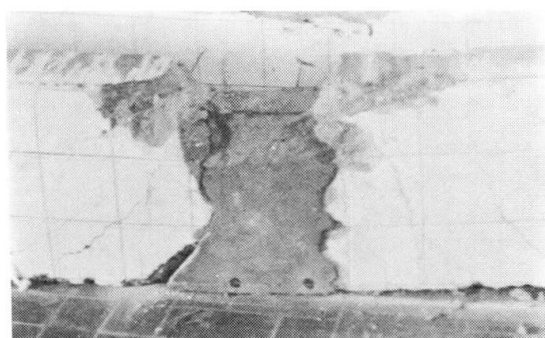


Fig. 9 Grout beam after failure

The following items were noticeable during the tests:

- Two repaired beams, loaded at ages 4 and 8 days, presented increase of collapse loads of 14% and 20%, respectively;
- The other one failed with 83% of initial collapse load, probably because before use, the grout was maintained in an open bag for 40 days, at risk of hydration by the 75% local air humidity; this result was dismissed.
- Although strains measured at the grout (2nd stage of load) were smaller than at the initial concrete (1st stage of load), as displayed at fig. 7, vertical displacements were larger, because a great part of the beam was already cracked.

4. CONCLUSIONS

Some conclusions could be extracted from this research:

- Strengthening with glued steel plates and with shotcrete agree very well with the expected values of failure loads (the expected increase in initial load at about 100% was reached);
- Repair of the damaged concrete area with grout reached very good results, and can be used when it is necessary to load the structure at early ages;
- From the economic point of view, shotcrete would be used to strengthen large areas, where it is possible to obtain great productivity, and the cost of equipment mobilization is attenuated;
- Glued steel plate strengthening is specially indicated for little areas, because large equipments are unnecessary; this technique, however, requires local temperatures below 70° C, and the designing of an efficient connection between steel plate and concrete beam;
- For all repair and strengthening techniques, a rigorous quality control is needed, including materials and execution, in order to warrant safety and durability.

5. ACKNOWLEDGMENTS

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Concrete Repair with Shotcrete
Réparation du béton par béton projeté
Betonreparaturen mit Spritzbeton

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SUMMARY

Shotcrete appears to be useful in repairing vertical and overhead surfaces of deteriorated concrete structures. It is essential that water-jetting, cleaning and shotcreting operations are all carried out meticulously. Nine reinforced concrete beam specimens sawn out of water-jetted and shotcreted concrete slabs have been loaded statically and under fatigue conditions. The varying parameter was the ratio between shotcrete thickness and total beam depth. According to the test results, the interface between old concrete and shotcrete should not coincide with the reinforcement plane.

RÉSUMÉ

Il est possible d'utiliser du béton projeté pour réparer les constructions endommagées en béton. Les travaux relatifs au traitement par jet d'eau, au nettoyage et à la projection de béton doivent être exécutés méticuleusement. Neuf poutres en béton armé sciées à partir de dalles traitées au jet d'eau et revêtues de béton projeté ont été soumises à des charges statiques et dynamiques, la variable étant le rapport entre l'épaisseur de la couche de béton projeté et la hauteur totale de la poutre. Les essais ont montré que la jonction entre l'ancien béton et le béton projeté ne doit pas coïncider avec le plan d'armature.

ZUSAMMENFASSUNG

Spritzbeton ist offensichtlich auch zur Reparatur der Betonkonstruktionen geeignet. Dabei sind aber sämtliche Arbeiten wie Wasserstrahlbearbeitung, Reinigung und Spritzen sorgfältig auszuführen. Bei den Versuchen wurden neun Stahlbetonbalken aus wasserstrahlbearbeiteten und mit Spritzbeton versehene Betonplatten mit statischer Last und Ermüdungslast beansprucht. Variierender Parameter war der Anteil der Spritzbetonschicht an der Gesamtbalkenhöhe. Nach den Versuchsergebnissen sollte die Fuge zwischen dem alten Beton und dem Spritzbeton nicht in der Bewehrungsebene liegen.



1. INTRODUCTION

Extensive damage due mainly to de-icing salts and repeated freeze-thaw cycles is found on many concrete bridges. The deteriorated concrete has to be removed and replaced by new concrete. For some years now the water-jet technique has been the most frequently used method of concrete removal in Sweden. Using water-jets technique or hydrodemolition means that concrete is removed by high pressure water-jets, see [4].

Experience from repair of concrete bridge decks in Sweden and laboratory tests carried out at the Department of Structural Mechanics and Engineering, the Royal Institute of Technology, Stockholm [1] and [2], has shown that a good bond between old and new concrete can be obtained without expensive stud bolts. In replacing deteriorated concrete on vertical and overhead surfaces, shotcreting is more convenient than making complicated shuttering and pouring conventional concrete.

Can shotcrete be used to repair a structural concrete member and assuredly adhere to carry its share of static and fatigue load? The tests described in this paper were aimed at answering this question. Since the damage thickness varies within a single concrete structure and between structures, the purpose was also to study how the location of the interface in relation to the reinforcement plane influences the load-bearing capacity of the composite structure. Reference [3] deals in detail with the test results.

2. TEST SPECIMENS AND TEST SPECIMEN PREPARATION

The test programme embraced a total of twelve reinforced concrete beams cut from slabs. Four concrete slabs were cast. Three slabs were water-jetted and the varying parameter was the thickness of the concrete removed. The aim was to precisely reach the reinforcement on the first slab (A), to uncover half of the reinforcement bars on the second one (B), and to completely uncover the reinforcement on the third slab (C). The fourth slab was a reference slab (R) which was not water-jetted.

In a second phase of test specimen preparation, the removed concrete was replaced by shotcrete to the level of the original slab thickness. The slabs were finally sawn into strips, resulting in beam specimens. The test specimens were designated A1, A2, A3 etc. The cross-sections are shown in Fig. 1 and material properties are listed in Table 1.

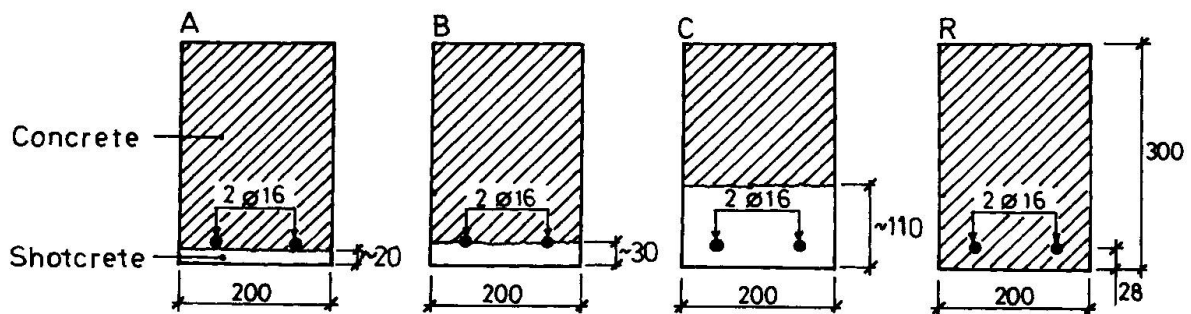


Fig. 1 Cross sections of specimens of type A, B, C and R.

	Cube length (mm)	Age (days)	Compressive strength (MPa)	Splitting tensile strength (MPa)
Concrete of 1st casting	150	238	31.5	2.8
Shotcrete	100	92	77.7	6.0
		Diameter (mm)	Yield strength (MPa)	Ultimate strength (MPa)
Reinforcement bars		16	466	579

Table 1 Material properties of the test specimens

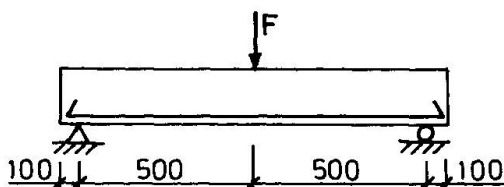
3. MAPPING OF THE LOCATIONS OF THE INTERFACES

After casting of the original concrete slabs the concrete cover was about 20 mm on all slabs. The concrete cover and a varying amount of concrete surrounding the reinforcement bars was removed by water-jet operations. Minimum, maximum and average values of the thickness of the removed concrete and of the ratio between uncovered perimeter and total circumference of the reinforcement bars were measured and are shown in Table 2.

Slab No.	Thickness of removed concrete (mm)			Ratio between uncovered perimeter and total circumference of rebar (percentage)		
	Minimum	Average	Maximum	Minimum	Average	Maximum
A	14	22	37	0	2	30
B	15	30	45	0	23	50
C	75	110	135	100	100	100

Table 2 Data on removed concrete and uncovered reinforcement bars

4. LOADING



One beam specimen of each type was statically loaded to failure and two of each type were loaded by fatigue. All beams were simply supported on two supports and loaded by a central single load according to Fig. 2.

Fig. 2 Loading of the beam specimens



5. STATIC TESTS

The results of the static tests are shown in Table 3. For specimen No. B2 with the interface between concrete and shotcrete coinciding with the reinforcement plane a combined shear and anchorage failure occurred. For the other three specimens flexural failures occurred. Calculated yield load at flexure and ultimate shear load for a monolithic R.C. beam with equal dimensions and material properties were 96 and 167 kN respectively.

Specimen No.	Yield load (kN)	Ultimate load (kN)	Final failure mode
A2	95	98	Flexural failure
B2	90	95	Combined shear and anchorage failure
C2	90	100	Flexural failure
R2 ¹	95	101	Flexural failure

¹ Homogeneous reference beam

Table 3 Obtained failure loads and failure modes

6. FATIGUE TESTS

The load varied sinusoidally and the frequency was increased from 1 Hz at the first cycle of repeated loading to 4 Hz at approximately the 1000th cycle. Thereafter the frequency was kept constant. The results are shown in Table 4. In conformity with the static tests, combined shear and anchorage failure occurred

Specimen No.	F _{max} ¹ (kN)	F _{min} ² (kN)	N ³	Failure mode
A1	70	10	313 000	Flexural failure
B1	70	10	23 000	Combined shear and anchorage failure
C1	70	10	227 000	Flexural failure
R3 ⁴	70	10	220 000	Flexural failure
A3	75	15	205 000	Flexural failure
B3	75	15	20 000	Combined shear and anchorage failure
C3	75	15	166 000	Flexural failure
R1 ⁴	75	15	268 000	Flexural failure

¹ F_{max} = maximum value of the sinusoidal load
² F_{min} = minimum value of the sinusoidal load
³ Number of cycles of repeated loading at failure
⁴ Homogeneous reference beam

Table 4 Number of cycles at failure and failure modes

for the two beams (B1 and B3) with the interface coinciding with the reinforcement. In all other cases flexural failure occurred. The number of cycles of repeated loading at failure for beams No. B1 and B3 was about one tenth of the corresponding number for the other beams, both composite and homogeneous.

7. PULL-OFF TESTS AND TORSIONAL TESTS OF CORES

The tests were terminated with pull-off tests and torsional tests of cores according to Fig. 3. The results are shown in Table 5. Since no pure interface failure occurred it can only be stated that the tensile and shear strength of the interface cannot have been less than the failure values obtained. The failure stress was 1-2 MPa at tension (pull-off tests) and 3-4 MPa at shear (torsional tests). No differences dependent on the location of the interface in relation to the reinforcement plane could be determined according to these tests.

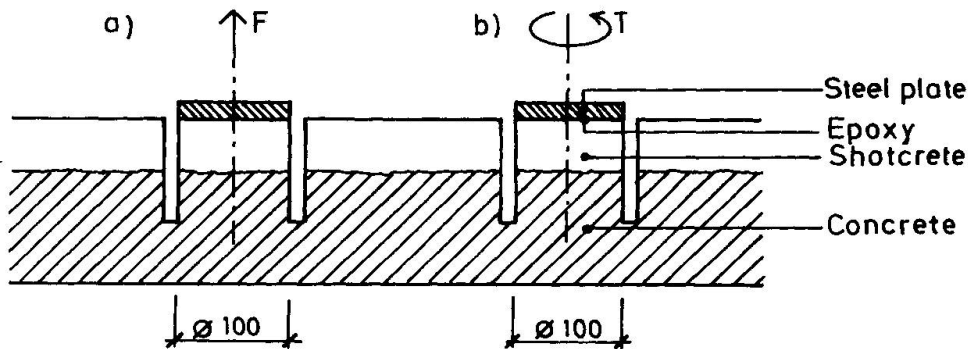


Fig. 3 Pull-off tests (a) and torsional tests (b) of cores

Specimen No.	Pull-off tests		Torsional tests	
	Number of cores	Failure stress average (MPa)	Number of cores	Failure stress average (MPa)
A2 ¹	3	1.47	3	2.99
A1 ²	3	1.73	3	3.04
B2 ¹	2	1.92	2	3.81
B1 ²	2	1.36	2	3.38
C2 ¹	3	1.77	3	3.41
C3 ²	3	2.06	3	3.62
R2 ¹	3	1.64	3	2.68
R3 ²	3	1.95	3	2.86

¹ The specimen was previously loaded statically
² The specimen was previously loaded by fatigue

Table 5 Pull-off tests and torsional tests of cores Ø100 mm



8. CONCLUSIONS

Shotcrete appears to be useful in repairing vertical and overhead surfaces of deteriorated concrete structures. It is essential that water-jetting, cleaning and shotcreting operations are all carried out meticulously. According to the tests the interface between old concrete and shotcrete should not coincide with the reinforcement plane. The test results show a change in failure mode and less resistance to fatigue for this case. The probable explanation is that the bond between reinforcement and concrete is less if the interface between old concrete and shotcrete coincides with the reinforcement plane.

ACKNOWLEDGEMENTS

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Control Tests for Marine Concrete Repair Materials

Essais de contrôle pour matériaux de réparation de béton marin
Kontrollproben für Material für Reparaturen an Meeresbauwerken

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SUMMARY

After a brief description of experimental work in which a large number of damaged concrete slabs and domes were repaired underwater, this paper explains why the existing "standard" slant-shear test is not satisfactory for comparing the effectiveness of repair materials, such as cementitious grouts and epoxy resins. An investigation of alternative control tests for repair materials is introduced, and the findings are summarized.

RÉSUMÉ

Cette communication contient une courte description de certains travaux expérimentaux portant sur la réparation sous-marine d'un grand nombre de dalles et de dômes en béton, lesquels avaient subi des dégâts importants. Une analyse du test standard "slant-shear" montre que ce test est peu fiable pour une étude comparative de l'efficacité des matériaux de réparation tels que des laits de ciment et des résines époxydes. D'autres essais de contrôle des matériaux de réparation sont indiqués.

ZUSAMMENFASSUNG

Nach einer kurzen Beschreibung von experimentellen Arbeiten, bei denen eine Vielzahl von beschädigten Betonplatten unter Wasser repariert wurden, erklärt der Artikel warum der bestehende "standard slant-shear"-Versuch für den Vergleich der Wirksamkeit von Reparaturmaterial wie Zementmilch und Epoxyharzen nicht befriedigend ist. Darauf folgt eine Einführung in die Untersuchung von alternativ Kontrollproben für Reparaturmaterial sowie eine Zusammenfassung der Ergebnisse.



1. INTRODUCTION

The use of concrete in the marine environment, and the subsequent damage of that concrete, has provided some unique maintenance and repair problems.

Concrete has been used successfully for the construction of offshore oil and gas production and storage platforms, support columns for jetties, causeways and bridges, harbour walls, breakwaters, piers and coastal defences. These structures are subject to many forms of damage caused by ship impact, severe wave slam, erosion by sediment laden water, differential settlement caused by scour, corrosion of embedded steel and frost damage.

Where damage occurs to terrestrial structures, access is usually a relatively simple matter, enabling the operator to assess the level of damage, cut back and clean the damaged area and subsequently effect a repair. A wide variety of products are available for the repair of normal land based structures, ranging from low viscosity epoxy resins to lightweight thixotropic mortars. Similarly, a wide variety of repair techniques can be used, these range from crack injection (using resins or cementitious grouts), patching with mortars and gunniting to the casting in-situ of complete new members.

In the marine environment, the range of techniques and materials available is severely limited. Firstly, where the damaged concrete is at significant depth, visibility is limited by the turbidity of the water, the lack of incident light and, often, by encrustations of marine growth; these make visual assessment of the original damage and any subsequent repair very difficult. Secondly, working at depth means that the dexterity of the operator is reduced and the length of time available to effect a repair is limited. Thirdly, the materials used must be compatible with underwater placement. This means that the material chosen must be capable of displacing seawater without being dispersed, must be able to bond to a saturated concrete surface, must be able to set and develop full strength at low temperatures and must provide an impermeable interface between parent concrete and repair material.

Because of the high cost of carrying out underwater repairs, together with the great difficulty of removing an inefficient repair, it is imperative that the repair contractor chooses the correct material for the job. To enable this choice to be made, the contractor requires a control test method that provides comparative information on the different repair materials in a way that is relevant to the manner in which they would be used. This test method would need to be simple to carry out, suitable for site use, and give consistent results.

This paper first describes traditional control tests, such as the slant shear test; then, in the light of extensive experimental work, shows why these are unsatisfactory. A study is reported in which various test methods were investigated with the aim of identifying and recommending one or more that could be usefully employed in both evaluating a repair material and, at the same time, producing results relevant to the behaviour of repaired prototype structures.

2. UNDERWATER REPAIRS OF DAMAGED CONCRETE ELEMENTS

As the result of earlier investigations at Imperial College [1] a collection of concrete slabs and domes existed that had been designed to model the roof members of offshore oil storage vessels. These 1/12 scale models had been subjected to hard impact from a dropped object and exhibited a range of damage, from fine hairline cracking to complete perforation. Certain of these concrete elements were then repaired in a project entitled "Repair and retesting of Concrete Slabs and Domes subject to impact damage" carried out as part of the Concrete Offshore in the Ninety's (COIN) programme[2].

In that project concrete slabs and domes exhibiting an intermediate level of damage were repaired underwater using a variety of repair materials. These materials included epoxy resins of varying viscosities and cementitious grouts, some of which were modified especially for underwater placement. The method of repair was essentially to inject the repair material into the annulus between the displaced cone and the main body of the concrete. (Fig. 1.). The slab or dome was then tested by static loading through a steel platen placed on the repaired area.

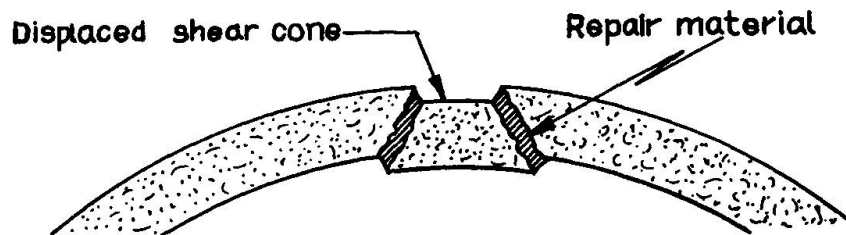


Figure 1. Sectional view of a dome repair

The results allowed a ranking, or scale, of the materials used. In parallel with the large scale repairs, all the materials were tested to BS 6319:Pt4:1984, [3] "the slant shear test for measurement of bond strength." The scale or ranking of materials produced by the slant shear tests did not, however, correlate with the ranking produced by the large scale repairs. As the large scale repairs were considered to be realistic and closely representative of the behaviour of a prototype structure, it was concluded that the slant shear test was not emulating the action of a real repair. These conclusions led to a further study [4] in which a variety of alternative test methods were investigated.

3. INVESTIGATION OF ALTERNATIVE CONTROL TESTS

These included modified slant shear tests, flexural tests and a direct tensile test. The direct tensile test was rejected at an early stage as it was apparent that difficulties in aligning the two ends of the test piece would cause unpredictable stress concentration and, consequently, unreliable results.



The slant shear test method used for epoxy resins was an adaption of the existing British Standard and consisted of testing a resin bonded scarf jointed prism by loading in compression (Fig. 2). A special jig was developed which separated and held the two halves a fixed distance apart and, at the same time, acted as a shutter for the resin injected into the crack.

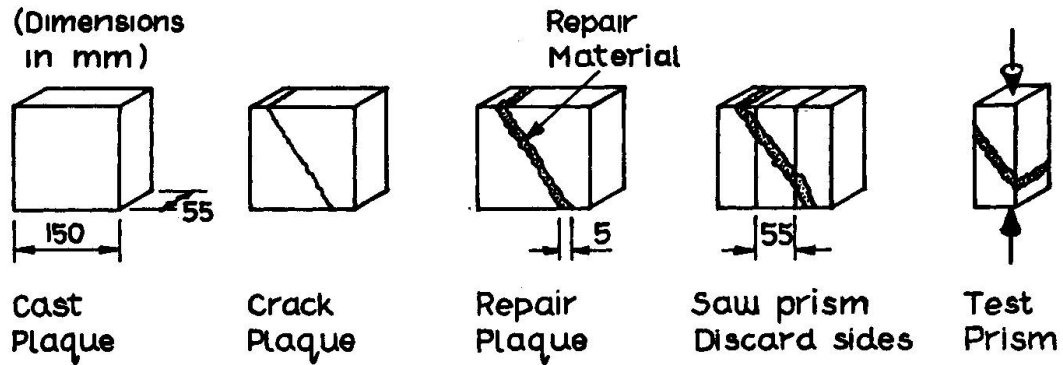


Figure 2 Resin bonded slant shear test method

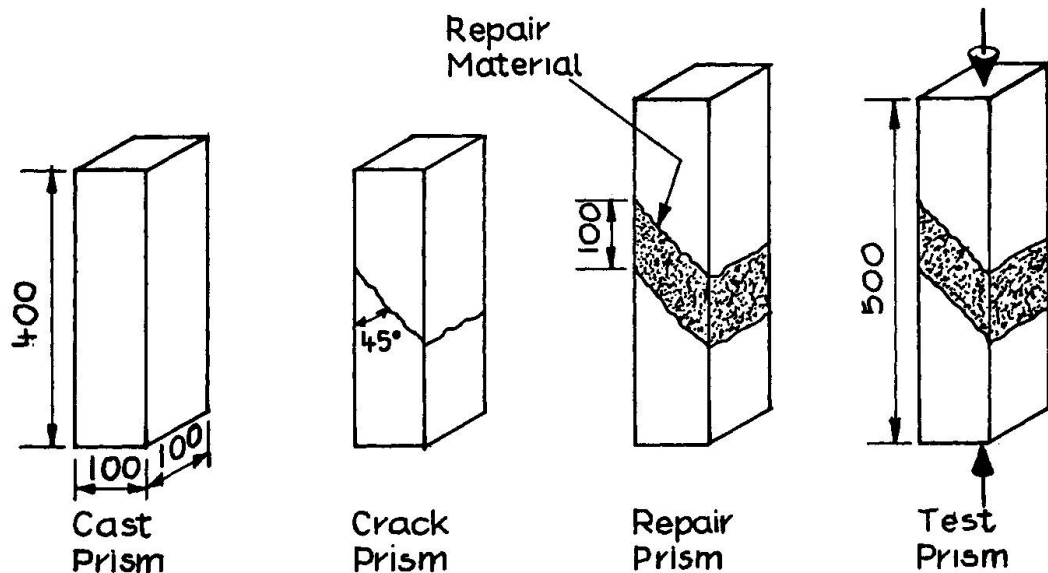


Figure 3 Cementitious grout bonded slant shear test method

The slant shear test method used for cementitious grout was similar in principle in that it also tested a scarf jointed prism in compression (Fig. 3). The test piece in this method was much larger and more robust, and the test was designed to use standard laboratory equipment and moulds.

The two flexural tests consisted of casting a short concrete beam, breaking it by flexural loading, bonding the two halves together and testing by four-point flexural loading (Fig. 4).



Cementitious Grout Bonded

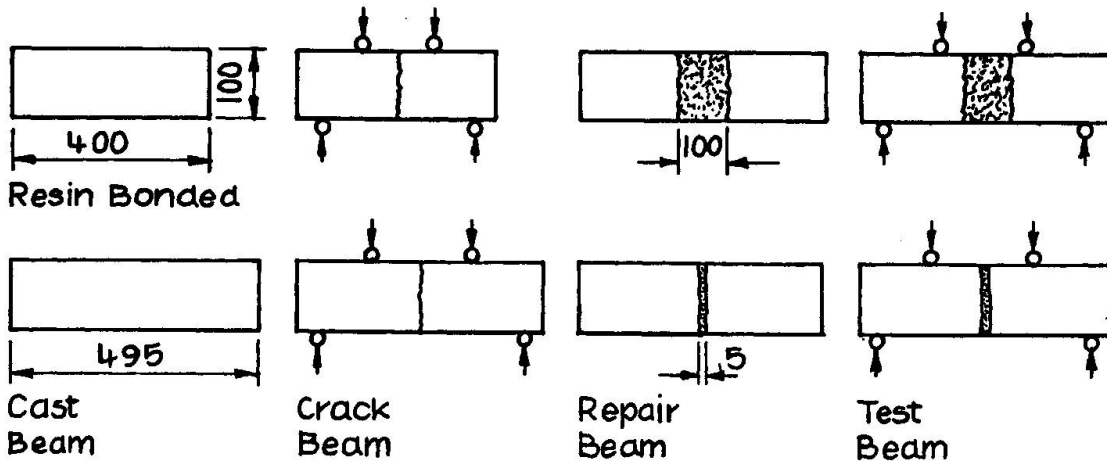


Figure 4 Flexural tests

The results of all these tests were assessed for consistency, ease of use and how they compared with the results obtained during the large scale slab and dome repairs.

4. CONCLUSIONS

For a number of reasons, it was concluded that the slant shear tests were not satisfactory. Firstly, the BS slant shear test was cumbersome to use and it proved impossible to inject some of the higher viscosity resins into the hairline crack demanded by the standard, although this problem was overcome in the modified tests where the crack width was fixed at 5mm. Secondly, it was felt that the slant shear test was not testing the repair material in the same mode in which it would be used real life. The slant shear test tests the ability of a thin layer of repair material to transmit compressive stress from one piece of concrete to another. Bond appears to have very little effect on this ability. Of far greater importance is the elastic modulus of the repair material and the roughness or interlock of the two concrete faces. In the tests reported here, where the repair material was of similar elastic modulus to the concrete (for example, in the cementitious grouts), then the compressive strength of the repaired prism was similar to the compressive strength of either the grout or the parent concrete, whichever was the weaker. Where the repair material was of a lower elastic modulus (for example, epoxy resin), then the distortion and stress concentrations in the resin caused failure of the repaired prism in the concrete adjacent to the repaired interface at loads lower than those found in plain concrete prisms.

Thus, the slant shear test, in its present form, is of limited use as a test of bond and merely gives a measure of the elastic modulus or compressive strength of the repair material. Both of these properties can be better tested by other existing methods. What is needed is some form tensile test. However, direct tensile tests are impractical and so a flexural test which is simple to use, gives reproducible results and can be used for all types of repair material is proposed by the authors.



5. RECOMMENDED TEST METHOD

This flexural test would give a severe test of the bonding power of a repair material and would test the bond in a mode that is likely to be encountered by a repaired prototype structure. The test is based on the flexural tests described above and consists of casting a short beam of concrete, of a mix design as similar as possible to the marine structures being repaired, breaking that beam in a controlled manner, separating the two halves to a fixed distance, filling the gap with the repair material and, after curing, testing the resulting composite beam by four point flexural loading (Fig. 5).

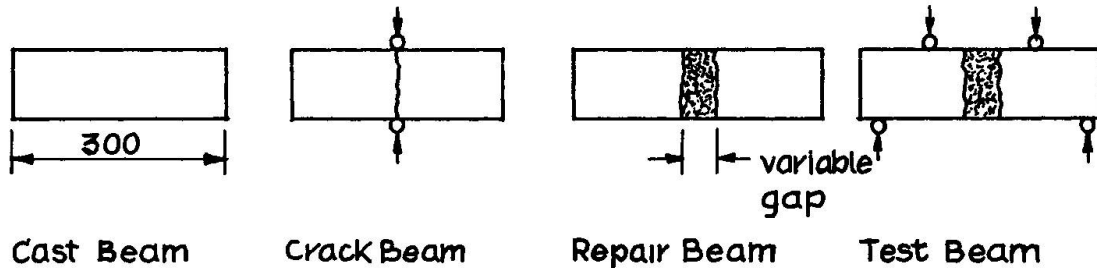


Figure 5 Recommended flexural test for resin or cementitious bonded beams

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Korrosionsstop in Stahlbetonwänden durch Wärmedämmsysteme

Arresting Corrosion in Walls by Means of Thermal Insulation Systems

Arrêt de la corrosion dans des murs par une isolation thermique

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ZUSAMMENFASSUNG

In diesem Beitrag wird dargestellt, wie durch Aufbringen geeigneter Wärmedämmsysteme die Stahlbeton-aussenwände von Gebäuden dauerhaft trocken gehalten werden können. Es ist einsichtig, dass in trockenem Beton kein Elektrolyt vorhanden ist, der zur Bewehrungskorrosion zwingend erforderlich ist. Folgerichtig kann auf eine konventionelle Betoninstandsetzung verzichtet werden. Die bisher durchgeführten Untersuchungen bestätigen dieses Verfahren.

SUMMARY

This paper shows how it is possible to keep dry the outer walls of buildings made of reinforced concrete by means of thermal insulation systems. It stands to reason that there is no electrolyte in dry concrete, which is really necessary for rebar corrosion. Consequently there is no need for conventional concrete repair. Tests made so far confirm this method.

RÉSUMÉ

Il est possible de 'garder au sec' des murs extérieurs de bâtiments, fabriqués en béton armé, par une isolation thermique. Il est évident qu'il n'y a pas d'électrolyte dans le béton sec, présence nécessaire à la corrosion de l'armature. Par conséquent, il est possible de renoncer à une réparation usuelle du béton. Les essais conduits jusqu'à présent confirment la méthode.



1. ZWECK UND ZIEL DES BEITRAGES

Aufgrund ungenügender Betondeckung der Bewehrung zeigen Hochbauten aus Stahlbeton Schäden durch Bewehrungskorrosion in den äußeren Schichten der Außenbauteile. Die Instandsetzung der Wände nach dem heutigen Stand der Technik ist aufgrund der Vielzahl der erforderlichen Arbeitsschritte fehleranfällig und teuer, Fig.1 [1].

Im folgenden soll gezeigt werden, wie durch Aufbringen geeigneter Wärmedämmsysteme die gesamte Stahlbeton-Außenwandkonstruktion dauerhaft so trocken gehalten werden kann, daß auch ohne eine konventionelle Betoninstandsetzung eine fortschreitende Korrosion der Bewehrung vermieden wird.

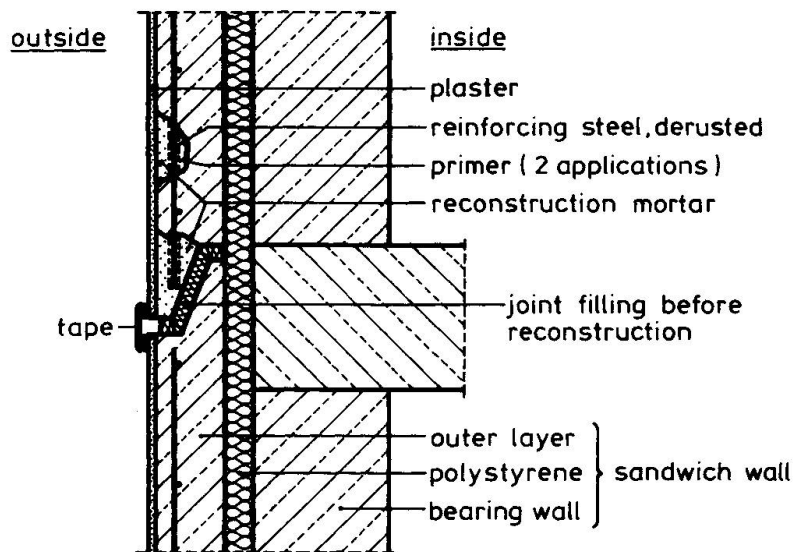


Fig.1 Konventionelle Instandsetzung geschädigter Betonoberflächen

2. BAUPHYSIKALISCHE GRUNDLAGEN

Zur Stahlkorrosion in Beton müssen folgende drei Voraussetzungen gleichzeitig erfüllt sein:

- Sauerstoff muß zutreten können,
- die Passivierung der Betonstahloberfläche muß aufgehoben sein (durch Karbonatisierung oder - im Hochbau seltene - schädliche Salze) und
- der Beton muß zur Elektrolytbildung ausreichend feucht sein.

Fehlt eine der genannten Bedingungen, entsteht keine Korrosion.

Der Zutritt von Sauerstoff kann bei diffusionsoffenen Baustoffen wie Beton z.B. direkt am Stahl verhindert werden, indem eine dichte Kunststoffbeschichtung auf den Stahl aufgebracht wird (konventionelle Betoninstandsetzung). Diese Art des Korrosionsschutzes wird durch Aufbringen einer Wärmedämmung nicht erreicht.

Die Karbonatisierung des Betons ist ein unvermeidlicher Vorgang, der in der Regel sehr langsam verläuft [2]. Sobald Bewehrungskorrosion jedoch sichtbar geworden ist, ist auch die Passivierung der Betonstahloberflächen aufgehoben. Durch Aufbringen einer Wärmedämmung wird die Bewehrung nicht passiviert.

Als dritte Möglichkeit der Korrosionshemmung bleibt noch, den Beton dauerhaft so trocken zu halten, daß mangels ausreichendem Elektrolyten keine Betonstahl-

korrosion stattfinden kann. Bisherige Berechnungen und Untersuchungen an Betonsandwichwänden haben gezeigt, daß durch ein nachträglich aufgebrachtes Wärmedämmsystem z.B. entsprechend Fig.2

- die Feuchte in der statisch tragenden Wand geringfügig weiter absinkt und
- insbesondere die Feuchte in der Vorsatzschale deutlich von zeitweiliger Durchfeuchtung auf Werte sinkt, die sogar die Feuchten in der Tragschale unterschreiten.

Da die Vorsatzschale nach Aufbringen eines Wärmedämmsystems keine Schlagregenbeanspruchung mehr erhält, wird die Bewehrung somit bei normal genutzten Innenräumen nicht weiter korrodieren.

Beobachtungen an ausgeführten Bauten bestätigen, daß Stahlbetonwände im Wohnungsbau und in vergleichbar genutzten Gebäuden an den zum Innenraum hin liegenden Wandoberflächen in der Regel erheblich karbonatisiert sind, dennoch wurden dort keine Korrosionsschäden beobachtet. Der Grund hierfür ist, daß Wohn- oder Büroräume nicht die zur Korrosion erforderliche Luftfeuchte aufweisen.

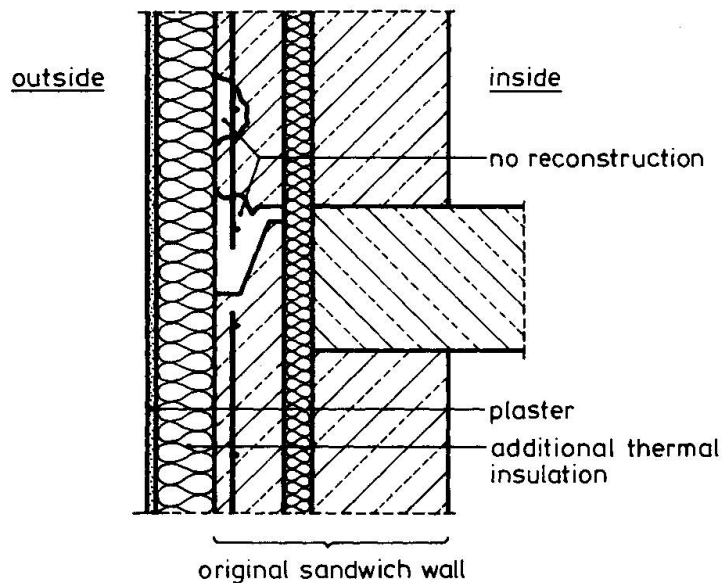


Fig.2 Stahlbetonaußenwand ohne konventionelle Betoninstandsetzung, aber mit nachträglich aufgebrachtem Wärmedämmverbundsystem

3. DURCHFÜHRTE UNTERSUCHUNGEN

Zur Absicherung des im vorigen Abschnitt entwickelten Ansatzes werden folgende drei Schritte durchgeführt:

- In Felduntersuchungen wird unter Praxisbedingungen die Feuchte in nachträglich gedämmten Betonsandwichwänden bestimmt.
- In Laboruntersuchungen wird ermittelt, ab welcher Umgebungsfeuchte Betonstahl in karbonatisiertem Beton korrodiert.
- An geschädigten Betonsandwichwänden wird die Korrosionshemmung unter Wärmedämmsystemen praktisch überprüft.

Liegt die im ersten Schritt ermittelte, praktische Feuchte in nachträglich gedämmten Betonsandwichwänden unter der im zweiten Schritt bestimmten Grenzfeuchte, wäre erwiesen, daß der Korrosionsprozeß aufgehalten wird. Der dritte Schritt soll als Bestätigung der vorgeschlagenen Maßnahmen dienen.



3.1 Temperatur- und Feuchtemessungen an Großtafelbauten

Zur praktischen Überprüfung der o.g. theoretischen Überlegungen werden Temperatur- und Feuchteverläufe in nachträglich wärmegeämmten Betonsandwichwänden über längere Zeit aufgezeichnet. Zur nachträglichen Wärmedämmung wurden für diese Untersuchung die in Tabelle 1 zusammengestellten Systeme aufgebracht.

Nr.	Wärmedämmsystem	Wärmedämmung	Witterungsschutz
1	Nullversuch	-	-
2	Polystyrol-Kunstharz-Wärmedämmverbundsystem	Polystyrolhartschaum	Kunstharzputz
3	mineralisches Wärmedämmverbundsystem	Mineralfaserplatten	Kalkputz auf Fasersilikatplatten
4	Belüftete Vorhangfassade	Mineralfaserplatten	Faserzementtafeln

Tabelle 1 Untersuchte Wärmedämmsysteme

Dazu wurden kombinierte Meßfühler aus Pt100-Widerstandsthermometern und kapazitiven Feuchtefühlern

- in einer Thermometerhütte in der Außenluft,
- in den Vorsatz- und Tragschalen der Außenwandelemente und
- in den dahinterliegenden, genutzten Wohnräumen

installiert; die von ihnen gemessenen Temperaturen und Gleichgewichts-Luftfeuchten werden regelmäßig automatisch aufgezeichnet. Die Auswertung der Messungen im Jahr 1988 zeigen Fig.3 und Fig.4 beispielhaft für Betonsandwichwände mit nachträglich aufgebrachtem mineralischen Wärmedämmverbundsystem bzw. Vorhangfassade; deutlich ist die Austrocknung beider Schalen zu erkennen.

3.2 Laboruntersuchungen an klimatisierten Betonstählen

Um mit den gewonnenen Feuchtemeßwerten Aussagen über das Korrosionsverhalten machen zu können, werden ungeschützte Betonstähle und künstlich karbonatisierte Stahlbetonproben bei verschiedenen Feuchten klimatisiert und auf ihren Korrosionsabtrag untersucht.

3.2.1 Atmosphärische Korrosion von Betonstahl

Der Grad der atmosphärischen Korrosion hängt von der Zusammensetzung der Umgebungsluft und deren relativer Luftfeuchte ab. Um Vergleichswerte für die Korrosion in karbonatisiertem Beton zu erhalten, wurden übliche Betonstähle in Berliner Industriemosphäre verschiedenen Luftfeuchten ausgesetzt. Die Ergebnisse sind in Fig.5 dargestellt; erst ab Luftfeuchten von ca. 60 % zeigen sich meßbare flächenbezogene Massenverluste m_p [3].

3.2.2 Betonstahlkorrosion in karbonatisierten Betonproben

Der Korrosionsprozeß von Stahl in karbonatisiertem Beton verläuft voraussichtlich anders als in der Atmosphäre, da

- der Sauerstoffzutritt erschwert ist,
- auch karbonatisierter Beton einen höheren pH-Wert hat als die übliche Industriemosphäre und
- die Elektrolytbildung sich im hygroskopischen Beton von der auf der ungeschützten Stahloberfläche unterscheidet.

Daher werden z.Z. karbonatisierte Betonproben mit verschiedenen Betonstählen bei 60, 70, 80 und 90 % relativer Luftfeuchte klimatisiert, nach festgelegten Zeiten geöffnet und die eingebetteten Stähle auf ihren Korrosionsabtrag untersucht.

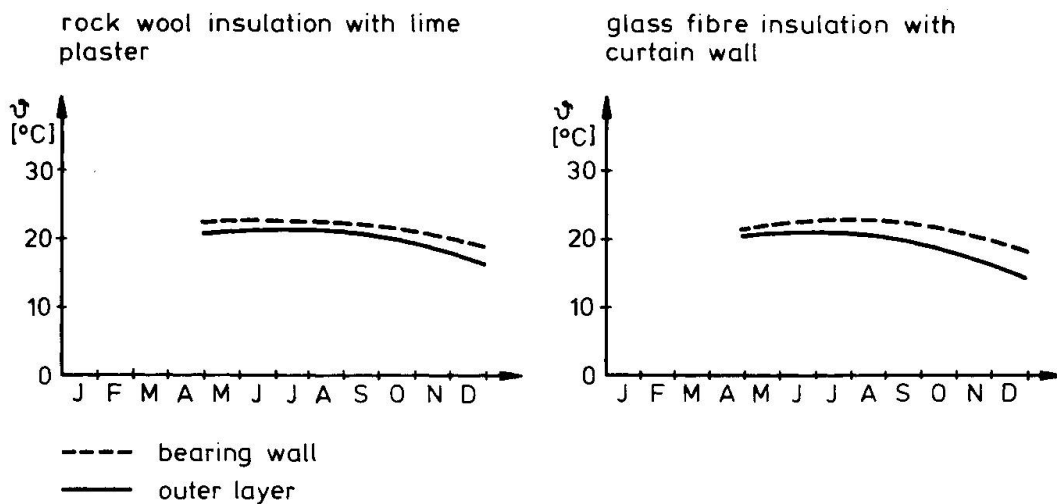


Fig.3 Temperaturverläufe in nachträglich wärmegeämmten Betonsandwichwänden, links mit mineralischem Wärmedämmverbundsystem, rechts mit Vorhangfassade (geglättete Meßwerte 1988)

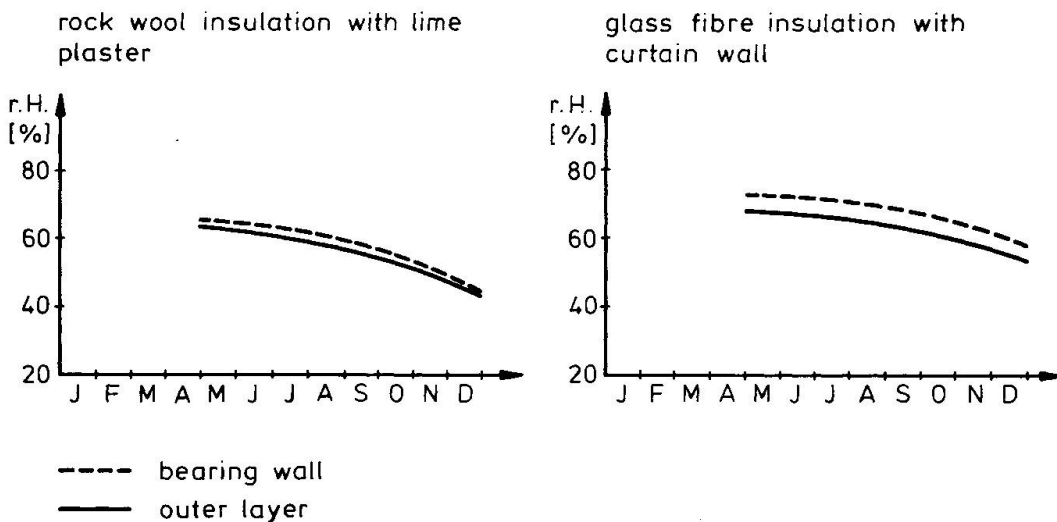


Fig.4 Verläufe der Gleichgewichts-Luftfeuchten in nachträglich wärmegeämmten Betonsandwichwänden, links mit mineralischem Wärmedämmverbundsystem, rechts mit Vorhangfassade (geglättete Meßwerte 1988)

4. AUSBLICK

Es wurde zuerst die Hypothese entwickelt, daß durch Aufbringen geeigneter Wärmedämmssysteme Stahlbetonaußenwände von üblich genutzten Hochbauten dauerhaft so trocken gehalten werden können, daß auf eine konventionelle Betoninstandsetzung verzichtet werden kann. Der Grund hierfür liegt darin, daß in trockenem Beton der Elektrolyt fehlt, ohne den eine Korrosion der Bewehrung unmöglich ist.



Die bisherigen Forschungsergebnisse bestätigen diesen Ansatz:

- Bewehrungsstähle korrodieren erst bei relativen Luftfeuchten von mehr als ca. 60 %.
- Durch das Aufbringen einer Wärmedämmung auf die Stahlbetonaußenwände wird die Feuchte in den Wänden unter den kritischen Wert abgesenkt, so daß keine Korrosion möglich ist.

Weitergehende Untersuchungen werden durchgeführt hinsichtlich

- längerfristiger Erfassung des Austrocknungsverhaltens,
- Korrosionsverhalten von Betonstahl in karbonatisiertem Beton und
- praktischer Überprüfung der Korrosionshemmung unter verschiedenen Wärmedämmsystemen.

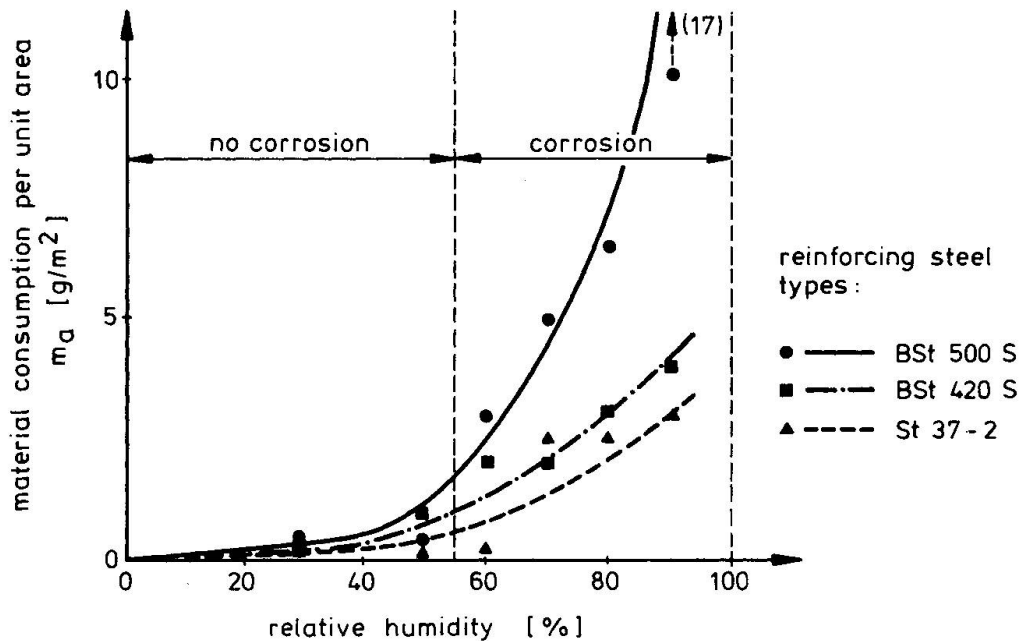


Fig. 5 Flächenbezogene Massenverluste m_a bei atmosphärischer Korrosion in Industrielatmosphäre nach ca. 40 Tagen Lagerung bei verschiedenen relativen Luftfeuchten

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