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Global Strategy to Enhance Service Life of Deteriorating Structures

Stratégie globale pour l'amélioration de la durée de service d'ouvrages détériorés

Umfassende Strategie für die Verbesserung der Lebensdauer von zerstörten Bauwerken

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SUMMARY

This paper presents a strategy to protect and strengthen deteriorating concrete structures and enhance their service life. A dual approach involving an acrylic rubber protective coating to prevent intrusion of aggressive agents such as chlorides and carbonation, and strengthening by bonding plates as external reinforcement can jointly rehabilitate such structures. These operations, however, need an integrated design strategy that interrelates material properties and structural performance and combines engineering science, technology and systems approaches.

RÉSUMÉ

L'article présente une stratégie pour protéger et renforcer des structures en béton abîmées, afin de prolonger leur vie. Il propose une combinaison de deux méthodes, c'est-à-dire, un revêtement en caoutchouc acrylique pour prévenir l'entrée d'agents corrosifs, par exemple, le sel et l'acide carbonique, et un renforcement extérieur comprenant des plaques collées. Ces opérations requièrent une stratégie de projet, intégrant les propriétés des matériaux ainsi que la performance désirée des structures.

ZUSAMMENFASSUNG

Dieser Aufsatz beschreibt eine Strategie zum verbesserten Schutz und zur Verstärkung von geschwächten Betonkonstruktionen, die zu einer verlängerten Lebensdauer führen kann. Eine Kombination zweier Methoden, das heisst, einesteils die Beschichtung der Oberflächen mit einem Acrylgummi zur Verhinderung des Eindringens aggressiver Stoffe (Salze und Kohlensäure) und andernteils die Verstärkung durch äusserlich aufgeklebte Platten, kann es erlauben, solche Bauwerke zu rehabilitieren. Diese Ansätze verlangen allerdings nach einer integrierten Konstruktionsstrategie, die Werkstoffeigenschaften und bautechnische Grundlagen zusammen berücksichtigt.

1. INTRODUCTION

In many respects the concrete construction industry is now at cross-roads. It faces a major two-pronged challenge – the need to preserve and extend the durable service life of existing deteriorating or otherwise structurally inadequate structures, and the capability to build new ones that are more durable and will require much less repair and retrofitting with the passage of time.

The paradox of concrete is that whilst being inherently durable and protective to steel through its alkalinity, the nature of the material and construction technology are such that sooner or later it will permit the ingress of deleterious elements which could destroy the electrochemical stability of steel and the integrity of the concrete. Concrete will set, harden and carry loads to a great degree even if it is not made with the correct constituents and proportions or not placed and cured properly. Its excellent performance in normal environments has led to the assumption that the material virtually needs no maintenance, that it will not deteriorate, and that the impermeability of concrete and protection of the embedded steel are somehow automatically and adequately catered for by the cover thickness and the presumed quality of concrete. Experience has shown that neither can be achieved as a normal and natural consequence of the process of concrete fabrication, and that deicing salts, marine exposure and aggressive salt-laden environments in severe climatic conditions can all lead to premature deterioration and loss of serviceability, strength and safety.

Extending the life span of deteriorating concrete structures would then involve two processes. One relates to stopping the deterioration process which may involve one or more mechanisms such as chloride/sulphate intrusion, moisture penetration, carbonation and/or alkali-aggregate reactivity. The second involves structural rehabilitation to correct deficiencies and to enhance structural stiffness, serviceability life, safety and total load capacity. These two processes are interactive and inter-related, and often there is no simple solution, no single approach, no easy magic formula that will ensure long-term trouble-free service life when concrete is exposed to elements that are known to damage it.

This paper advocates an integrated, global ***Design Strategy*** which would involve a thorough understanding of material behaviour and structural performance and which would unify engineering science, technology and systems approach in tackling engineering problems [1].

2. PROTECTION OF CONCRETE

2.1 Need for Protection

Concrete exposed to an unfriendly environment will suffer a slow, steady but continual and progressive process of deterioration. Long experience of material and structural degradation has taught us that such deterioration is an overall, synergistic and time-dependent process, a complex combination of many individual mechanisms, the exact role, effect and contribution of each of which to the totality of damage is not clearly understood. The mechanism of the development of strength and a tight, refined pore structure in concrete is also a time-dependent process so that there is always the possibility and opportunity for aggressive agents to permeate concrete, particularly at the very early ages of its life. Further, concrete is a material of low tensile strain capacity, of the order of 150–200 microstrains, such that microcracking will always occur in some form or other, providing another means for deleterious agents to penetrate concrete.

2.2 Acrylic Rubber (AR) Surface Coating

The most effective solution to enable concrete to develop its strength and, more importantly, a closed pore structure, and at the same time cut off the transportation path of chlorides, sulphates and other deleterious ions is to use a concrete surface coating which will act as a curing/durability agent. An "***Acrylic Rubber Coating***" developed by the authors provides such a protective system [2]. To be effective such a coating should not only have adequate diffusion characteristics, but also possess excellent engineering properties in terms of tensile strain capacity, elasticity, thermal stability and bonding strength, as well as weathering resistance. The only test and evidence of the long-term capability of a surface coating to maintain its stability and integrity is its performance characteristics in an aggressive environment [3].

Fig. 1 shows the extent of corrosion suffered by steel bars embedded in 200x200x300 mm uncoated and coated prisms of concrete without and with NaCl contamination and partially immersed in sea water. The results show the ability of AR coating in preventing the penetration of chlorides from the surrounding sea water and also effectively and significantly reducing the damaging effects of the trapped slat within the concrete. This AR coating can also act as a water-proofing material, as confirmed in Fig. 2, which shows the loss in flexural strength of concrete prisms containing alkali-reactive aggregates and subjected to outside exposure. Although the coating did not prevent the expansive alkali silica reactivity fully (for reasons not discussed here), the strength loss was much less than that in specimens coated with an epoxy resin, or in uncoated specimens.

The total ability of the AR coating in controlling chloride penetration into concrete is illustrated in Fig. 3 which shows chloride distribution in uncoated and coated reinforced concrete beams made with 0.5% salt added during mixing and built in an aggressive marine environment with sub-tropical weather conditions of high temperature, humidity and wind, and exposed to salt-laden breeze and sea water splash. The performance data over 8 years is ample proof of the role and effectiveness of the coating and its ability to maintain its continuity and integrity.

Penetration of carbonation is a time-dependent process, and the effectiveness of the AR coating in controlling its penetration is illustrated in Table 1 which shows the depth of carbonation obtained from core tests drilled from four structures, located at distances of 0.1 to 10 km from the sea, and which had been coated as part of a rehabilitation process. The data in the table show that the coating has not only prevented the progress of further carbonation, but also enabled a "***realkalisation process***" of the carbonated concrete. This is a unique property of the AR coating.

3. STRUCTURAL REHABILITATION

Structural inadequacy may arise for a number of reasons such as material degradation, deficiencies in design or construction, or merely the need to upgrade load capacity necessitated, for example, by heavier axle loads in bridges. Of all the techniques available for structural strengthening, the plate bonding technique – which consists of providing external reinforcement in the form of adhesive–bonded steel plates – provides a structurally efficient, technically sound and economically efficient method of enhancing both the serviceability load and ultimate load carrying capacity of the member. The practical advantages of the technique are the comparative ease and economy with which the operations can be carried out, and the strengthening operations can be carried out when the structure is still in use and loaded. The engineering merits of the method are that there are no major redistribution of existing stresses, and the changes in overall structural sizes are very small.

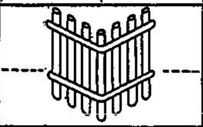
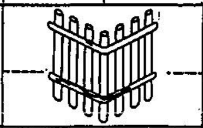
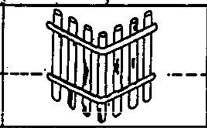
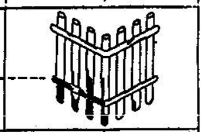
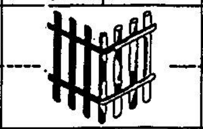
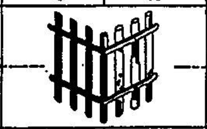
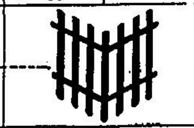
Initial salt, %			0		0.2		0.8		1.0	
Concrete cover, mm			20	30	20	30	20	30	20	30
AW coated	Appearance	In air / sea water								
	Corroded Area(%)	φ 16mm	0	0	5	0	25	10	30	15
		Links	0	0	60	20	70	40	80	50
Non coated	Appearance	In air / sea water								
	Corroded Area(%)	φ 16mm	100	80	100	80	100	90	100	100
		Links	100	100	100	85	100	70	100	100

Fig. 1 Effect of AR coating on steel corrosion after 8 years' exposure

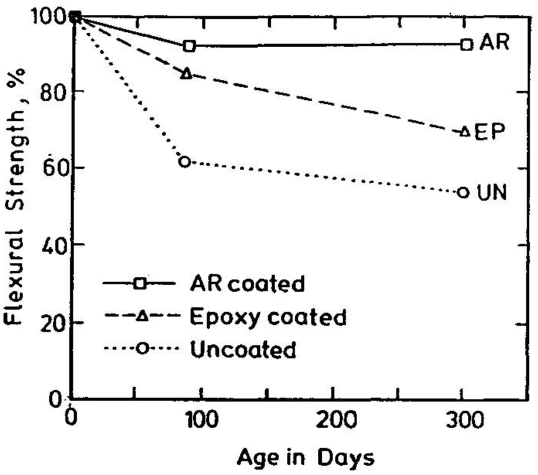


Fig. 2 Loss of flexural strength due to ASR expansion

Structure	History			Carbonation depth, mm		
	Con-struction	Repair	Investigation	Before Repair	After Repair	
				Uncoated	Uncoated	AR Coated
Building A	1968	1979	1987	12.5	16.4	7.1
Building B	1973	1981	1987	9.8	13.0	8.0
Building C	1961	1983	1987	27.6	30.0	0.0
Building D	1958	1981	1987	22.3	25.0	1.0

Table 1 Effect of AR coating on carbonation

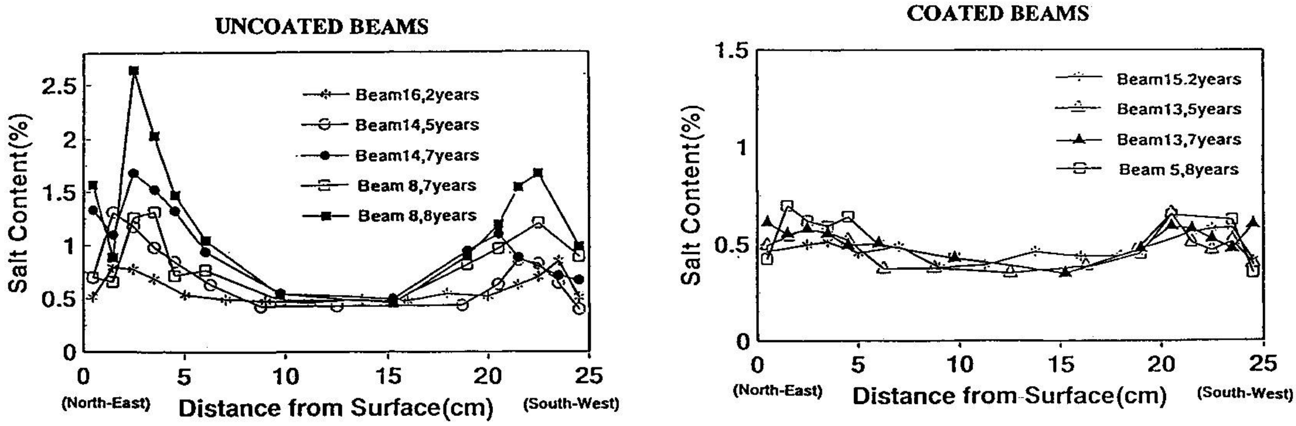


Fig. 3 Chloride distribution in beams with 0.5% salt

With the correct choice of adhesives and plate geometry, and taking care of the anchorage stresses at the ends of the plates, design guidelines are now available to ensure that the concrete–adhesive–plate composite system can develop full composite action at all stages of loading and fail in a ductile manner, after attaining full flexural strength [4,5].

3.1 Structural Benefits

The provision of a steel plate at the tension face of a beam significantly reduces both deflections and crack widths, more than what would be achieved by additional internal reinforcement equivalent to that of the external plate, (Figs 4 and 5). The stiffening and crack controlling action of the plates can lead to substantial increases in serviceability loads for various serviceability conditions as shown in Table 2. Plating structurally damaged beams can also lead to substantial reductions in deformations and increases in serviceability loads, as shown in Table 3.

Several practical applications have shown that the technique of plate bonding can provide a cost–effective method of enhancing the life span of structurally inadequate members. The technology can be extended to the use of non–metallic plates, and long–term exposure tests show that the technique is highly tolerant to even fairly large areas of deterioration of the plate–adhesive system without damage to structural performance.

4. CONCLUSIONS

The philosophy put forward in this paper is that engineering operations need an integrated global "***Design Strategy***" that interrelates material properties to structural performance and is based on the concept of "***Structural Integrity***" which combines engineering science, technology and systems approach. Material characteristics have an unpredictable influence on structural behaviour, but microstructural properties cannot directly be extrapolated to engineering performance. An integrated design approach – a "***holistic***" approach – can help us to design structures that show long-term and durable service life.

REFERENCES

1. SWAMY, R.N., Design – The Key to Concrete Material Durability and Structural Integrity.

2. SWAMY, R.N., and TANIKAWA, S., An External Surface Coating to Protect Concrete and Steel from Aggressive Environments, RILEM Materials and Structures, Oct. 1993.

3. OSHIRO, T., SWAMY, R.N., and TANIKAWA, S., In-situ Evaluation of a Flexible Surface Coating for Concrete to Prevent Chloride Penetration and Steel Corrosion, ACI Publn. SP-145, 1994.

4. SWAMY, R.N., JONES, R., and BLOXHAM, J., Structural Behaviour of Reinforced Concrete Beams Strengthened with Epoxy-bonded Steel Plates. The Structural Engineer. Feb. 1987.

5. SWAMY, R.N., JONES, R., and CHARIF, A., The Effect of External Plate Reinforcement on the Strengthening of Structurally Damaged R.C. Beams. The Structural Engineer, Feb. 1989.

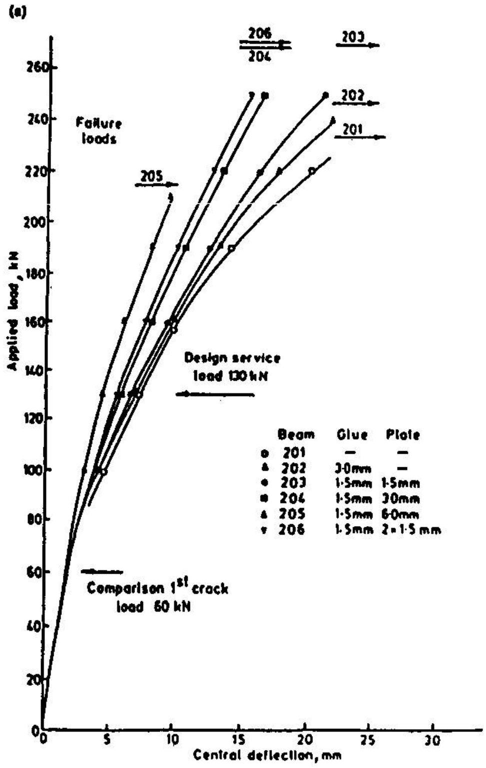


Fig. 4 **Effect of bonded plates on mid-span deflection**

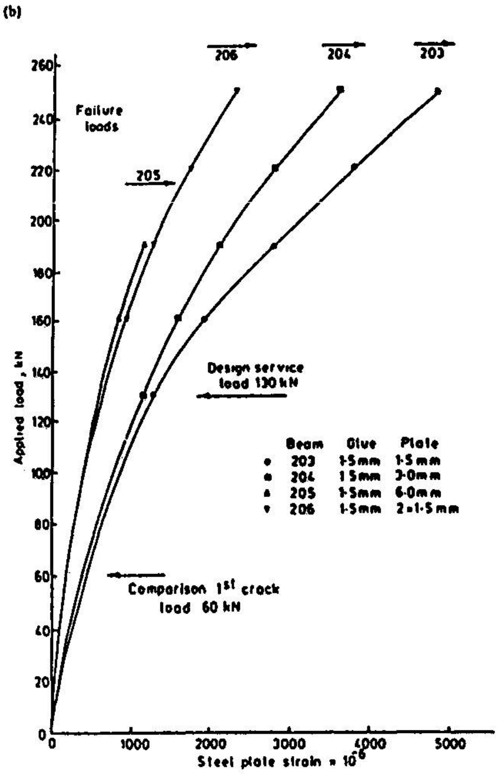


Fig. 5 **Effect of bonded plates on steel plate strain**

	Plate Thickness		
	1.5 mm	3.0 mm	6.0 mm
Adhesive thickness	1.5-6.0 mm	1.5-6.0 mm	1.5-6.0 mm
Deflection	4-11%	10-26%	30-36%
Rotation	6-17%	23-37%	38-56%
Max. crack width	37-53%	38-60%	70-99%
Steel bar strain	16-43%	50-72%	70-110%

Table 2 **Increase in serviceability loads of plated beams**

	Plate thickness	
	1.5 mm	3.0 mm
Adhesive thickness	3.0 mm	3.0 mm
Deflection	0%	20%
Rotation	8-20%	33%
Max. crack width	23%	60%
Steel bar strain	40-43%	72%

Table 3 **Increase in serviceability loads of cracked and plated beams**

Forecasting the Condition of a Reinforced Concrete Structure under Corrosion

Prévision de l'état d'une construction en béton armé, sous corrosion

Ueberwachung und Zustandsprognose eines korrodierenden Stahlbetontragwerks

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SUMMARY

For monitoring the effects of rebar corrosion in a structure, some methods have been developed which are minimally or totally nondestructive. To determine the areas where rebars are corroding, a potential map is drawn. The extension of the corroding areas is predicted by using the results of these measurements and those of carbonation depth or chloride profile, as well.

RÉSUMÉ

Les auteurs exposent les méthodes peu ou non destructives mises au point pour détecter les effets corrosifs sur les armatures d'une construction. Ces procédés permettent d'effectuer un relevé graphique du potentiel électrique des zones d'armatures déjà corrodées. L'extension des zones de corrosion est prévue en utilisant aussi bien les résultats de ces mesures que ceux de profondeur de carbonatation ou de profil de teneur en chlorure.

ZUSAMMENFASSUNG

Zur Ueberwachung der Auswirkung von Bewehrungskorrosion in einem Tragwerk wurden einige Methoden entwickelt, die ganz oder weitgehend zerstörungsfrei ablaufen. Um die Fläche zu bestimmen, wo die Bewehrung korrodiert, wird das elektrische Potential kartographiert. Das Ausmass der geschädigten Zonen wird aus diesen Ergebnissen in Verbindung mit der Karbonatisierungstiefe und dem Chloridprofil bestimmt.

1. INTRODUCTION

Concrete cover in reinforced concrete structures can undergo several damaging actions. For example, freezing and thawing cycles result in physical effects (temperature gradient) and chemical actions (chloride effects). Reinforcing steel in these structures can also be deteriorated by chemical actions, mainly due to chlorides and carbon dioxide.

The corrosion process of rebar includes two stages. For the first stage, aggresive reagents, such as chlorides, penetrate through the concrete cover but rebars are not yet corroding. The second stage starts when the amount of aggressive reagents contacting steel, reaches a critical value.

So, it is of importance to monitor corroding rebars, by applying non-destructive methods, before significant deteriorations are visible on the concrete surface. If corroding rebars are detected, the deteriorated areas can be well located and eventual repairs can be made at low costs. This principle has been applied to the following procedure which has been developed by Laboratoires des Ponts et Chaussées (LPC - France).

2. MONITORING THE CONDITION OF THE STRUCTURE

2.1 : Potential mapping

Potential map is drawn by measuring the half-cell potential E of rebar against a reference electrode, placed at various locations on the structure surface. This monitoring technique is now common. The distance between the nearest checking points is as short as possible. For the French LPC laboratories this length is of about 25 mm. This method is non-destructive when some rebar are no more covered and can be connected to the measurement circuit. If no rebar is visible, a small hole is drilled for connecting the rebar mat to the voltmeter.

The interpretation of the potential E values can be difficult. For example, it appeared that in a reinforced concrete elements in tidal zone, the rebar half-cell potential depends on the tide height and then the amplitude of potential change is about 100 mV. So, potential maps are used only for determining the areas where rebar half-cell potentials E are of the same order of magnitude, i.e. for example, E lower than -350 mV_{CSE}, or E higher than -200 mV_{CSE}. When the E value is very low, rebar is likely corroding. But, in that case, a small hole is drilled for examining the actual state of this rebar. Thus, it may be asserted that rebars are corroding in such areas. It is of a great importance to determine the locations and the extents of the areas where rebars are corroding, because such areas are more dangerous if they are in tensioned members of the structures.

More commonly, potential maps are used to determine the parts of a structure where repairs are to be made. They give no information on rebar corrosion rate.

Figure 1 gives an example of potential mapping. It deals with a bridge, in mountain area, on which deicing salts have been applied. This map shows that the corroding rebars are also in the lower part of the members, near their ends. The reason of this deterioration was that the waterproof membrane (on the bridge deck) was not effective at its ends.

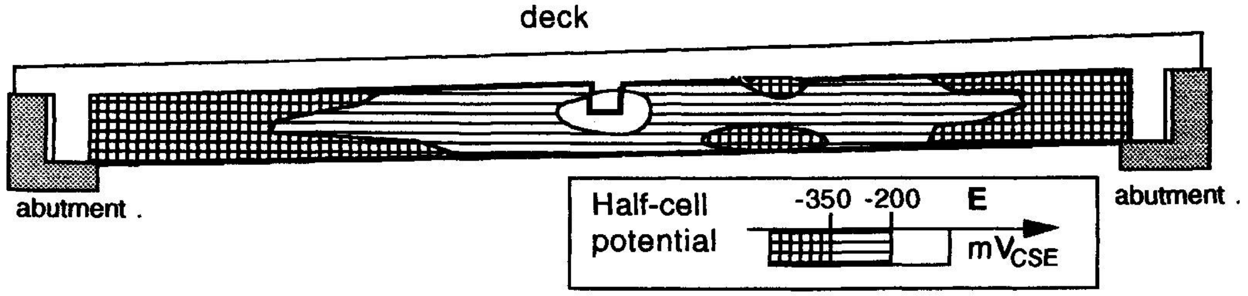


Fig. 1 : Example of potential map of a bridge member, about 7 m long and 0.5 m high.

2.2 : Carbonation depth

As far as rebar corrosion is concerned, the most common aggressive reagent is, in France, carbon dioxide. It means that carbonation deteriorates French concrete structures, more often than chlorides do. This gaseous atmospherical compound can decrease the pH of the concrete cover (carbonation process) and induce rebar corrosion. So, it is of importance to know how deep carbon dioxide has penetrated into the concrete cover around a rebar.

Carbonation depth is determined by using color indicator (phenol phtaleine). This measurement can be almost non destructive if the fresh surface on which the indicator is applied, is obtained with a rotating saw. However, such a slot must be correctly made, by a skilled personel. The value of the carbonation depth X_c is to be compared with the concrete cover thickness. It means that if the whole concrete cover is carbonated and if the relative humidity around the structure is not too low, rebars are likely corroding.

Another type of result can be obtained by measuring carbonation depth X_c at various ages t. Such measurements give the effective diffusion coefficient D' of carbon dioxide into concrete, according to the formula :

$$X_c = \sqrt{D' t} \qquad (1)$$

Diffusivity D' can be used for predicting the extension of damaged concrete area, as stated later.

2.3 : Chloride profile

Chloride ions in concrete cover can depassivate rebars, when their content reaches a critical value which depends on the pH of this cover. However, only chloride ions dissolved in the hardened cement pore water can be involved in corrosion processes.

So, soluble chloride content is determined at various depths in concrete. Results are plotted in a curve (chloride profile) giving the chloride content against the concrete depth (Fig. 2). If some assumptions are met, chloride profiles can be interpreted in terms of a diffusion process. One of these assumption is that the concrete humidity shall be almost constant. But some theoretical studies [2] [3] [4] have also shown that chloride penetration into concrete can be assumed to be a diffusion process only if the concrete cover is not deteriorated during the chloride penetration.

Under these assumptions, a diffusion coefficient of chloride ion in concrete can be determined by using the chloride profile which gives chloride content $[Cl^-] = C$ versus cover depth x, at an age t,. This gives the effective diffusion coefficient D according to :

$$C(x,t) = C_0 \{1 - \mathrm{erf}(\frac{x}{2\sqrt{Dt}})\} \qquad (2)$$

where erf is "error function" and C_0 is a parameter which is sometimes called "specific" chloride content. It is equal to the highest chloride content in the concrete cover, i.e. near its surface.

Many experimentations have shown that, in practice, chloride penetration into concrete placed in a regular atmosphere, can be described as a diffusion process [5] [6].

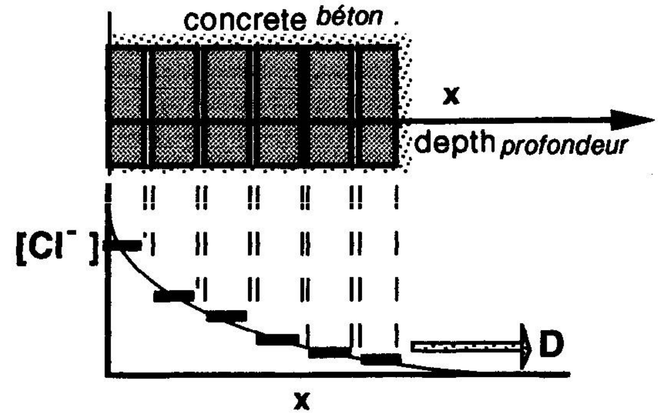


Fig. 2 :

Chloride profile which can be used for determining the chloride diffusion coefficient D.

Chloride content at depth x is in fact measured by using a concrete slice whose center is at depth x.

These slices are parts of cores drilled from the concrete cover. They are about 4mm thick

2.4 : Air permeability

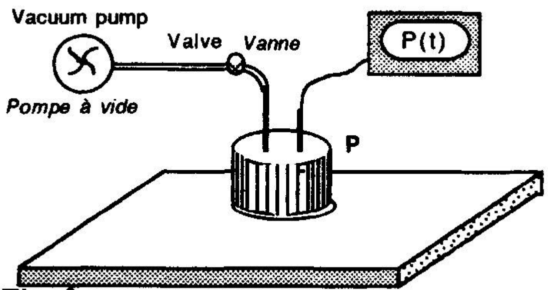
The chemical deteriorations of concrete are mainly due to aggressive reagents coming from the environment. These reagents are fluids (liquid, gas) and it is of importance to determine the permeability of the concrete cover to such fluids.

Fig. 3 gives the principle of an equipment used for determining the concrete surface permeability to air [7]. For this test, a tight vessel is placed on the concrete surface. The air in this vessel is evacuated by using a vacum pump. Then, the valve of the pump is shut and air enters back in the vessel. Its pressure P is measured in function of time t.. The curve P(t) obtained obeys the law

$$P = P_0 [1 - e^{-\frac{t + t_0}{T}}] \quad (3)$$

where t_0 and P_0 are experimental constant and parameter T characterizes the concrete permeability.

It means that parameter T makes it possible to locate areas where concrete is of high permeability (microcracks, etc) . So, this parameter is related to the risk of rebar corrosion.



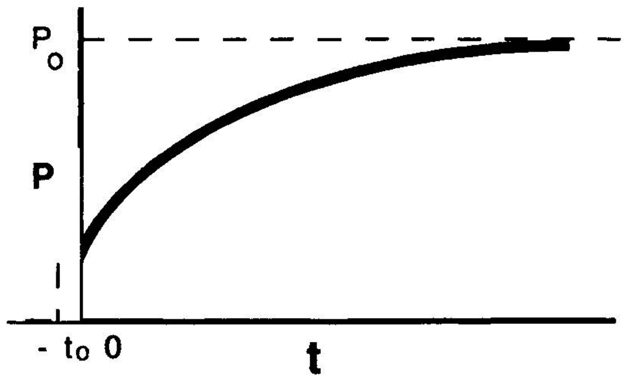


Fig. 3 :
Principle of the equipment for measuring the air permeability of a concrete surface.

3. FORECASTING THE CONDITION OF REINFORCED CONCRETE STRUCTURE

As far as the future deterioration of a reinforced concrete strcture is concerned, the following procedures make it possible to predict when a studied part of the structure must be repaired due to the rebar corrosion. It is to be noticed that visual examinations are needed to assess the validity of some test results.

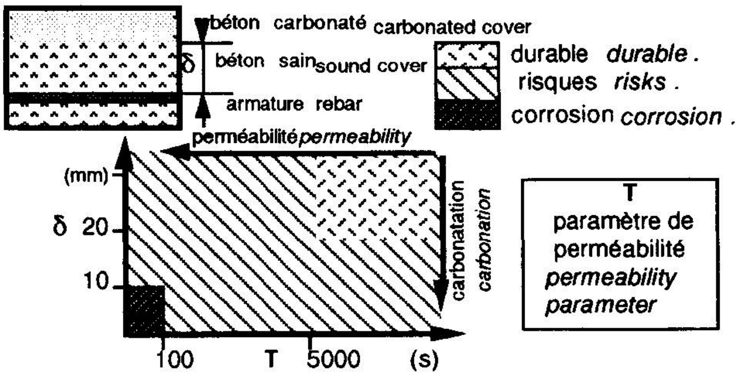
Any prediction procedure is based on test results, namely carbonation depth, chloride profile and possibly air permeability.

As mentioned here above, results of carbonation depth and air permeability measurements can predict if rebar, in the investigated area, will likely corrode [8]. Fig. 4 shows that, when the carbonation depth is thin and the air permeability low, rebar corrosion will not occur in short term. On the contrary, if the concrete cover is carbonated and if it is permeable to fluids, rebar are likely corroding.

Fig. 4 :

Procedure for predicting the corrosion of rebar by using results of carbonation depth and air permeability measurements.

Here it is assumed that no chloride is involved in the corrosion process.



This simple prediction method is no more valid, when the rebar corosion is likely due to chlorides. So, another procedure is applied in this latter case.

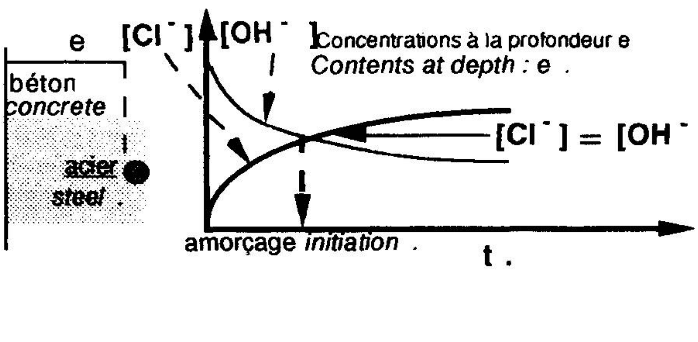
A rebar can be depassivated (corroded) when the chloride content at its level, reaches a critical value, which is equal to $[Cl^-]_{cr} = [OH^-]$ [9]. Another empirical rule states that $[Cl^-]_{cr} = 0.6\ [OH^-]$. In fact, due to the difficulties for determining the concrete pH, these two rules are equivalent.

This rule is indeed applicable both for carbon dioxide and for chloride induced corrosion. The reason is that, when concrete is carbonated (pH of about 9), the critical chloride content is very low and it is approximately equal to the cholride content in common, drinking water.

So, for predicting the corrosion of a rebar at a given location, it is necessary to know the diffusion coefficients of carbon dioxide (D') and of chloride (D), as well as the "specific" chloride content C_0. If these parameters are known, relations (1) and (2) give the future carbonation depth and chloride content at an age t. Then, prediction curves can be drawn. Fig. 5 gives examples of such curves : it indicates when rebar corrosion can start in the investigated part of the structure. In this example, the chloride and OH^- contents, at a given depth e, are calculated versus (future) time t, by using diffusion laws and measured diffusion coefficients. The crossing of theses two curves gives the date of the corrosion initiation.

Fig. 5 :

Prediction of the chloride and OH^- contents which make it possible to determine when rebar will corrode in a given part of a structure



It is to be noted that
- such a prediction is valid only if the concrete cover is not damaged by any other process (delamination, etc),
- sometimes the chloride diffusion coefficient decreases in function of time (because of the concrete change), then the prediction based on a constant diffusion coefficient is conservative.

4. CONCLUSION

The methodology developped, in Laboratoires des Ponts et Chaussées, for assessing the opportunity of a repair is now operational. It applies to structures which are under periodic survey. It means that the damaged areas in these structures are not wide spread : the rebar corrosion is often of low rate. It is not always necessary to measure this corrosion rate, because the aim of the prediction is to determine in which part of the structure and when a given part of the structure needs to be repaired.

The following concluding remarks can be drawn about this prediction methodology.

Many damages in reinforced concrete bridges are due to rebar corrosion. So, it is of importance to determine the conditon of rebar in every part of these structures. This can be achieved by drawing potential maps which show out where rebars are corroding.

If rebars are not yet corroding in some parts of the structure, it is now possible to predict the start of their corrosion. This is achieved by measuring carbonation depth and chloride profiles. So, this methodology is applicable when rebar corrosion is due to concrete carbonation and to chloride ingress, as well.

REFERENCES

1. ZAKIC B. D., RYZYNSKI A., CAI GUO-HONG, JOKELA J. Classification of damage in concrete bridges. Materials and Structures 1991, vol. 21, p. 268 - 275

2. MIDGLEY, H.G., ILLSTON J. M. The penetration of chlorides into cement pastes. Cement and Concrete Research 1984, vol. 14, p. 546 - 558

3. CADY P. D., WEYERS R. E. Chloride penetration and deterioration of concrete bridge decks, Cement ,Concrete, Aggregates, 1983, vol. 5, n° 2, p. 81 - 87

4. RAHARINAIVO A., GENIN J.M.R. Sobre la corrosion de armaduras de hormigon en presencia de cloruros Materiales de construcción 1986, vol. 36, n° 204, p. 5 - 15

5. CARPIO PEREZ J. J. "Etude de la dépassivation et de la repassivation des armatures métalliques dans les bétons" Thèse Université de Technologie de Compiègne 1991

6. WEST R. E., HIME W. G. Chloride profile in salty concrete Materials Performance 1985 (July) p. 29 - 36

7. BERISSI R., BONNET G., GRIMALDI G.Mesure de la porosité ouverte des bétons hydrauliques Proceeding First Intern. Congress "From materials science to construction materials engineering" RILEM Paris Sept. 1987 p. 119 - 122

8. RAHARINAIVO A., GRIMALDI G., CARPIO J. J. Méthodes d'évaluation de la durée de vie des éléments en béton armé. Colloque International "Détérioration des matériaux de Construction" La Rochelle 12-14 Juin 1991

9. REZEL D. Structure hyperfine et cinétique de formation des composés de corrosion humide du fer en milieu aqueux chloré : rôle de la rouille verte-I Thèse Université de Nancy I- ESSTIN 1988

Service Life Prediction of Protective Systems for Concrete Bridge Decks in Alberta

Prévision de la longévité des systèmes de protection des tabliers de ponts en béton, dans l'Alberta

Vorhersage der Lebensdauer von Schutzsystemen für Betonbrückenfahrbahnen in Alberta

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SUMMARY

This paper summarises certain aspects of deck reinforcement corrosion damage in a recently completed service life prediction study. It describes reinforced concrete deck damage as it relates to deck electrical potential levels. The paper also relates deck active half cell levels to time. Together these two relationships permit service life predictions for certain types of reinforced concrete deck systems subjected to corrosive chloride based de-cing salt environments.

RÉSUMÉ

Les auteurs résument certains aspects concernant l'endommagement des armatures des tabliers de ponts en béton. Ce rapport fait partie d'un récent examen sur les prévisions de la durée de vie, qui décrit les relations entre les dommages subis par les dalles en béton armé et les champs de différence de potentiel électrique qui y règnent. L'article donne le comportement temporel des valeurs électriques des demi-cellules actives de ces mêmes dalles. Ces observations permettent de prédire la longévité de certains types de tabliers de ponts en béton armé soumis à la corrosion du sel de dégel.

ZUSAMMENFASSUNG

Dieser Bericht ist eine Zusammenfassung bestimmter Aspekte der Beschädigung der Bewehrung in Betonfahrbahndecken. Er ist ein Teil einer kürzlich beendeten Untersuchung über Lebensdauervoraussagen und beschreibt die Beziehung, die zwischen der Beschädigung von bewehrten Betonfahrbahndecken und elektrischen Spannungsfeldem auf Fahrbahndecken besteht. Weiterhin ist beschrieben, wie sich aktive elektrische Halbzellenwerte auf Fahrbahndecken über die Zeit verhalten. Zusammen erlauben diese beiden Beobachtungen eine Lebensdauervoraussage für bestimmte Typen von Fahrbahndeckensystemen aus bewehrtem Beton, die der Korrosion durch Tausalz ausgesetzt sind.

1. INTRODUCTION

A significant number of bridge structures were introduced into the road networks of Canada and other countries influenced by freezing weather conditions during and prior to the time that roadway de-icing salts began to be used. The decks of these structures were designed and built prior to the recognition by the bridge engineering community that significant corrosion-induced damage would generally occur to the decks upon application of chloride based roadway de-icing salts. Over the last fifteen to twenty years the introduction of various new and existing technologies into deck construction, including epoxy coated reinforcement, membranes, sealers and significantly less permeable concretes, has curtailed this problem in new structures. However, the multitude of structures that pre-date these new developments have required and will continue to require maintenance and rehabilitation to offset the effects of de-icing salt induced corrosion. The study [1] that is summarized within this paper resulted in the development of models that are considered to be useful in characterizing corrosion induced damage as it relates to measurable electrical potential levels within older reinforced concrete deck systems. As well, predictive guidelines and models, that were originally developed in the study, for assessing service life of unprotected and protected deck systems used in Alberta, are presented herein. All of the relationships developed within the study were based to a large degree upon data collected by Alberta Transportation and Utilities (AT&U) pertaining to bridge structures located throughout the Province. Where data was lacking or incomplete "expert" opinion of experienced bridge engineers was employed in order to complete the development of the prediction models. Specific details about the data are presented within the original study [1].

The advent of bonded concrete overlays in the range of 50 mm to 100 mm thickness, sometimes used to enhance the service life of corrosion susceptible bridge decks, has led to overlay debonding as a second form of deck system failure, distinct from the corrosion induced mechanism. Debonding as a form of deck system failure was investigated and reported upon within the original study[1], but debonding is not reported upon within this paper.

2. BACKGROUND

2.1 Alberta's Climate and Its Impact on the Bridge System

Alberta is a landlocked province located in Western Canada (Figure 1), situated at great distance from major water bodies. Its climate is relatively dry with varying amounts of precipitation and cyclic air temperature dependent upon latitude and longitude within the province. Furthermore, the general weather patterns vary throughout the year as the climate fluctuates with four distinct seasons during the course of the year. Average daytime temperatures in the summer months are typically well above freezing, and during the winter months the average temperatures are below freezing. Spring and autumn are transitionary times of the year during which frequent freeze/thaw events occur.

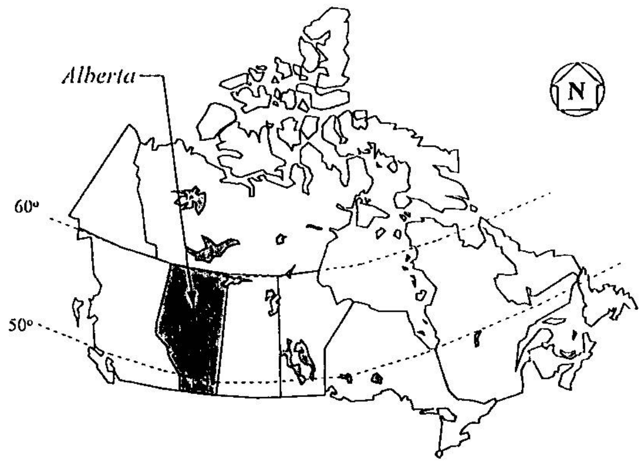

Figure 1 Map of Canada Showing Location of Alberta

One of the most negative impacts to the Alberta roadway bridge system has resulted from Alberta's freezing and thawing climate which has led to the widespread use of de-icing salts during the last 20 years, and for longer periods of time in certain portions of the Province. As it is now well known ([2] for instance), the application of chloride based de-icing salts to unprotected reinforced concrete elements generally results in reinforcement corrosion and associated concrete damage in the form of delamination and spalling. Unprotected, reinforced concrete decks, which exist on a significant number of the Alberta bridge structures, have consequently been affected by these circumstances.

2.2 Service Life and Failure

The *service life* of a bridge deck and/or its overlay, within the context of this study, is considered to be the period of time from deck and/or overlay installation to when the deck system fails to perform in an acceptable manner due to some predetermined level of damage occurring. Experience in Alberta suggests that the two major reasons for damage and ensuing failure are i) deck reinforcement corrosion resulting in delaminations and subsequent deck and/or overlay deterioration and ii) debonding of an overlay from the concrete deck. The corrosion-induced damage phenomenon is the focus of this paper.

Damage levels generally build up over time and it is considered that there is a threshold level of damage above which a deck system is considered completely failed. Complete failure is either due to the deck becoming unsafe structurally or due to it developing an unacceptably poor ridability resulting from damage levels that are too great to permit selective repairs. Levels of damage which preceed the threshold level sometimes manifest in the form of minor spalling and/or potholes and some limited amount of associated poor ridability. However, levels of damage that have proven economical to repair are often limited to internal delamination, cracking and debonding that are invisible, but which can be detected by non-destructive diagnostic methods such as chain drag delamination surveys and/or electrical potential corrosion surveys. Figure 2 depicts a generic life cycle for a bridge deck subjected to a corrosive environment and a repair application. It presents characteristic stages which are considered pertinent to this subject matter.

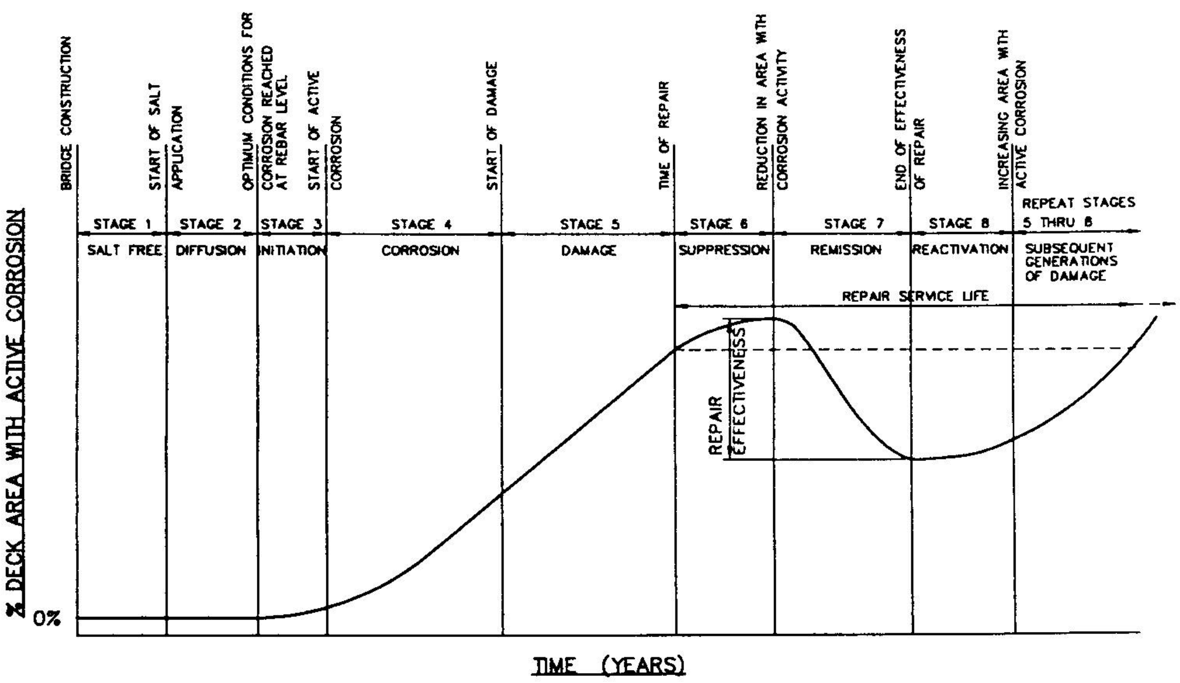

Figure 2 Bridge Deck Corrosion Life Cycle

NOTES TO FIGURE 2

Stage 1 - Salt Free - period of time from date of construction to start of de-icing salt application
Stage 2 - Diffusion - period of time from start of salting to the time that optimum conditions for reinforcement corrosion are reached
Stage 3 - Initiation - period of time to overcome protective barriers to corrosion and until active corrosion potentials can be measured
Stage 4 - Corrosion - period of time from the start of active reinforcement corrosion to when damage(spalling) is observed
Stage 5 - Damage - period of time from the start of observed corrosion induced damage to the time of repair
Stage 6 - Suppression - period of time from performing a repair or installing a protective system until reduction in deck areas of active corrosion potentials
Stage 7 - Remission - period of time during which the deck area with active corrosion potentials declines due to the effect of the repair
Stage 8 - Reactivation - the period of time during which the effectiveness of a repair begins to decline and measured deck areas with active corrosion potentials begin to increase
Stage 9 - Subsequent Generation Damage - the period of time during which deck area with active corrosion potentials is again increasing until some quantity of damage is again reached that results in another repair

3. CORROSION DAMAGE PREDICTION USING ELECTRICAL POTENTIAL SURVEYS

3.1 Relating Damage to Electrical Potentials

Based upon the premise that deck reinforcement corrosion activity can be predicted to exist by active ASTM copper/copper sulphate electrode (CSE) half cell potential readings [3], it is proposed that a mathematical relationship exists relating the area of corrosion induced damage within a bridge deck to the measured level of active CSE readings. This proposed relationship is as follows:

$$\text{Damage} = K \times \text{CSE} + A \qquad (1)$$

where Damage = % area of deck damaged due to rebar corrosion
K = a coefficient defining the rate of increase of Damage with time
A = a constant.
CSE = % total deck area with CSE readings more negative than some specified active level, such as -300 mV

It is likely that the relational coefficient "K" and the constant "A" vary depending upon both the specified level of CSE activity considered (eg. -300 mV vs. -350 mV vs. -400 mV etc.) as well as due to a series of physical and environmental characteristics that can vary between bridges, including such factors as deck reinforcement density, reinforcement cover, concrete strength/permeability, salt application rates, deck moisture levels, bridge girder type, influence of deck repairs and/or overlays.

Attempts to develop damage prediction equations that directly account for most of the many influencers were unsuccessful at this time. Comparisons of measured deck damage with measured CSE data from 66 different bridge structures repaired primarily during the 1980's, with the only variable considered being whether or not the bridge girder system was concrete or steel, resulted in the following relationships for predicting corrosion-induced damage in reinforced site cast concrete decks:

$$\text{Damage(concrete girder)} = 0.182 \times \text{CSE}_{300} - 3.90 \qquad R^2 = 0.84 \qquad (2)$$

$$\text{Damage(steel girder)} = 0.0894 \times \text{CSE}_{300} - 1.25 \qquad R^2 = 0.16 \qquad (3)$$

where CSE_{300} = the % total deck area with CSE potentials < -300 mV

The data was separated and relationships developed for the two categories of girder types because of the perception, based upon observations of experienced field personel, that the two types of bridges tend to perform differently, probably due to the inherent differences in flexibility and thermal sensitivity. Figure 3 shows equations (2) and (3) plotted, and it appears from this derivation that corrosion-induced damage begins to develop when CSE readings are more negative than -300 mV over about 10% to 25% of the deck area. Figure 3 also shows that *on average* less damage occurs in steel girder bridge decks than for concrete girder decks at a given level of CSE measured corrosion activity. It is hypothesized that the presence of the steel girder tends to increase the corrosion activity measured within the deck, but that this additional activity associated with the steel girder is not likely causing any additional damage to the deck.However, it is important to note that the variability of damage observed on steel girder decks, as represented by the reliability index, R^2, is significantly greater.

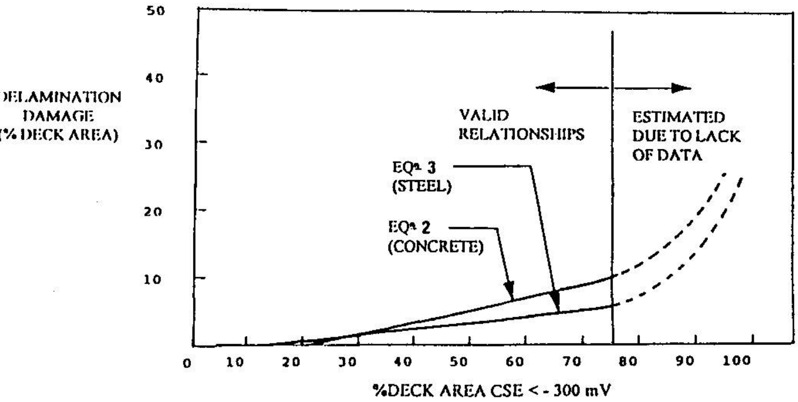

Figure 3 Damage vs.CSE , -300 mV

Equations relating Damage to average levels of CSE readings and to CSE levels more negative than -400 mV have been developed as well and are presented in the original study [1]. AT&U initiated its CSE half cell monitoring program in 1977, prior to the publication of ASTM C876. At that time, AT&U decided to select the -300 mV level to represent active corrosion. AT&U has used -300 mV consistently in its ongoing monitoring program having never adopted the corrosion activity level of -350 mV stipulated in ASTM C876.

3.2 Relating CSE Potentials to Time

A primary goal of this research was to develop service life prediction relationships that would permit the prediction of time to a defined level of deck damage, given an easily measurable deck characteristic like CSE potentials. A relationship complimentary to the Damage vs. CSE relationship (1) of the previous section is proposed to facilitate this prediction, and it relates the CSE levels in a deck system with time. The following relationship is considered to be appropriate for the majority of the corrosion life cycle during which corrosion rate of increase is considered to be linear (Stages 4 & 5 as per Figure 2):

$$CSE_{300} = C \times \text{time} + B \qquad (4)$$

where CSE_{300} = % of total deck area with CSE potentials < -300 mV
C = a coefficient defining the rate of increase of CSE_{300} with time
B = a constant.

This general relationship has been developed into a series of specific relationships applicable to various bridge deck conditions and systems through simple linear regression analysis of CSE data from several hundred bridge sites. In the data analysis process it was discovered that some bridge decks for a given protective system category performed better than other decks and so *Poor, Average* and *Good* performance sub-categories were established. Non-linear relationships have also been developed to represent Stages 3, 6, 7 and 8 of the corrosion cycle but they are not presented here (refer to [1]). Coefficients developed for use in equation (4) for three bridge deck protective systems are as follows:

PROTECTIVE SYSTEM	CORROSION STAGE	Slope C	Constant B	Reliability Index (R^2)
None - Exposed Concrete				
- Poor Performance	4&5	10.6	-104	0.83
- Average Performance	4&5	5.7	-104	0.86
- Good Performance	4&5	2.2	-57	0.12
Asphalt Covered Concrete				
- Poor Performance	4&5	11.8	-56.6	0.5
- Average Performance	4&5	3.75	-56.6	0.63
New LSDC Overlays on Concrete*				
- Poor Performance	4&5	4.60	0	.29
- Good Performance	4&5	0.80	-2.6	.61

** LSDC - Low slump dense concrete*

The sub-categories *Poor, Average* and *Good* are believed to result directly from environmental and physical characteristics unique to a bridge. Structure type, structure flexibility, traffic levels (which are likely often closely related to de-icing salt application rates) and freez/thaw cycles are all considered to influence the performance of a bridge deck. With the presently available data these factors cannot be incorporated into the prediction equations directly, and presently these important influencers are subject to some engineering judgement in the service life prediction process that is presented in the following section.

4. SERVICE LIFE PREDICTION OF DECK PROTECTION SYSTEMS

Application of the relationships presented in Sections 3.1 and 3.2, in conjunction with engineering assesment of influencing factors has resulted in the development of Table 1 Bridge Deck (Corrosion) Service Life Prediction, which lists the expected service life for site cast reinforced concrete bridge decks utilizing various protective systems presently in use in Alberta.

Two types of service life are listed within Table 1, the definitions of which have been established to be as follows:

Repairable Service Life is represented by 5% damage which corresponds to 40 to 65% of a deck having CSE_{300} levels.
Failure Service Life is represented by 60 to 80% damage which corresponds to 90 to 100% of a deck having CSE_{300} levels.

Some of the systems listed in the table have no supporting analysis at this time because sufficient data does not exist to undertake accurate analysis. It is anticipated that the data will soon be available, and the framework for analysis proposed herein and applied to the older deck systems will be appropriate to replace the listed service life predictions, now based upon expert opinion, with analytically supported predictions.

TABLE 1 BRIDGE DECK (CORROSION) SERVICE LIFE PREDICTION

PROTECTION SYSTEM	BASIC SERVICE	AVERAGE LIFE	ADJUSTMENTS TO BASIC AVERAGE SERVICE LIFE DUE TO VARIOUS INFLUENCERS									
			GIRDER TYPE		TRAFFIC INTENSITY			FREEZE THAW CYCLES		DECK DRAINAGE		
	OPTIMUM REPAIR 5% DAMAGE	FAILURE >60% DAMAGE	CONCRETE GIRDERS	STEEL GIRDERS	ADT > 5000	ADT 5000> > 600	ADT < 600	> 115 PER YEAR	< 90 PER YEAR	BAD	GOOD	EPOXY MEMBRANE
AS CONSTRUCTED												
EXPOSED CONCRETE DECKS	20	30	1	-3	-5	0	10	-3	2	-2	2	16
ASPHALT COVERED DECKS	18	32	0	-3	-7	-1	7	-4	2	-3	3	n/a
NEVER REPAIRED LSDC OVERLAY	20	40	0	-5	-6	0	12	-3	3	-3	2	10
AS REPAIRED 1												
SILICA FUME CONCRETE OVERLAY	18	35	0	-5	-5	0	10	-3	3	-3	3	15
THIN LATEX MODIFIED CONCRETE OVERLAY	10	20	0	-6	-7	0	8	-3	3	-3	3	15
MEMBRANE AND ASPHALT OVERLAY	16	28	0	-3	-5	0	7	-3	3	-4	4	n/a
50 mm LSDC OVERLAY	15	27	0	-5	-6	0	10	-3	3	-3	3	15

NOTE 1 - All "AS REPAIRED" predictions listed in this Table are derived by expert opinion and are not based upon data analysis.

ACKNOWLEDGEMENTS

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REFERENCES

1. SKEET J. and KRIVIAK G., Service Life Prediction of Protective Systems for Concrete Bridge Decks in Alberta. Alberta Transportation and Utilities Report No. ABTR/RD/RR-94/01, January 1994

2 AMERICAN CONCRETE INSTITUTE, Corrosion of Metals in Concrete. ACI Committee 222, ACI-222R-85

3. ASTM C 876-91, Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete. Annual Book of ASTM Standards, Vol 03.02, May 1991.

Arresting Corrosion in Concrete Walls Using Thermal Insulation

Arrêt de la corrosion dans les murs en béton par une isolation thermique

Korrosionshemmung in Betonwänden durch Wärmedämmung

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SUMMARY

Exterior walls of concrete buildings frequently exhibit surface deterioration due to corroding reinforcement. Laboratory and field experiments show that it is possible to arrest the rebar corrosion in exterior walls of concrete buildings in Central European or similar climates by additional thermal insulation. This method makes it possible to arrest the rebar corrosion without the usual concrete repair and at the same time to save heating energy.

RÉSUMÉ

Les murs extérieurs des bâtiments, réalisés en béton armé, sont souvent détériorés en surface par la corrosion de l'armature. Il résulte d'expériences en laboratoire et in situ qu'il est possible d'arrêter la corrosion par une isolation thermique, dans des conditions de climat d'Europe centrale ou similaires. Cette méthode rend possible l'arrêt de la corrosion sans réparation conventionnelle du béton, tout en réduisant la consommation énergétique pour le chauffage.

ZUSAMMENFASSUNG

Aussenwände von Betonbauten zeigen häufig Oberflächenschäden infolge korrodierender Bewehrung. Labor- und Felduntersuchungen ergaben, dass es möglich ist, in mitteleuropäischem oder vergleichbarem Klima durch zusätzliche Wärmedämmung die Bewehrungskorrosion in solchen Aussenwänden zu stoppen. Diese Methode ermöglicht eine Korrosionshemmung ohne konventionelle Betoninstandsetzung bei gleichzeitiger Einsparung von Heizenergie.

1. INTRODUCTION

Exterior walls made of reinforced concrete frequently show surface deterioration due to rebar corrosion. Hand-applied concrete repair of these walls requires numerous repair steps (fig. 1 left). This repair method is expensive and the results may be imperfect.

It can be demonstrated by diffusion calculations that it is possible to dry the exterior walls of concrete buildings in Central European or similar climates by attaching an additional thermal insulation to the outside of these walls. Preliminary results were reported at the IABSE symposium in Lisbon, 1989 [1]. By the undermentioned tests it will be verified that it is possible to arrest rebar corrosion by means of additional thermal insulation (fig. 1 right).

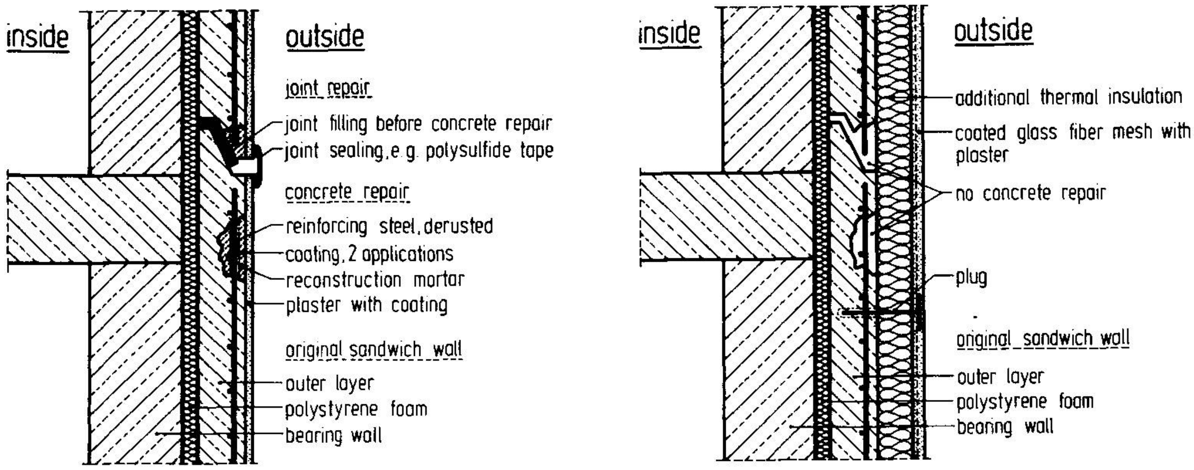

Fig. 1 Common method of concrete repair to the exterior of a sandwich wall (left), alternative repair using an external thermal insulation composite system (ETICS, right)

2. THEORY OF ARRESTING CORROSION BY THERMAL INSULATION

Steel corrosion in reinforced concrete requires that the following physical conditions occur at the same time [1, 2], if one of these conditions is missing, there will be no corrosion:

- the presence of oxygene is necessary - the concrete porosity makes the penetration of oxygene inevitable,
- the passivation of the rebar surface must be neutralized by chlorides or carbonation (the penetration of chlorides is a problem only for de-icing salt contaminated bridges and marine structures, carbonation of concrete is a slow but inavoidable process) - when deteriorations are visible the rebar surface is evidently depassivated,
- the concrete moisture must enable an electrolyte.

Thus, the only possibility of arresting rebar corrosion is to lower the concrete moisture level in a way, that there is no sufficient electrolyte. Experience with existing dwelling and office buildings in Central European climate shows that the concrete walls inside the buildings are considerably carbonated, but the rebars do not corrode there. It can be concluded that there is no electrolyte present in the dry but carbonated concrete.

Diffusion calculations demonstrate that an additional thermal insulation to the outside of a concrete sandwich exterior wall (cp. fig. 1 right) only slightly reduces the moisture of the bearing wall but significantly reduces the moisture in the outer layer of the wall. Thus, rebar corrosion may be arrested in concrete exterior walls in Central European climate by the application of thermal insulation.

3. EXPERIMENTS

3.1 Laboratory Tests

In a first step, reinforcing steel specimens were stored in industrial atmospheres of different relative humidities. Later, the mass loss (material consumption) by corrosion was investigated. The tests showed that for a relative humidity up to 50 % no corrosion occured, and that 60 % r.h. or more caused growing steel corrosion depending on the reinforcing steel type used [1, 2].

In a second step, accelerated carbonated concrete specimens with steel bars were stored in different relative humidities. Later the specimens were investigated for steel corrosion (fig. 2):

- all steel bars showed a basic corrosion caused by the carbonation of the concrete specimens,
- after four years of investigation the basic corrosion level did not change in specimens which were stored in climates with a relative humidity of 60 or 70 %, for specimens stored in a climate of 80 % r.h. the rebar corrosion increased negligibly,
- when stored in 90 % relative humidity the rebar corrosion in the specimens showed a significant growth over the time.

These experiments show that steel corrosion in reinforced concrete can be arrested if the concrete moisture level is in equilibrium to not more than 80 % relative humidity to the surrounding atmosphere, a result which does not depend on the reinforcing steel type.

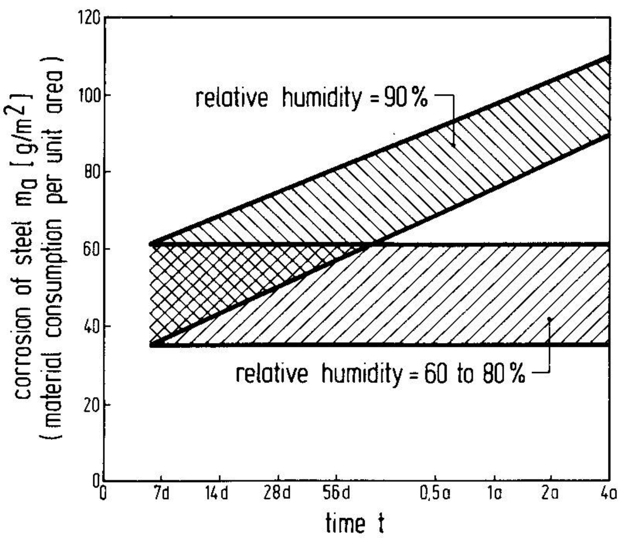

Fig. 2 Mass losses of rebars in carbonated concrete specimens stored at different r.h.

Type	Additional thermal insulation	Weather protection
1	None (zero test)	No additional
2	6 cm polystyrene foam	Polymer resin plaster
3	6 cm mineral wool	Lime plaster
4	6 cm fiberglass	Curtain wall

Table 1 Investigated types of additional thermal insulation systems

3.2 Long-Term Field Tests

To verify the above mentioned theory, long-term field tests were made on a residential building in Berlin. Two external thermal insulation composite systems (ETICS) and a curtain wall were attached to the building in September 1987 (table 1) [1, 2].

Temperatures and moistures in the concrete sandwich exterior walls, with and without the additional thermal insulation systems, were recorded for more than five years. The combined transducers for temperature and relative humidity (for measuring the equilibrium humidity) were installed as follows:

- rain and sun protected in the atmosphere,
- in the bearing walls and the outer layers of the investigated sandwich walls,
- in the used apartments behind the investigated walls [2].

The measurements were recorded automatically using a print recorder as well as a personal computer. The measured equilibrium humidities were compared with computer calculations carried out by a time-dependent water-vapor diffusion calculation program [3]:

- the outer layers of the sandwich walls were drying behind the additional thermal insulation significantly, and after a few years the concrete moisture level was in an equilibrium at the measured relative humidity of 40 to 70 % (fig. 3),
- the bearing walls were drying, too, and after a few years the concrete moisture level was in an equilibrium at the measured relative humidity of 30 to 60 %.

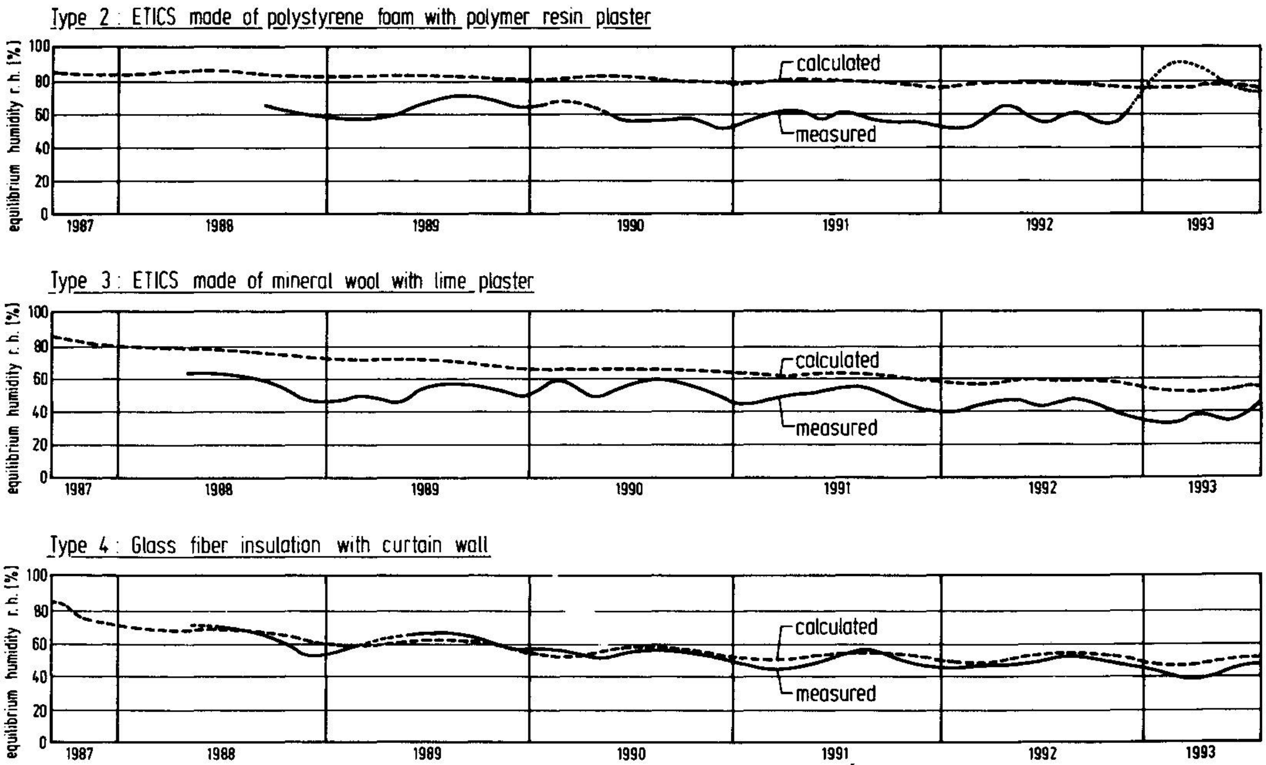

Fig. 3 Measured and calculated equilibrium humidities in the outer layers of concrete sandwich exterior walls with additional thermal insulation

The moisture levels measured in the additionally insulated sandwich walls were dependent on the type of thermal insulation and on the rain protection: Behind an ETICS made of polystyrene foam with polymer resin plaster the concrete wall was drying slowly, behind an ETICS made of mineral wool insulation with lime plaster it was drying more quickly; and behind a curtain wall with fiberglass insulation the sandwich wall was drying most quickly [2].

3.3 Practical In-Situ Tests

To investigate the reliability of the above mentioned method in practice three series of field tests were made:

- steel bars were placed in the airspace between the curtain wall and the additional fiberglass insulation (type 4, cp. table 1),
- steel bars were mounted between the different additional thermal insulations (type 2, 3 and 4, cp. table 1) and the outer layer of the original sandwich wall, too.

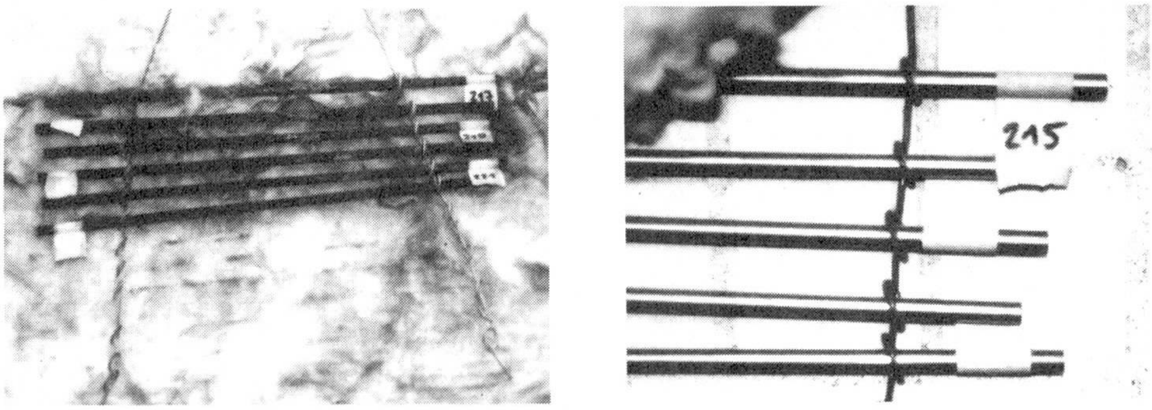

Fig. 4 Unprotected steel bars after one year of in-situ storage between a curtain wall and the fiberglass insulation behind it (left) and between the fiberglass insulation and the outer layer of the sandwich exterior wall (right)

During one year, the thermal insulation systems were opened every three months and the steel bars were examined:

- the bars which were only rain protected, i.e. in the airspace just behind the curtain wall (type 4), were clearly corroded (fig. 4 left), while the rebars mounted between the fiberglass insulation and the sandwich wall were free of any corrosion (fig. 4 right),
- the rebars mounted between the mineral wool insulation (type 3) and the sandwich wall were almost free of corrosion,
- the rebars mounted between the diffusion delaying polystyrene foam insulation (type 2) and the sandwich wall were partially corroded.

After the visual observation the mass loss (material consumption) of all steel bars was investigated. It was obvious that the thermal insulation systems of type 3 and type 4 were suitable to assess the rebar corrosion in concrete exterior walls. Type 2 seemes to be not sufficient, but in this practical test the steel bars showed corrosion in atmospheric conditions and not in realistically carbonated concrete (cp. fig. 2).

4. CONCLUSIONS

Field and laboratory tests showed that it is possible to arrest rebar corrosion in concrete walls in Central European or similar climates by the application of thermal insulation. Additional corrosion protection (as applied in common concrete repair) seems to be not necessary because

- the concrete layers of sandwich exterior walls with additional thermal insulation will get moistures in equilibrium to relative humidities of 30 to 70 % after a few years of drying,
- steel in carbonated concrete specimens needs more than 80 % relative humidity for corrosion (fig. 5).

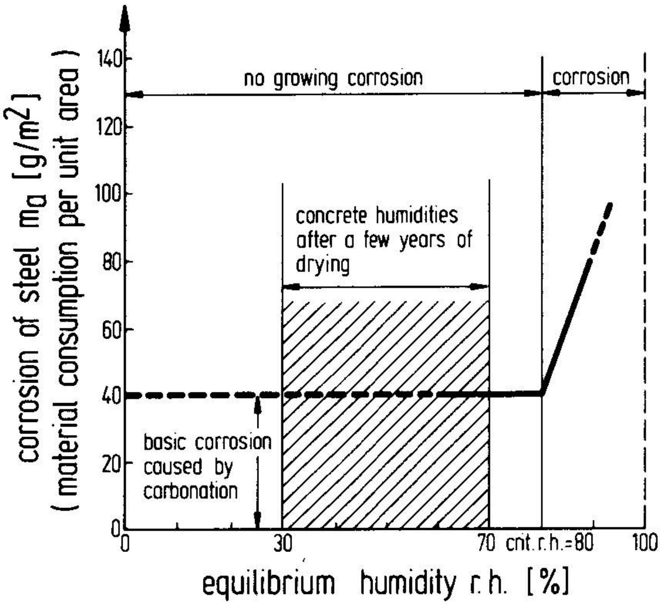

Fig. 5 Scheme of arresting rebar corrosion in concrete exterior walls using additional thermal insulation

Thus, the growth of corrosion of the reinforcement in concrete exterior walls with additional thermal insulation will be impossible. Practical in-situ tests have confirmed the advantages of this method if suitable thermal insulation systems are attached to the exterior walls. It can in fact be established that rebar corrosion in these exterior walls will be arrested, so that a state-of-the-art concrete repair of such walls - which is expensive and may be imperfect - is rendered unnecessary.

Compared with the expense of a hand-applied concrete repair, the costs of additional thermal insulation are quite low. Thus, the rehabilitation of concrete buildings using thermal insulation instead of the common concrete repair will profit, too [4].

REFERENCES

1. MARQUARDT, H. Korrosionsstop in Stahlbetonwänden durch Wärmedämmsysteme. Proceedings of the IABSE-Symposium "Durability of Structures", Lisbon, Sep 6 - 8, 1989. IABSE-Report 57, vol. 2, pp. 797 - 802, Zürich, 1989.
2. MARQUARDT, H. Rehabilitation of Concrete Buildings Using Thermal Insulation. Structural Engineering International, vol. 4, no. 3, pp. 167 - 170, Zürich, 1994.
3. RICKEN, D. Ein einfaches Berechnungsverfahren für die instationäre Wasserdampfdiffusion. Bauphysik, vol. 13, no. 2, pp. 33 - 37, Berlin, 1991.
4. WEIDLICH, B., KERSCHBERGER, A. Energiegerechte Bauschadensanierung. C. F. Müller, Karlsruhe, 1992.

Repair of Corrosion Damages and Protection of Cooling Water Intake/Circulation Structures

Réparation de dommages par corrosion et protection des installations d'eau réfrigérante

Reparatur von Korrosionsschäden und Schutz bei Kühlwasserbauwerken

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SUMMARY

Once corrosion reaches the initial corrosion phase, effective remedial measures must be taken so as to minimise the potential damage and economic loss. A holistic approach with multiple remedial repair/protection measures is essential in arresting corrosion in complex structures. The implementation of those measures in the sea water intakes, pump houses and circulating water conduits of a large power plant requires a comprehensive, long-term program covering surveillance, testing, repair, and maintenance. This paper discusses the implementation of such a program at a power plant as a successful example.

RÉSUMÉ

Il faut intervenir au stade initial de la corrosion des ouvrages par des contremesures efficaces, afin de minimiser les dommages potentiels et les pertes financières. Une approche exhaustive, comportant de multiples mesures de rénovation et de protection, s'avère indispensable pour stopper les effets corrosifs dans les structures complexes. La mise en oeuvre de ces moyens d'intervention dans le circuit de réfrigération d'eau de mer d'une grande centrale électrique (ouvrage d'entrée, station de pompage et tuyauteries de circulation) implique une conception globale et à long terme, incluant surveillance, essais, réparation et entretien. La communication traite de l'application réussie d'un tel programme dans une centrale électrique.

ZUSAMMENFASSUNG

Bereits im Anfangsstadium von Korrosion müssen wirksame Gegenmassnahmen getroffen werden, um mögliche Schäden und wirtschaftlichen Verlust zu minimieren. Bei komplexen Bauwerken braucht es dazu einen ganzheitlichen Ansatz mit mehreren Reparatur- und Schutzmassnahmen. Die Umsetzung solcher Massnahmen im Seewasserkühlkreislauf eines Grosskraftwerkes (Einlaufbauwerk, Pumpenhaus und Umlaufleitungen) benötigt ein umfassendes, langfristiges Vorgehen mit Ueberwachung, Tests, Reparatur und Unterhalt. Der Beitrag berichtet von einer erfolgreichen Durchführung eines solchen Programms.

1. INTRODUCTION

Cooling water intake/circulation structures in power plants located on a seacoast region typically consist of the pump house structures, water delivery conduits, discharge structures and discharge conduits. These structures have frequently experienced accelerated deterioration due to reinforcing steel corrosion, resulting from chloride intrusion. As an increasing amount of effort is being spent on maintenance and repair of these facilities, the industry has been going through a learning curve to achieve cost-effective repair methods. This paper presents a holistic approach to maintenance and protection of cooling water structures against steel corrosion and discusses the implementation of a comprehensive repair/maintenance program at a coastal power plant as a successful example. It is believed that this approach is generally applicable to other marine structures.

2. CORROSION PROBLEMS IN THE SEAWATER INTAKE/CIRCULATION STRUCTURES

The fundamental cause of steel corrosion is electrical potential differences of the steel, in which corrosion (oxidation) of steel occurs at the anodic area and reduction of oxygen occurs at the cathodic area. Oxygen and water are key ingredients for the cathodic reactions, while chloride ions depassivate the anodic regions. Steel corrosion may involve various physical and chemical reactions, depending on the environments. In general, elements of the cooling water structures are subjected to a variety of exposures, ranging from salt water leakage from screen wash stations and glands of pumps to full immersion in seawater. A complete coverage of these conditions is beyond the scope of this paper. Only an introductory discussion is presented to illustrate the complex nature of the problem.

Circulating water conduits contain full head seawater, flowing at moderately high velocity under pressure. It has been recently realized that corrosion of these immersed structures exhibits many characteristics different from the more common corrosion observed in structures continuously exposed to air. Steel corrosion in such an environment is predominantly controlled by the rate of cathodic reaction, due to the limited access of oxygen. Although past experience has shown that marine structures fully immersed in seawater can withstand the severe seawater attack for many years, the circulating water conduits are very vulnerable to corrosion attack due to periodical dewatering of the conduits for cleaning and inspection. During even very short periods of dewatering, oxygen can quickly diffuse through the concrete cover and refuel cathodic reactions. As a result, it is not uncommon to see delaminations and spalling in these conduits, even though they have been in service for merely ten to twenty years.

In the pump houses and discharge structures, there are pumps and valves which are susceptible to salt water leakage due to ineffective seals and pump packing problem, as well as traveling screen wash stations, all of which cause periodical spray, ponding and wetting of the concrete surface. During a drying period, water evaporates from the concrete surface, leaving salt inside the concrete pores. During a wetting period, seawater is quickly absorbed into the concrete, due to capillary "wick" action. Consequently, the concrete components develop chloride concentrations many times greater than that required to initiate corrosion. Furthermore, the chloride content is nonuniform which leads to a substantial potential difference on the steel surfaces and creates a very severe corrosive environment. Multiple pumps, piping and electrically powered equipment inside these structures may induce stray currents, frequently transient in nature, adding additional complexity to the situation.

Different exposure conditions often act synergistically to impose an increasingly aggressive attack on the reinforcing steel. Reference (1) discussed a number of case histories with the interaction of several degradation phenomena. Due to the complex nature of the problem, the authors believe that simple solutions and a single repair/maintenance measure are rarely adequate. A holistic approach to the problem is essential. Multiple repair/protection/maintenance measures need to be systematically combined to arrest several interrelated degradation phenomena. This is the basis of the repair/maintenance program discussed below.

3. CORROSION DAMAGE PROTECTION STRATEGY

The electrochemical corrosion of reinforcing steel involves complex physical and chemical processes. At present, these corrosion mechanisms are still not fully understood. A methodology to provide a quantitative measure of the service life and performance of a corrosion-damaged structure does not exist. In the opinion of the authors, a reliable durability assessment and service life prediction must be based on the field observations and past experience with actual structures and environments. The authors' field experience shows that, under most actual circumstances, corrosion in marine structures propagates at a nonlinear and accelerating rate. For practical purposes, the progression of steel corrosion can be described in three stages: (1) initiation phase; (2) initial corrosion phase; and (3) free corrosion phase. The initiation phase covers the period between the construction of a structure and depassivation of the reinforcing steel. The initial corrosion phase starts with the initiation of corrosion. The free corrosion phase is associated with formation of substantial delamination and cracking. Such delaminations and cracks provide easy access of chlorides, oxygen and water to the reinforcing steel, causing accelerated propagation of corrosion.

In practice, we find that the time for visible corrosion to appear varies in a wide range, depending on many factors such as the concrete quality, concrete cover, construction quality, and severity of the exposure environment. In many seawater intake/circulation systems, considerable corrosion damages were observed in ten to twenty years after the construction. A long delay in repairs may allow the condition to develop into the free corrosion phase, which imposes risk of structural failure and personnel safety and rapidly increase repair/maintenance costs.

Application of the Present Value concept indicates that prompt action to arrest and repair corrosion damage is fully justified, since corrosion damage generally increases more rapidly than the effective rate of interest. Further, the increased interference and impact on the plant operations must be added to the cost of money. Experience demonstrates that the most cost-effective repair strategy is to take remedial measures before the corrosion reaches the free corrosion phase so as to minimize the potential damage and economic loss. Furthermore, a persistent and continuing effort of maintenance and protection of such structures must be exercised in order to maximize the benefits of repair work and, thereby extend the service life of the plants.

4. CORROSION DAMAGE PROTECTION PROGRAM

In the past, general maintenance practices in the power plants usually focused on the plant operation activities with little regard for the progressive detrimental effects of the long-term exposure of seawater on structures. Concrete deterioration and local damage showed that "sustained" maintenance/repair practices would be essential to preserve structural integrity, prevent disruption to plant operations and extend the service life. A comprehensive refurbishment program is being successfully implemented at a nuclear power plant on the Pacific coast. Over four years of consistent efforts in carrying out this program have resulted in stabilization of the degradation trend, thereby improving service performance and minimizing the risk to operations. As a result, the service life of the plant is expected to extend for at least another 40 years with the continuation of the program. This program is discussed in detail below.

4.1 Site Condition Survey

In developing a program for future repair and maintenance, it is necessary to determine the rate at which corrosion will proceed and its effects on the safety of the structures. While corrosion rates depend on complex actions and interactions of many factors and, hence, cannot be accurately quantified, their trends can be clearly bracketed with sufficient accuracy to form a basis for decisions. The condition survey was implemented in two phases.

Phase 1 - Detailed inspection/testing: The objective of the initial inspection and testing was to establish the baseline data regarding the extent of deterioration and to serve as a basis for future monitoring and for service life prediction. Extensive testing was carried out during the initiation stage, which includes visual inspection, hammer sounding to determine delaminations, rapid chloride test to determine the soluble chloride content, half-cell potential test to detect corrosion activities, impact echo test to detect concrete defects like internal/remote cracks, polarization resistance test to estimate corrosion rates and many other tests. In practice, the hammer sounding test has proven to be the most cost-effective and reliable way of finding delaminated and corrosion-damaged areas.

Phase 2 - General monitoring plan: The initial inspection and testing showed that chlorides had penetrated into the concrete in high concentrations, well above the threshold value to initiate corrosion. Although systematic repairs of the severely corroded areas have been undertaken, it is expected that many other areas which have not shown problems to date may develop delamination and/or cracks in the future. A monitoring program was therefore implemented to keep track of the general condition of the intake/circulation system. The monitoring has been carried out periodically, such as during every plant outage. The extent of the monitoring is generally limited to visual inspection and hammer sounding, supplemented by other tests as deemed necessary, especially in local areas. The degraded areas of delaminations, cracks, spalls are charted in a consistent manner to graphically display the inspection results. The main objective of the general monitoring plan is to confirm the trend and progression of the corrosion problem at the plant and detect severely damaged area as early as possible and to develop guideline for planning and prioritizing future repairs.

4.2 Engineering Assessment

From a technical point of view, delaminated and spalled areas should be repaired as soon as practicable, because a long delay in repairs accelerates corrosion and ultimately repair costs. Due to manpower, economic and plant schedule constraints, the amount of repair work may be limited in a given time period. Therefore, the repair work should be prioritized in a systematic way so as to minimize overall costs. A structural assessment should be performed to evaluate the corrosion deterioration effects on the structural integrity. It involves (1) identification of structurally critical areas, (2) evaluation of reductions in safety margins in the degraded areas, and (3) categorization of damaged areas for repair prioritization. The structural components may be divided into four main categories as follows:

- (1) Critical areas in terms of structural adequacy and safety, such as structurally sensitive areas and occupational safety-hazard areas,
- (2) Critical areas in terms of rapid corrosion growth and high economic impact areas where delayed repairs will cause substantially higher costs,
- (3) Non-critical areas showing corrosion damage,
- (4) Non-critical areas with no present corrosion damage.

According to the priority criteria, the areas falling under Categories (1) and (2) are the critical areas in terms of structural adequacy and safety, and rapidly increasing cost of delayed repairs. Corrosion damage in these areas should be repaired as soon as practicable. The areas under category (3) do not require immediate attention and may be scheduled for repair in subsequent years to suit economic options. The areas under category (4) should be monitored.

4.3 Repair/Maintenance Measures

Patching repair is the basic method employed to restore structural integrity of local areas in a corrosion-damaged structure. However, in a typical seawater intake structure, the chloride permeation may progress globally to contaminate extensive portions of the structure, resulting in a general deterioration due to steel corrosion. The patching repair method alone cannot arrest this general deterioration. The effective measures to control such deteriorations include: (1) keeping the pump house structure dry so as to maintain concrete resistivity at a level higher than that associated with steel corrosion, (2) choking off the oxygen supply to cathodic areas, e.g., use of epoxy coating over the concrete surface, and (3) cathodic protection of reinforcing steel in appropriate areas. Proper implementations of these measures are discussed as follows.

4.3.1. Patching repair: In order to ensure successful use of the patching repair method, a guideline application procedure should be developed to address issues like material compatibility and substrate bonding. The guideline includes identifying repair priority, shoring needs, repair sequence as to maintain the structural safety margins, selection of patch materials, surface preparation and curing. To ensure durable repairs, zinc silicate primer and zinc ribbon attachments can be used at the perimeter of the patched areas so as to eliminate or mitigate the reversal of galvanic cells. Epoxy primer is applied to the substrate surface to minimize any adverse interaction between a patch and its substrate. Material selection is one of the critical factors, since improper use of patching materials has become the most common cause of patching repair failures. In the program, the criteria for material selection is based on exposure conditions, structural adequacy, durability, easy construction and cost-effective aspects. The field experience shows that well-constructed, dense concrete with proper addition of pozzolanic admixtures is a very satisfactory material for long lasting repairs. Proper curing and sealing of the perimeter of the patch is important.

4.3.2 Coating systems: Application of surface coatings and sealers proved to be an important protection measure against further ingress of chlorides, especially in the areas periodically exposed to salt water sprays. Selection of the coating system depends on the exposure conditions, effectiveness of the material as a barrier to chloride penetration, and ability of the material to allow vapor migration.

4.3.3 Leakage control: In the pump houses, the most cost effective way to control corrosion is to take adequate preventive measure to stop the leaks and to keep concrete as dry as possible. Simple and inexpensive measures, such as providing adequate seals for glands and valves, using pump spray shields, and the restoration or replacement of hatches and joint seals, are the key elements of the program. Other preventive measures such as providing proper drainage and maintaining floor drains clean and free of debris, should also be implemented,.

4.3.4 Communication: Corrosion awareness communication is an important part of the program. Routine walkdowns and frequent discussions among engineering, construction, maintenance and operation personnel can heighten the awareness of corrosion damage factors like salt water leaks, equipment sprays, plugged drains, water ponding on roofs and floors, rust streaks, cracks, and coating/sealers. This has been one of the key measures in keeping a pulse on the structure performance and has led to timely responses, and appropriate corrective actions.

4.3.5 Cathodic Protection: Cathodic protection has a long and successful history in application to marine structures. Although cathodic protection for above water structures is still in the developmental stage, use of sacrificial aluminum-alloy anodes in protecting reinforcing concrete immersed under seawater has proven to be relatively simple, economical and very efficient. This is primarily due to the high anode-to-ground resistance such that anode currents are able to distribute uniformly over a very large area. This sacrificial anode cathodic protection is especially appropriate for intake/discharge tunnels which are fully submerged in seawater during operational periods. A successful example of such application is the cathodic protection of Jubail seawater canals in Saudi Arabia during the mid-1980's (ref.(2)). Reported performance to date has been excellent. At the power plant, design and construction of similar cathodic protection systems are currently in progress for the circulating water conduits and auxiliary salt water piping.

4.3.6 New technologies: Many innovative proprietary corrosion repair procedures and methods are being developed which give promise of controlling the on-going corrosion and of reducing the cost of repairs. To verify their effectiveness in the exposure environments at the plant, an independent laboratory test is being carried out in conjunction with field testing to investigate several corrosion inhibitors. The test program is set up to simulate (1) the chloride contamination levels, (2) the concrete mix design, and (3) the drying/wetting exposure conditions at the plant.

5. CONCLUSION

Cooling water intake/circulation systems in coastal power plants are exposed to a very severe corrosive environment. The varying exposures often act synergistically to impose an increasingly aggressive attack on the reinforcing steel. Once the corrosion has caused substantial delaminations and cracks, the corrosion rate will increase in a nonlinear and accelerating rate. A long delay in repairs rapidly increases repair costs and ultimately imposes risks of structural failure. For effective results, a comprehensive refurbish program should include the following attributes:

- A sustained, long-term inspection/monitoring/repair plan,
- Thorough engineering assessment and prioritization of degraded areas for repair,
- A holistic approach to combine multiple repair/maintenance measures such as patching repair, coating treatment, control of water leakages and cathodic protection,
- A persistent maintenance effort
- Adaptation to effective new technologies

The experience shows that cathodic protection should be installed early in construction for seawater intake structures. More communication within the industry is needed to share the experiences and information. Various responsible agencies and private sectors should put the resources together in developing new, effective technologies of controlling the corrosion problem.

REFERENCE

1. B.C. Gerwick, "International Experience in the Performance of Marine Concrete", Concrete International, May 1990
2. J.A. Hejji, D.G. John, R.K. Sullivan, "Protection of the Reinforced Concrete Seawater Cooling System at Jubail, Saudi Arabia", Corrosion 89, NACE

Case Study of a Damaged Reinforced Concrete Bridge

Etude d'un cas de détérioration du béton d'un pont

Untersuchung der Ursachen des Betonverschleisses einer Brücke

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SUMMARY

This paper presents the results of an inspection carried out in one bridge built 50 years ago, aiming at identifying the origins of general concrete cracking. The results of the observations and tests made it possible to conclude that the cracking was due to alkali-silica reactions. Subsequently, the corrosion of the reinforcing steel caused larger cracks and concrete spalling.

RÉSUMÉ

Ce travail présente les résultats d'une inspection réalisée sur un pont construit il y a 50 années, afin d'identifier les causes de la fissuration généralisée du béton. Les résultats des observations et des essais effectués ont permis de conclure que la fissuration du béton a résulté des réactions alcali-silice. Postérieurement, la corrosion des armatures a provoqué le développement de la fissuration et l'éclatement du béton.

ZUSAMMENFASSUNG

In dieser Arbeit werden die Ergebnisse einer an einer Brücke vorgenommenen Inspektion dargelegt, die vor etwa 50 Jahren gebaut wurde, um die Ursachen für das allgemeine Aufbrechen des Betons festzustellen. Die Ergebnisse der durchgeführten Untersuchungen und Tests lassen darauf schliessen, dass der Beton aufgrund von Alkali-Silika Reaktionen gerissen ist. Später sind die Risse breiter geworden, und infolge von Korrosionsphänomenen bei den Bewehrungen ist es zur Ablösung von Beton gekommen.

1. INTRODUCTION

The main cause of deterioration in reinforced concrete structures in Portugal is corrosion of the reinforcing bars, started by the attack of chlorides or by the carbonation of concrete. Known cases about the occurrence of expansive reactions in concrete have only been observed in marine structures and dams. In the first case, due to the attack of sulphates [1], in the latter, due to alkali-silica reactions, since silicious aggregates were used [2]. The deterioration due to the formation of secondary ettringite as reported by Brouxel and Hantzo [3], was not yet observed by us.

In this case, the preliminary inspection of the structure seemed to suggest that the corrosion of the reinforcing steel, due to chlorides or carbonation, might have been the primary cause of cracking, as a limestone coarse aggregate was used. However, the results of the tests carried out have pointed to the existence of alkali-silica reactions, the resulting cracking causing thus subsequent corrosion of the rebars.

2. ELEMENTS CONCERNING THE STRUCTURE

The Duarte Pacheco bridge, built in the 40's, is located in the Alcântara valley, a short distance from the Tagus estuary, in Lisbon. It is made up of a central arch with a span of about 100 m, linked by two 85 m long transition portal frames to two lateral arches with a 43 m span each (Fig. 1).

Almost all the information on the structure was lost. It is however known that in manufacturing the concrete coarse limestone aggregate from the region, natural silicious sand and ordinary portland cement were used. The geological information later obtained states that the limestone from the Lisbon area is from the cretaceous, where flint formations are often present, either in the form of inclusions, at times microscopic, or in the form of layers of small thickness not exceeding 10 mm.

3. INSPECTION METHODOLOGY

The general methodology adopted for inspecting the concrete was basically the one recommended by ACI [4], as indicated below:
- Data collection on the bridge construction;
- Preliminary inspection with a view to identifying the problem and defining a detailed inspection plan;
- Detailed visual inspection in order to identify the various types and extents of deterioration in the concrete;
- Carbonation tests, potential corrosion measurements and core drilling for lab tests;
- Lab tests on collected samples, namely chlorides and sulphates contents, examination and compressive strength of cores, SEM/EDX and X-ray diffraction tests;
- Identification of the causes of concrete deterioration.

4. OBSERVATIONS AND TEST RESULTS

4.1 Visual Inspection

The different levels of deterioration found in concrete were characterized as follows [5]:
- Disperse cracking almost imperceptible to the naked eye;
- Disperse cracking or aligned cracking following the main reinforcing bars, less than 0.1 mm wide;

- Disperse cracking or aligned cracking following the main reinforcing bars, about 0.1 mm wide;
- Disperse cracking or aligned cracking following the main reinforcing bars, about 1 mm wide, without concrete spalling;
- Disperse cracking or aligned cracking following the main reinforcing bars, some millimeters wide, with different stages of concrete spalling;
- Localized spalling of the concrete.

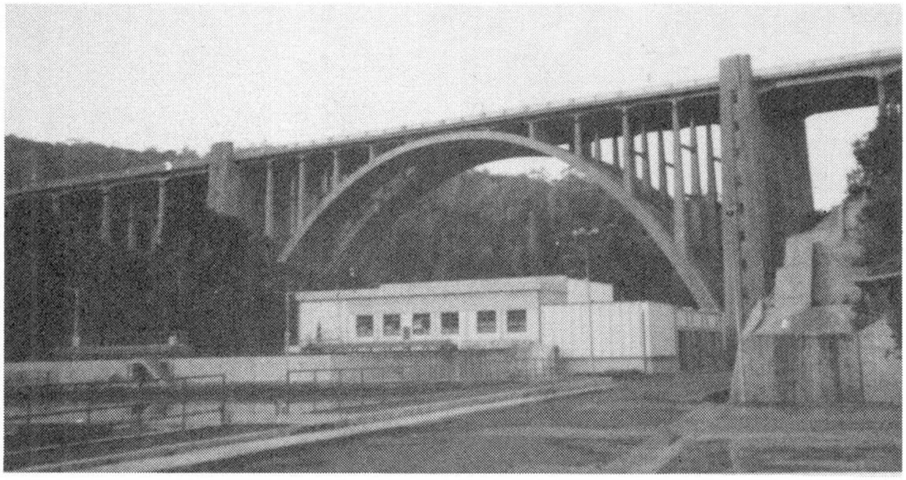


Fig. 1 General view of the bridge

The areas of greater deterioration were located in the central arch, below which the road traffic is more intense, in places more exposed to rain and sunshine. Figs. 2 and 3 show typical examples of cracking observed in concrete.

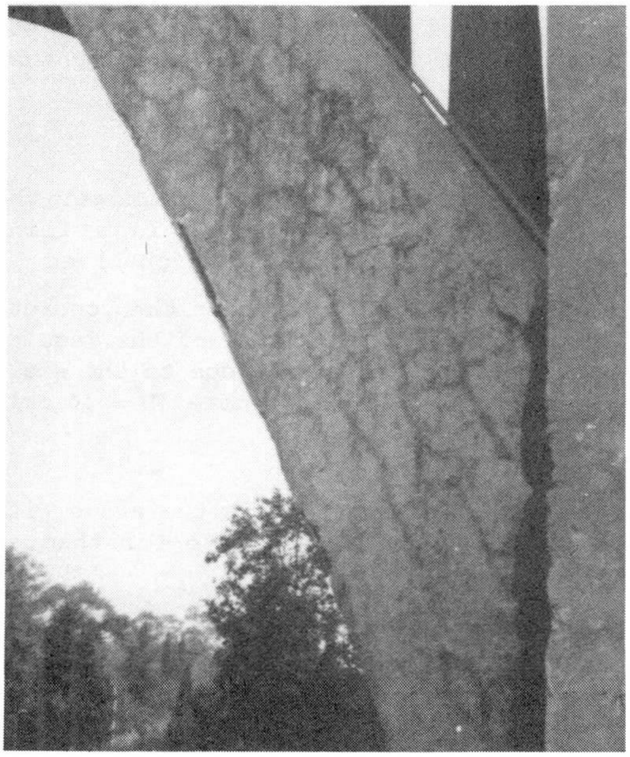


Fig. 2 Central Arch, southern view. Map cracking

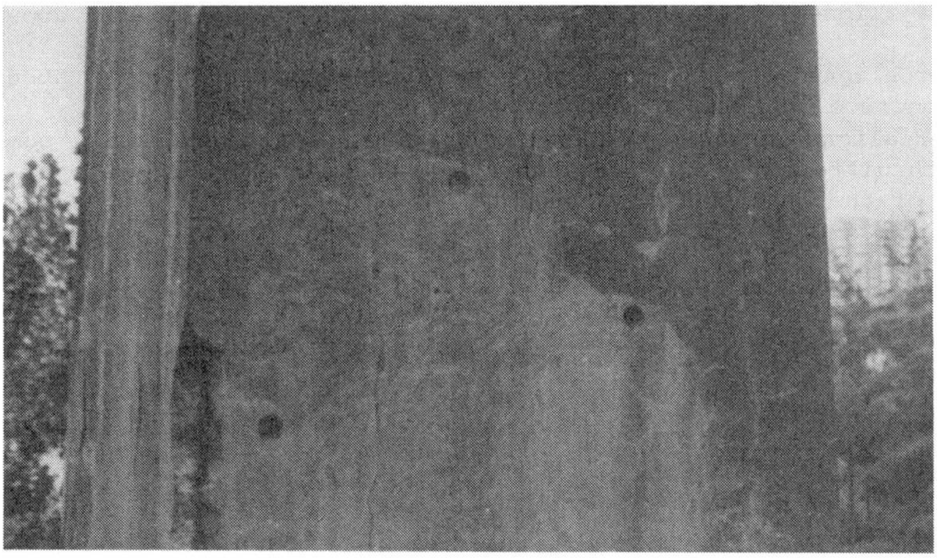


Fig. 3 Column of the Central Arch, facing the south. Cracking aligned with the main reinforcing bars

4.2 Depth of carbonation

The results show that the depth of carbonation was generally less than 10 mm, never exceeding 15 mm.

4.3 Penetration of chlorides and sulphate content

The tests were carried out in laboratory on powder samples taken by a diamond drill from the same places where the carbonation depths were measured.

It was observed that the penetration of chlorides at 10 and 20 mm depths, respectively show maximum contents of 0.15% and 0.07%, in relation to the concrete weight. Considering a cement content of 400 kg/m^3, it can be seen that even for a 20 mm depth the chlorides content is in most cases less than the maximum limit allowed by ENV 206 (0.4% of the cement weight). Mention must be made of the fact that the average chlorides content, expressed in relation to the cement mass is 0.39% and 0.19% at 10 and 20 mm depths, respectively.

As regards to sulphates, results of 4.5% and 1.5% were obtained in relation to the cement weight, respectively for the same 10 and 20 mm depths. It may be noted that there is an increase of sulphates near the surface.

4.4 Examination and compressive strength of cores

The visual inspection of cores, after air drying of the surface, showed the existence of wet areas around flint particles. A crack beginning in a flint particle, at a 80 mm depth, developing towards the surface, was also observed.

The strength tests carried out on cores with h = d = 45 mm showed that the concrete presented an average compressive strength of 70 MPa. The dispersion of the results was fairly high, with values ranging from 41 MPa to 96 MPa, mainly due to the small diameter of the cores, compared with the maximum size of the aggregate (D = 40 mm).

4.5 Corrosion potentials

It was observed that the corrosion potentials were more negative in the areas with greater cracking. The values were not however sufficiently negative for them to indicate active corrosion on the steel surface.

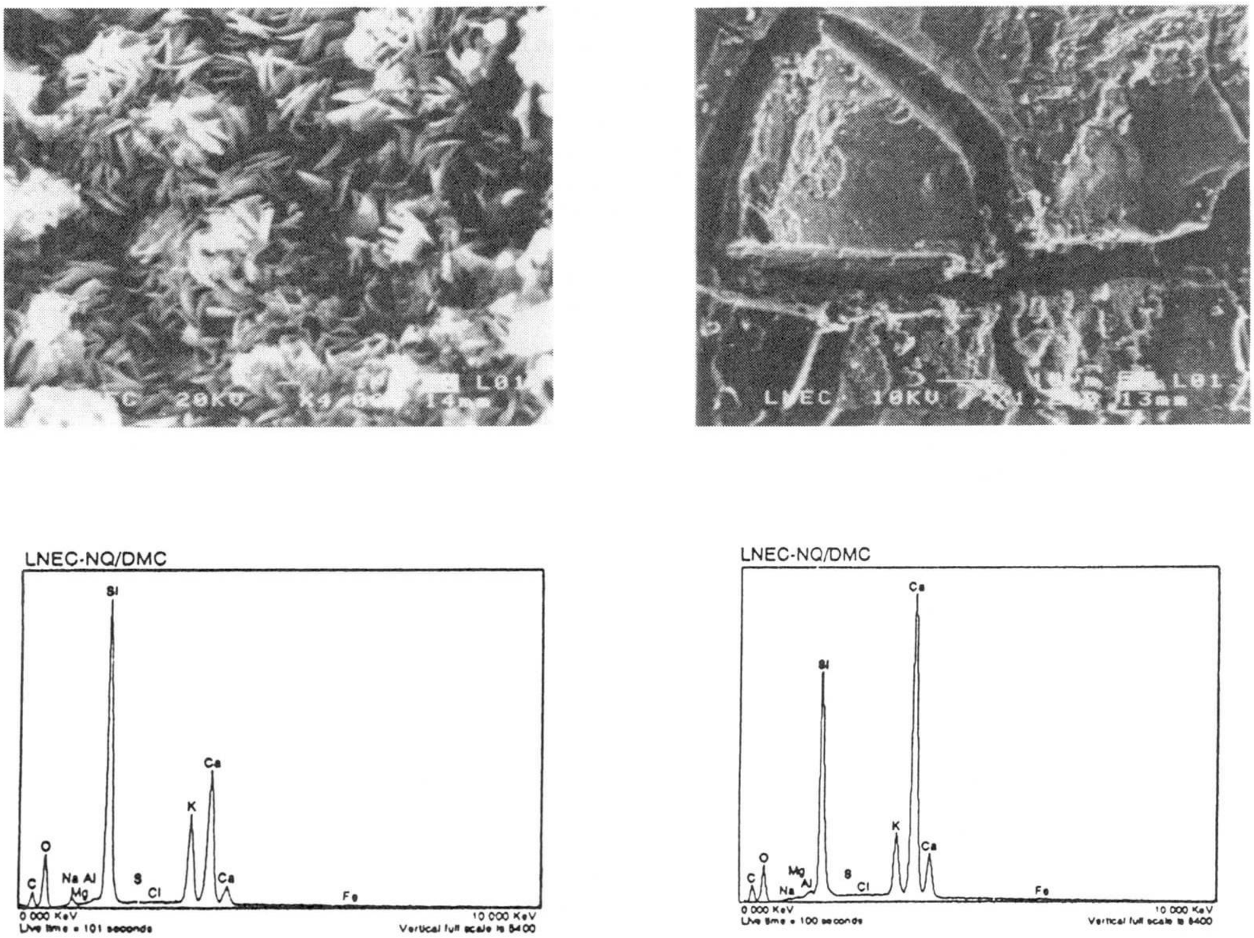
4.6 SEM/EDX tests

Scanning electron microscope (SEM) observations and energy-dispersive X-ray (EDX) microanalysis in the steel/concrete interfaces of some samples taken from the

Central Arch, showed the absence of chlorides and the presence of a high number of ettringite agglomerates, both in a massive or acicular crystals forms.

Observations were also carried out on other samples taken from more cracked areas of the Central Arch. These observations showed in limestone/paste interfaces the presence of typical compounds of alkali-silica reaction (Fig. 4), such as hydrated calcium and potassium silicates (crystalline and amorphous), as well as ettringite, either in the characteristic form of acicular crystalls, or in a massive form. Other authors [6] have observed similar morphologies in the presence of alkali-silica reactions.

The X-ray microanalysis of the alkali-silica reaction products showed that the crystalline products are less rich in calcium (Ca) than the amorphous ones. The occurrence of ettringite in cracks, pores and slightly disperse in the paste was also observed.



a) b)

Fig. 4 Products of alkali-silica reaction (calcium and potassium silicates) in a limestone/paste interface.
a) Crystalline products and corresponding X-ray spectrum;
b) Cracked gel and corresponding X-ray spectrum.

4.7 X-ray diffraction analysis

The presence of ettringite in the paste was also confirmed by X-ray diffraction. In the samples corresponding to more cracked areas, sulphates in proportions slightly higher than those observed in samples from uncracked areas were detected.

The X-ray diffraction analysis of the coarse aggregate particles taken from the concrete indicated that they were mainly made up of calcite mineral with traces of quartz.

5. CONCLUSIONS

Carbonation induced corrosion is only relevant in certain areas, where the cover was small due to construction defects.

The low chloride content of concrete indicates that steel corrosion is not due to chloride attack. The non-existence of chlorides in the rebar corrosion products, as showed by SEM observations, reinforces this conclusion.

The results on the corrosion potentials did not show, even in more cracked areas, the existence of active corrosion at the steel surface.

The visual observation of cores, the results of the SEM/EDX tests, and the X-ray diffraction analysis, suggest that the concrete cracking results from the alkali-silica reactions between the alkalis of cement and reactive silica inclusions of limestone aggregate. A slight attack by sulphates of external origin, resulting from the atmospheric pollution (SO_2) may also be present. Subsequently concrete cracking, may have provided, in some places, the conditions for depassivation of the reinforcing steel and the consequent emergence of corrosion, causing thus the increase of cracks width and concrete spalling.

6. ACKNOWLEDGMENTS

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7. REFERENCES

1. COUTINHO A.S., Aspects of Sulfate Attack on Concrete. Cement Concrete and Aggregates, Vol. 1, n° 1, 1979.
2. REIS M.O.B., Study of Alto Ceira Dam Concrete. Chemical and Physico-Chemical Analysis. LNEC Report (in Portuguese), 1991.
3. BROUXEL M. and HANTZO Q., Durability of a Bridge Affected by an Internal Expansive Phenomenon. Third CANMET/ACI International Conference on Durability of Concrete, Supplementary papers, Nice, France, 1994, pg 355-371.
4. Guide for Evaluation of Concrete Structures Prior to Rehabilitation. Reported by ACI Committee 364, ACI Materials Journal, 1993, pg 479-498.
5. GONÇALVES ARLINDO et al, Study on the Causes of Deterioration of Concrete in Duarte Pacheco Bridge. LNEC Report, 1994.
6. BERUBÉ M.A., CARLES-GIBERGUES A., La Durabilité des Bétons face à l'Alcali-Réaction. La Durabilité des Bétons, P. Ponts et Chaussées, Paris, France, 1992.

Beam Exposure Study of Epoxy-Coated Reinforcement

Etude de l'exposition des poutres avec des armatures enrobées d'époxy

Balkenbewitterungsstudie hinsichtlich epoxidbeschichteter Bewehrung

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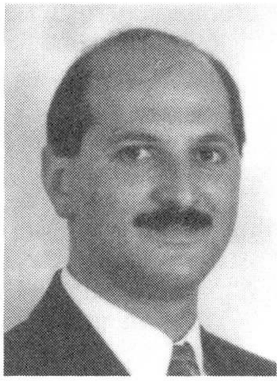

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SUMMARY

The performance of fusion-bonded epoxy-coated reinforcement under conditions which simulate a highly corrosive environment and under loading conditions producing concrete cracking was evaluated. Reinforcement details included longitudinal bars, stirrups, and splices. Coating condition was a variable to assess the effects of coating damage and patching on performance. The results indicated that corrosion initiation and progression was enhanced by concrete cracking and damage to coating. To enhance performance, coating damage and concrete cracking need to be reduced, and patching practice and coating adhesion to substrate need to be improved.

RÉSUMÉ

L'objet de l'étude est d'évaluer la performance des barres enrobées d'époxy dans un milieu corrosif et sous des charges produisant des fissures dans le béton. Les types des barres incluent les barres longitudinales, les étriers et les crochets. L'état de l'enrobage a été considéré comme une variable afin d'évaluer l'effet du dommage et des retouches subies. Les résultats ont montré que le début de la corrosion des barres et l'extension de la corrosion ont été provoqués par les fissures du béton et le dommage à l'enrobage. Afin de parvenir à une meilleure performance, il faut réduire les dégâts à l'enrobage et les fissures du béton, tout en améliorant les méthodes de retouche et d'adhésion de l'enrobage aux barres.

ZUSAMMENFASSUNG

Untersucht wurde das Verhalten aufgeschmolzener Epoxidbeschichtung von Bewehrungsstahl unter Bedingungen, die in hochkorrosiver Atmosphäre zur Beton-rissbildung infolge äusserer Belastung führen. Die Bewehrungsdetails umfassten Längs-eisen, Bügel und Stösse. Hauptparameter war der Zustand der Beschichtung, um die Auswirkung von Beschädigungen und Ausbesserungen zu bestimmen. Die Ergebnisse zeigen, dass Beginn und Fortschritt der Korrosion durch die Betonrissbildung und Schäden an der Beschichtung begünstigt wurden. Um die Leistungsfähigkeit der Beschichtung zu verbessern, müssen diese beiden Einflüsse reduziert und die Technik zur Ausbesserung von Beschichtungsschäden sowie die Haftung der Schicht am Untergrund verbessert werden.

1. INTRODUCTION

Concrete within the tidal zone in a marine environment undergoes cyclic wetting and drying and significant localized chloride accumulation. Surface cracking due to loading facilitates further chloride penetration which eventually precipitates corrosion of reinforcement. Although epoxy-coated steel is widely specified for new construction in corrosive environments and for replacing corroded steel in rehabilitated structures, the long-term performance of coated reinforcement has been questioned recently. Of concern are the effects of coating damage and debonding on corrosion resistance. In the following accelerated corrosion testing, the effectiveness of fusion-bonded epoxy-coated bars, with controlled levels of damage, was evaluated.

2. EXPERIMENTAL PROGRAM

2.1 Concrete and Epoxy-Coated Steel Details

Three similar concrete batches containing 222 kg/m^3 of ordinary portland cement and a maximum aggregate size of 20 mm were used to cast three groups of beams. The w/c ratio was about 0.62 producing concrete with high permeability and an average 28-day compressive strength of 26 MPa. The beams were 0.2x0.3 m in cross section and 3.0 m long. Clear concrete cover was 50 mm.

U.S. grade 60 plant-coated and fabricated bars were used. Beam reinforcement consisted of two 19 mm coated bars at the tension side, one 10 mm coated stirrup at midspan, and two 10 mm uncoated bars at the compression side. Coating thickness measurements on the longitudinal bars and stirrups were within the acceptable range of 130-300 μm. A predetermined amount of damage in the form of small rectangles was introduced in the coating on some bars using a sharp blade. For the longitudinal reinforcement, damage was estimated (as % of surface area) and distributed along the middle 0.9 m of the bar. Damage spots were located between transverse lugs and at the lugs themselves. For the stirrups, damage was estimated for roughly half the length of the stirrup, and was distributed along the outer surfaces of the bends.

2.2 Test Variables and Conditions

Three groups with a total of 34 beams were included in the test. In the first group, the stirrups were isolated from the other bars to allow corrosion monitoring of the longitudinal tensile bars. In the second group, the tensile bars were isolated to allow corrosion monitoring of the stirrups. No isolation was used in the third group to allow corrosion monitoring of both longitudinal tensile bars (which included spliced bars in some cases) and stirrups. Table 1 summarizes the variables included in each beam group. The beams were positioned in such a manner that their own weight was causing bending about the weak axis. Loading as a test variable refers to imposed loads causing bending about strong axis. The three loading conditions selected for the test were as follows: Uncracked Unloaded: At rest condition (no cracks or imposed loads) during exposure; Cracked Unloaded: A load was applied to produce a crack of 0.3 mm width then the load was removed during exposure; Cracked Loaded: A load was applied to produce a crack of 0.3 mm width then the load was held during exposure.

Coating damage level up to or exceeding current specification limits may occur in field applications.[1] Recent discussions regarding modifying the specifications proposed limiting the total bar surface area covered by patching material to 3%. For this test, the bar surface conditions investigated were mainly: the as received condition (no visible damage); and 3% damage, exposed or repaired. Repairs were done according to the manufacturer's instructions using a liquid epoxy patching material and following recommended touch-up techniques.

2.3 Test Setup and Procedure

There were two replicate beams for each test condition. The two replicates were stressed back to back as in the model shown in Fig. 1.

The exposure conditions consisted of 3.5% NaCl solution flowing over the beam surfaces (within a defined exposure area) continuously for 3 days followed by air drying for 11 days. Periodic wetting and drying ensures continuous transport of corrosive substances to steel surfaces to promote corrosion. The cracked beams were subjected to cycles of loading and unloading twice during each exposure cycle: one time during wetting and the other during drying. Five load cycles were imposed each time up to a level producing the selected maximum crack width. Loading and unloading may promote physical damage to coating and to concrete at crack locations and increase exposure to corrosive substances.

Bar Condition (Damage Level and Condition, P=Patched)	**Loading Condition**		
	Uncracked Unloaded	**Cracked Unloaded**	**Cracked Loaded**
Group I Beams, Monitoring Longitudinal Bars			
As Received	B1, B2	B3, B4	B5, B6
3% Damaged	B7, B8	B9, B10	B11, B12
3% Damaged, P		B13, B14	
Group II Beams, Monitoring Stirrups			
As Received	B15, B16	B17, B18	B19, B20
As Received, P	B21, B22	B23, B24	B25, B26
3% Damaged, P		B27, B28	
Group III Beams, Monitoring Long. Bars and Stirrups			
Mixed Longitudinal Bars and Stirrups			
3% Damaged, P		B29, B30	
Mixed Splice Bars and Stirrups			
3% Damaged, P		B31, B32	B33, B34

Table 1 Summary of Beam Exposure Study Specimens

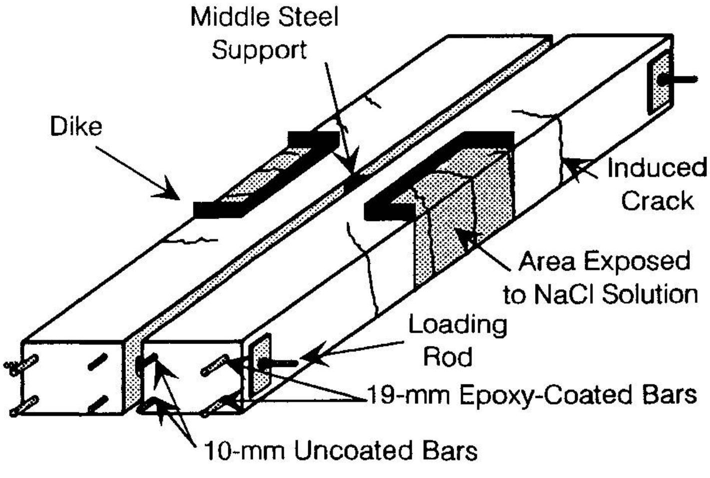

Fig. 1 Model of Beam Test Specimen

2.4 Corrosion Monitoring

The surfaces of beam specimens were visually inspected periodically for any signs of staining or crack development due to corrosion but none showed such activity during the 400-day observation period. For corrosion monitoring, half-cell potential measurements against a saturated calomel reference electrode (SCE) were made. The points of measurement were closely spaced along the concrete surfaces parallel to the coated bars. After 400 days of testing, half of the beams were demolished to relate measurements to actual bar condition. The test was continued with the remaining beams to provide more information in the future.

3. TEST RESULTS AND DISCUSSION

3.1 Half-Cell Potentials

Examples of average potentials measured on dry and wet regions along the beams versus time of exposure are shown in Fig. 2. Time-to-onset of suspected corrosion as indicated by potential drop varied among beams with different coating damage and loading conditions. In general, as received bars and stirrups delayed corrosion more than bars with 3% damage. Corrosion cells in cracked beams were suspected to form after the first salt application. Bars with patched damage showed comparable time-to-corrosion as bars with unrepaired damage suggesting that patching was not effective. In general, similar periods to suspected corrosion initiation were observed for bars in cracked loaded and unloaded beams. The similarity means that crack width did not affect the time to active conditions for corrosion.

At the beginning of the exposure test, half-cell potentials were not stable. The potentials measured on all uncracked beams were between approximately -50 and -200 mV SCE. This potential range reflects steel passivity. The initial potentials for all cracked beams, however, ranged between approximately -50 and -500 mV SCE. The cause of the different initial potential values may be attributed to early contact of steel with chlorides penetrating through the cracks.

After an initial or delayed potential drop, or fluctuation, the potential remained steady with time. The potential varied consistently within a narrow range of highly negative values which indicated that active conditions persisted for the remainder of the test. Corrosion progression in this situation was almost certain. Sufficient chloride ions were available to maintain activity. The final potentials for uncracked and cracked beams that exhibited an appreciable drop in potential reached about -400 and -650 mV SCE, respectively. The range of final potentials agrees very well with that presented by Wheat and Eliezer[2] for general corrosion due to loss of passivity which is -450 to -600 mV. Sagüés[3] also measured potentials in the range of -350 to -475 mV SCE after almost 300 days of exposure of uncracked columns containing coated bars.

Due to hydrolysis, the localized lower pH of solution within an active pit encourages further corrosion at the available potential level. This phenomenon explains why corrosion progression and pitting continued on some bars at a stable half-cell potential between -400 and -600 mV SCE.

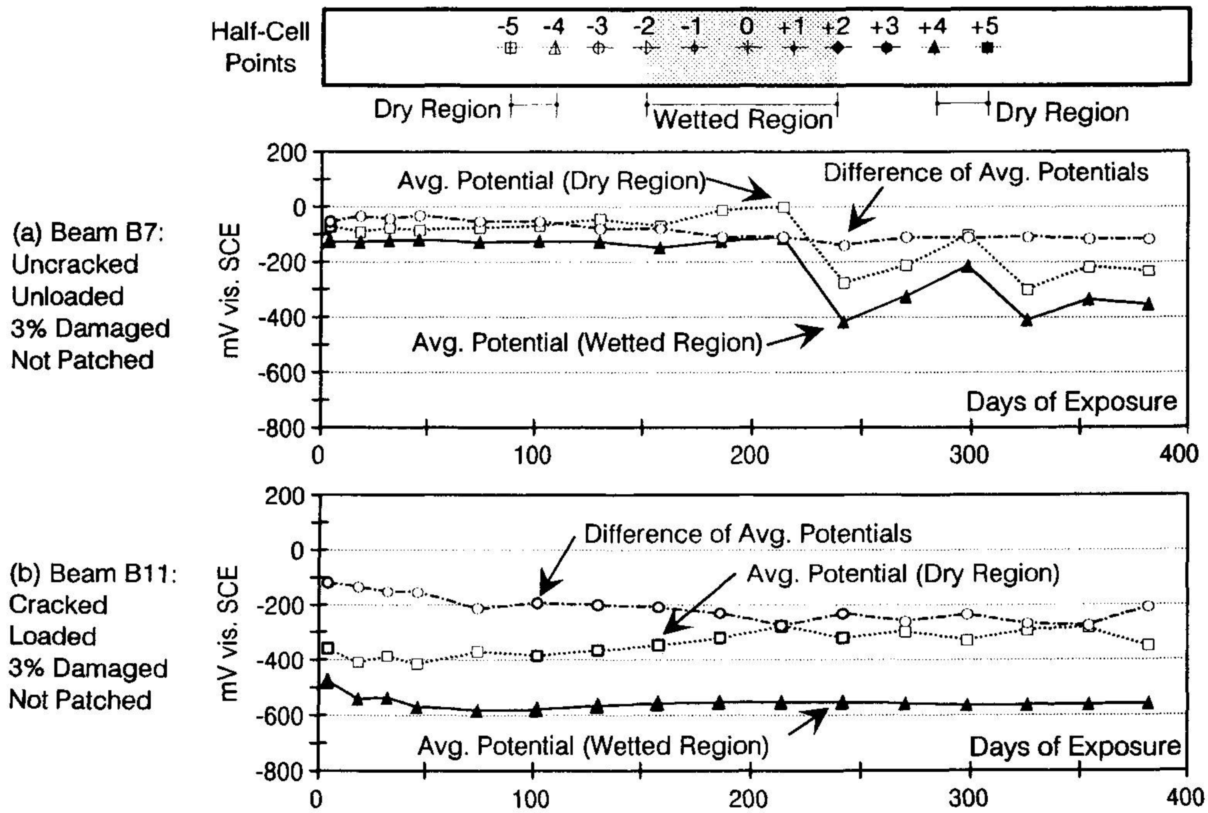

Fig. 2 Average Potentials of "Dry" and "Wet" Regions

Based on the discussion above and observations of actual bar condition, electrical half-cell potential values correlated well with the state of corrosion of coated steel. Figure 3 summarizes the relation between the ranges of measured potentials and corrosion state. Although the extent of corrosion spreading on the bar surface did not correlate with steel potential, the corroded area tended to increase as corrosion severity increased.

Previous studies have indicated that the electrical potential of an anode cannot be used to indicate rate of corrosion. Elsener and Bohni[4] found that the local potential gradient was a better way to identify the type of corrosion and to locate corroding sites. Large differences of potentials along a concrete member may be used as indicators of macrocell formations. The closely-spaced potential measurement points along the beam surfaces allowed the identification of predominantly anodic and cathodic sites. Potential differences were studied between adjacent and far points (about 0.6 m apart). The state of corrosion was consistently related to these differences as follows:

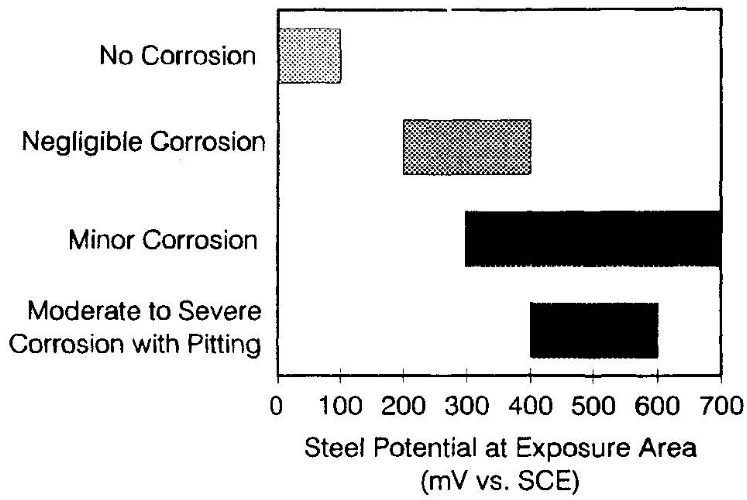

Fig. 3 Relation Between Corrosion Activity and Steel Potential

- No corrosion was associated with potential gradients < 150 mV.
- General corrosion (negligible-moderate) was associated with potential gradients > 150 mV.
- Pitting corrosion in presence of chlorides was associated with potential gradients > 200 mV.

3.2 Condition of Coated Reinforcing Steel

Based on visible surface corrosion of steel, coated bars and stirrups exhibited more corrosion in cracked beams than uncracked beams. In general, corrosion was not much different on bars and stirrups in cracked loaded and unloaded beams. Thus, whether cracks were wide or narrow had less impact on corrosion performance than whether concrete was cracked or not. The severe corrosion testing conditions for both coated bars and stirrups resulted in significant loss of coating adhesion to steel. Coating debonding occurred around all corroded sites and damaged spots. The straight portions of stirrups initially had stronger coating adhesion than the bent portions. However, all portions exhibited significant debonding after one year of exposure, particularly in cracked beams. Significant pitting was only observed on damaged longitudinal bars in cracked loaded beams. In addition, blisters formed mainly on the bottom sides of bars (in casting position) facing air voids in concrete.

The undamaged epoxy-coated bars and stirrups in uncracked beams retained their original appearance with negligible or no corrosion or blistering despite the high chloride content (about 5-6 kg/m^3). For these bars, there was no or very limited loss in coating adhesion to substrate steel. These results indicate that originally intact epoxy coating can provide adequate protection to reinforcing steel from chloride-induced corrosion. Corrosion was also apparent on repaired areas and cut bar ends indicating that patching was not effective.

For longitudinal bars, undercutting was confined to some mill marks and exposed steel areas in uncracked beams. Undercutting increased slightly in cracked beams and spread around the crack locations and areas of no previous damage. For stirrups, undercutting increased noticeably in cracked beams and mostly covered the bends and hook ends. It is believed that exposure to excessive amounts of chlorides for extended periods and macrocell formation on the stirrups eventually led to corrosion initiation and breakdown of coating.

4. CONCLUSIONS

- Cracking of concrete surfaces promoted corrosion of epoxy-coated steel. The impact of crack width on corrosion initiation and later progression was not significant.

- Systematic periodical measurement of half-cell potentials at the end of wet periods was valuable in predicting the corrosion state of embedded epoxy-coated bars. Corrosion was negligible when potentials remained below -400 mV SCE without significant potential gradients along the monitored bar. Pitting corrosion was always associated with potentials in the range of -400 to -600 mV SCE and steep potential gradients in excess of 200 mV.

- Severity of corrosion on epoxy-coated bars was related to both coating damage level and loading condition. As received bars and stirrups in uncracked members performed well, whereas those with damaged coating in cracked beams showed the worst performance. Patching of damaged coating and cut bar ends reduced the severity of corrosion, but did not provide full protection.

- The quality of concrete at the bar interface affected the location where corrosion initiated and progressed. There was a tendency for the epoxy coating to develop blisters and to break down at voids in contact with bar surface.

- To improve the effectiveness of coated steel, damage to coating needs to be reduced, patching requirements need to be modified, concrete quality needs to be improved, coating adhesion to steel needs to be enhanced, and treatment of cracked concrete surfaces needs to be considered.

ACKNOWLEDGMENT

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REFERENCES

1. KAHHALEH K., Corrosion Performance of Epoxy-Coated Reinforcement. Dissertation, The University of Texas at Austin, May 1994.

2. WHEAT H.G., AND ELIEZER Z., Some Electrochemical Aspects of Corrosion of Steel in Concrete. *Corrosion*, NACE, Vol. 41, No. 11, November 1985, pp 640-645.

3. SAGUES A., Mechanism of Corrosion of Epoxy-Coated Reinforcing Steel in Concrete. Report No. FL/DOT/RMC/0543-3296, University of South Florida, April 1991.

4. ELSENER B., AND BOHNI H., Potential Mapping and Corrosion of Steel in Concrete. *Corrosion Rates of Steel in Concrete,* ASTM STP 1065, N.S. Berke, V. Chaker, and D. Whiting, Eds., American Society for Testing and Materials, 1990, pp 143-156.

Strength of Concrete Beams during Concrete Breakout

Résistance de poutres en béton armé en cours de réparation

Tragfähigkeit von Betonbalken während des Ausspitzens

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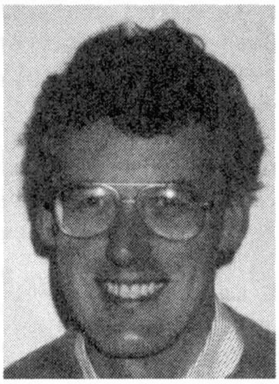

John Cairns is a Chartered Structural Engineer with research interests in structural aspects of concrete repair and bond between reinforcement and concrete. He currently serves on a Task Group of CEB and on the Concrete Society Working Party on Epoxy coated reinforcement.

SUMMARY

The paper provides guidance on assessment of the load carrying capacity of reinforced concrete beams when weakened by exposure of tension reinforcement during structural repairs.

RÉSUMÉ

Cette étude porte sur l'évaluation de la capacité de charge de poutres en béton armé, présentant une armature apparente lors de réparations.

ZUSAMMENFASSUNG

Der Beitrag gibt Empfehlungen zur Beurteilung der Tragfähigkeit von Stahlbetonbalken, wenn diese durch Freilegen der Zugbewehrung zu Reparaturzwecke geschwächt sind.

1 INTRODUCTION

Repairs to reinforced concrete beams suffering from chloride induced corrosion frequently necessitate breaking out of concrete around the full perimeter of reinforcing bars. Bond between bar and concrete is then lost over the length exposed. In the absence of bond, an exposed bar cannot act compositely with the remainder of the member, and normal assumptions of plane section behaviour no longer hold true. The usual Code of Practice procedures for evaluation of section strength are no longer directly applicable.

Although the need to ensure structural stability of a member weakened by removal of concrete cover to tension bars is mentioned in many texts on repair, little detailed guidance is available. Design calculations for structures so weakened therefore tend to be based on conservative assumptions. It is common practice to ignore any contribution from exposed bars when assessing structural strength of weakened members. As exposed bars are assumed ineffective, temporary support will in many cases be required to ensure an adequate margin of safety is maintained. Where it is not feasible to utilise props, repairs have to be carried out in a piecemeal fashion, necessitating a long and slow repair programme.

The assumption that an exposed bar makes no contribution to member strength errs on the side of caution, and significant stress may develop in an exposed (unbonded) bar if the ends are adequately anchored[1]. Substantial savings in repair costs might be possible if reliable methods were available for evaluation of the strength of members with all or part of the reinforcement exposed.

The aim of this paper is to show that relatively simple procedures may be employed to estimate the length over which bars may be exposed without loss of strength.

2. SECTION BEHAVIOUR WHEN BARS EXPOSED

2.1 General Aspects Whether reinforcement is bonded to concrete or not, conditions of equilibrium of forces and compatibility of deformations must be satisfied. Equilibrium of a reinforced concrete beam may be described by an equation of the form :

$$M = f_{st} \cdot A_{st} \cdot z \qquad \text{Eqtn. 1.}$$

where M = applied bending moment.
f_{st} = tensile stress in reinforcement.
A_{st} = cross sectional area of reinforcement, assumed constant.
z = lever arm between centroid of reinforcement and concrete in compression.

When reinforcement is bonded, the lever arm, z, is sensibly constant, and stress in the reinforcement varies in proportion to the applied bending moment. In the absence of bond, however, stress in reinforcement cannot vary along the bar, and it is instead the lever arm which must change in response to a varying applied moment. For equilibrium to be satisfied, the centre of the concrete in compression must therefore move towards the tension reinforcement under a reducing bending moment, Figure 1. Structural action of the member alters from the flexural behaviour of a beam towards that of a tied arch.

2.2 Bending strength

Where reinforcement is rigidly bonded to concrete, compatibility of deformations is satisfied through normal assumptions of plane section behaviour. If bars are disbonded, however, plane section assumptions no longer hold true, and compatibility must be satisfied over the length of bar between points of anchorage. Strains reduce towards the support when bars are bonded, Figure 1(a), but remain constant when bars are exposed. Elongation of the exposed bar will therefore exceed that of an equivalent bonded bar under a given load. To satisfy compatibility, there must also be an increase in elongation of the concrete between anchorages. The necessary increase in elongation of the concrete is achieved through an increase in curvature of the concrete section near midspan. This will in turn increase the maximum compressive strain in the concrete at midspan. As failure of reinforced concrete is deemed to occur at a limiting compressive strain, it is clear that bending strength may be affected by loss of bond.

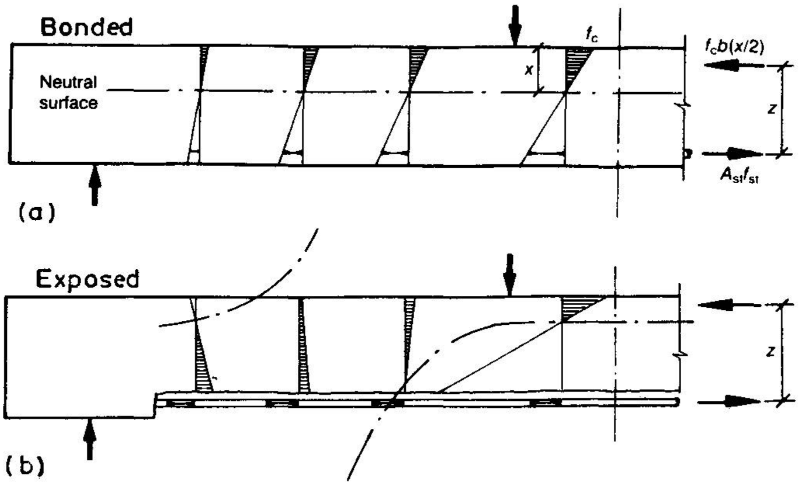

Figure 1 Pattern of strains in reinforced concrete beams

2.3 Shear strength

Normal assumptions of dowel action, of aggregate interlock effects, and of the state of stress in the compression zone of the beam no longer hold when concrete is broken out around tension bars. Links will be ineffective when their corners are exposed. This leads to fears that shear strength will be reduced when bars are exposed. Equation 1 may be differentiated to give Equation 2.

$$V = dM/dx = d(f_{st}.A_{st}.z)/dx = f_{st}\ A_{st}\ dz/dx + z\ A_{st}\ df_{st}/dx \qquad \text{Eqtn 2.}$$

The first component on the right of Equation 2 represents the contribution of arch action, the second represents beam shear. In flexural behaviour of elastic materials, only beam action is present, and the lever arm between tension and compression parts of the couple remains constant along the length of the member. The term (dz/dx) is then zero. If bond is lost through exposure of reinforcement, (dz/dx) is non-zero, for the reasons discussed above, but (df_{st}/dx) is instead zero. Leonhardt & Walther[2] (amongst others) have reported an increase in shear strength in beams detailed to fail in shear with normal bonded reinforcement when bond strength was reduced. Cairns and Zhao[1] & Raoof[3] have conducted tests which show strength of beams which would fail in shear if reinforcement were fully bonded is not reduced by exposure of bars. Cairns[4] has demonstrated an increase in shear capacity when 50% of the bars in a section are exposed. An increase in the shear contribution of arch action therefore offsets a reduction in the beam contribution.

2.4 Other failure modes

Figure 1 also shows that if the exposed length extends close to the support, tensile strains start to develop in the top 'compression' face of the beam, and compressive stresses develop in the 'tension' zone. A compression failure of the concrete may occur as the lever arm reduces towards the support and the centre of compression of the concrete moves towards the "tension" face. Flanged 'T' sections are more vulnerable to this mode of failure, as the compression force carried in the flange near midspan must be resisted within the thickness of the web. The possibility of end anchorage failure is also increased if bond is lost within the shear span.

The potential modes of failure are summarised graphically in Figure 2. In subsequent sections, simplified methods for predicting the length of bar which may be exposed without significant strength loss are developed for each mode.

3. DERIVATION OF SIMPLIFIED ANALYSIS

Cairns and Zhao[1] have undertaken finite element analyses of flexural strength of beams with all reinforcement exposed over a portion of the span of simply supported beams. Behaviour of the entire member, and not just the behaviour of critical sections, must be considered. Eyre also developed analytical expressions for analysis of beams with disbonded bars[5]. These analytical methods both require to be implemented on computer, and it is desirable to have simpler (if less accurate) methods of calculation for practical use. As a first stage, this paper develops methods that could be used to estimate the length of bar that may be exposed without significant loss of strength. The analysis is based on the following observations and assumptions :

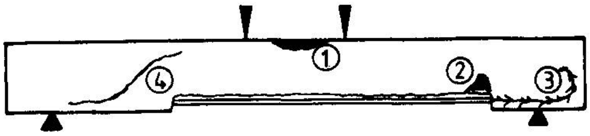

Figure 2 Failure Modes of beam with exposed bars.

a) bending strength is little affected if reinforcement attains yield strength f_y (although ductility is reduced).
b) the pattern of stress in the concrete at midspan at ultimate load is little affected by the loss of bond if reinforcement attains yield.
c) bond slip at the ends of the bar may be neglected.
d) at ultimate load, strains in the concrete within the shear span are small in relation to those within the constant moment zone.

Each of the four failure modes represented in Figure 2 must be verified separately, and the least strength calculated for the various failure modes will control. Only simply supported beams are considered. Mode 1, crushing of concrete at the point of maximum moment within the exposed length, is considered first.

The elongation of reinforcement over the exposed length when reinforcement yields, d_{st}, is given by Equation 3. Notation is described in Figure 3.

$$d_{st} = L_{exp}.f_y/E_{st} \qquad \text{Eqtn. 3.}$$

If concrete is assumed to fail at a limiting compressive strain of 0.0035, and from assumption (d) above, the elongation of the concrete at the level of the tension bars at ultimate is given by Equation 4. Neutral axis depth x_1 should be calculated in the normal way for a section with bonded reinforcement at the ultimate limit state.

$$d_c = L_0.0.0035(d/x_1-1) \qquad \text{Eqtn. 4.}$$

In circumstances where the loading pattern does not provide a constant moment zone, or where the constant moment zone is very short, the length L_0 should instead be taken as the length of beam L_{cr} over which crushing of the concrete develops. From an expression derived by Phipps[6], this may be taken as :

$$L_{cr} = 3.5\ x - 0.0075x^2 \quad , \text{ and } \quad L_0 \geq L_{cr} \qquad \text{Eqtn. 5.}$$

For compatibility to be satisfied, elongation of tension reinforcement over the exposed length must equal that of the concrete at the corresponding level. If the elongation of the concrete at ultimate load d_c equals or exceeds that of the reinforcement at yield d_{st}, then the reinforcement will attain yield at ultimate, and beam strength will be unaffected by exposure.

Figure 4 shows a comparison between the ratio d_{st}/d_c as calculated using Equations

3-5 and the ratio of M/M_{bonded} obtained in tests by Cairns & Zhao[1], Minkarah and Ringo[7], and by Eyre[8]. The reduction in strength as a result of exposure of reinforcement will be less than 10% provided the ratio of d_{st}/d_c exceeds 1.5. The limiting value of 1.5, and not 1.0 as might be expected, reflects errors introduced by the various simplifying assumptions. Equating d_{st} and d_c from Eqtns 3 & 4 and introducing the 1.5 factor leads to Equation 6, which provides an estimate of the maximum length of bar which can be exposed while retaining at least 90% of fully bonded strength (for this failure mode). It should, however, be noted that only a very limited amount of data is available for cases where the more highly stressed end of the exposed length lies within the shear span and more than one effective depth distant from the constant moment zone, and the Equations presented here should be treated with great caution in such circumstances until more data is available to permit validation.

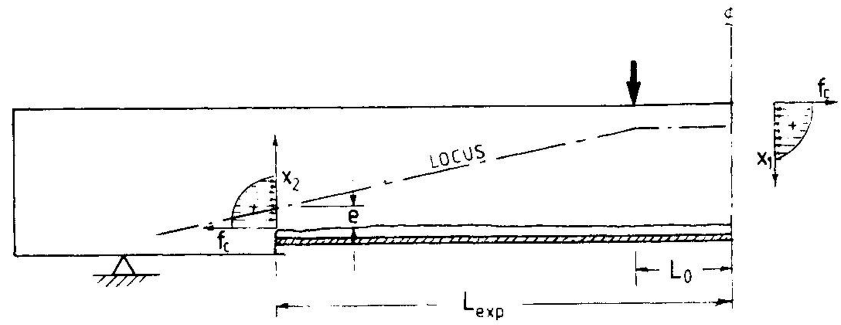

Figure 3 Pattern of stress at critical sections.

$$L_{exp} < (d/x_1-1)\ 0.0023.E_{st}.L_0/f_y \qquad \text{Eqtn. 6.}$$

Now consider the second mode of failure, Figure 2, crushing of the concrete in the 'tension' face of the beam at the end of the exposed length close to a support. The chain dashed line in Figure 3 denotes the locus of the centroid of concrete in compression between midspan and support. The least distance between the locus and the face of the concrete is denoted 'e'. As before, it is assumed that beam strength will not be significantly reduced if reinforcement attains yield. The total force in the tension bars, P_{st}, is then :

$$P_{st} = A_{st}\ f_y \qquad \text{Eqtn. 7.}$$

and must be equal in magnitude to the compression in the concrete, P_c. Assuming the rectangular stress block of Fig. 5 for concrete, a limit is reached when

$$P_{st} = 0.67\ f_{cu}.b_w\ 0.9x_2 \qquad \text{Eqtn. 8.}$$

It may be assumed with acceptable accuracy that $e = x_2/2$. Therefore, in order to avoid a reduction in strength, the distance from the centroidal locus to the concrete surface, e, should not exceed

$$e = 0.83\ A_{st}.f_y/(f_{cu}.b_w) \qquad \text{Eqtn. 9.}$$

This analysis neglects the presence of 'compression' reinforcement. Limitations on space prevent a more detailed consideration of this aspect. However, compression bars will increase the length of bar which can be exposed without

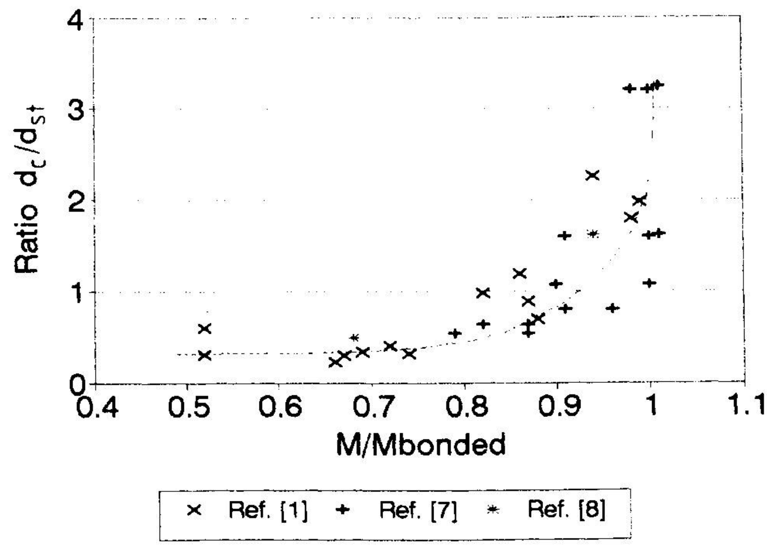

Figure 4 Comparison with test data.

significant loss of strength. Tests by Raoof[3] have confirmed that strength loss is reduced by provision of compression reinforcement.
It must also be verified that end anchorage of bars is adequate to develop the yield strength, Failure Mode 3, Figure 2, This may be done using standard Code of Practice procedures. If bars suffer from corrosion, due allowance should be made.

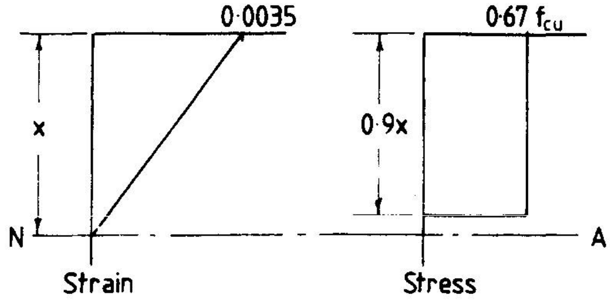

Figure 5 Simplified stress block for concrete at ultimate limit state.

Finally, it must be verified that shear strength is adequate within the portion of the member over which reinforcement is fully bonded. Normal Code of Practice procedures may be used. As mentioned above, there is no experimental evidence that shear strength is reduced by bond loss/exposure of reinforcement.

Partial safety factors on materials have been omitted in derivation of the above expressions, but should be included in any practical application.

The Author is not aware of any test data on strength of reinforced concrete beams with exposed bars under torsional loading.

4 CONCLUSIONS

The paper has demonstrated that it will be possible in certain circumstances to expose bars over a significant proportion of span without loss of strength, and simplified methods of analysis to estimate allowable exposure lengths have been presented. The expressions presented will assist in planning of repair programmes.

REFERENCES

1. Cairns, J., & Zhao, Z. Structural behaviour of concrete beams with reinforcement exposed. Proceedings Institution of Civil Engineers : Structures & Buildings. Vol 99. May 1993. pp141-154.
2. Leonhardt, F & Walther, R. The Stuttgart shear tests, 1961. Cement & Concrete Association Library Translation No. 111, London, 1964.
3. Raoof M & Lin Z. Implications of structural damage to concrete elements. Proc. Conf. on Bridge Management, Surrey Univ., UK. Ed. Harding J E, Park G A R, & Ryall M J. Thomas Telford, London. 1993. pp66-74
4. Cairns, J. Strength in shear of concrete beams with exposed reinforcement. Proceedings of the Institution of Civil Engineers. May 1995.
5. Eyre, J.R., & Nokhasteh, M.-A. Strength assessment of corrosion damaged reinforced concrete slabs and beams. Proceedings Institution of Civil Engineers, Structures & Buildings. Vol 94. May 1992. pp197-203.
6. Phipps, M E. The strain capacity of compression-zone concrete subjected to short-term loading. Magazine of Concrete Research, Vol 28, No. 95. June 1976.
7. Minkarah, I., & Ringo, B.C. Behaviour and repair of deteriorated reinforced concrete beams. Transportation Research Record. No. 821. 1982
8. Nokhasteh M-A, Eyre J R & McLeish A. The effect of reinforcement corrosion on the strength of reinforced concrete members. Proc Conf. on Structural Integrity Assessment, Ed Stanley, P. Elsevier, London, 1992. pp314-325

Implications of Exposure of Main Steel during Patch Repairs

Sollicitations d'une armature principale mise à nu lors de réparations locales

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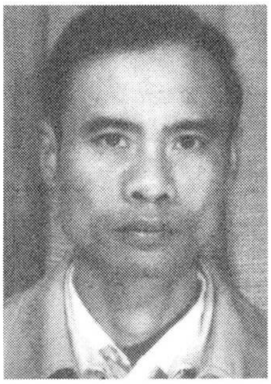

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SUMMARY

Results are reported for an extensive series of experimental works on 132 reinforced concrete beams in which loss of cover and exposure of the reinforcement in the course of the patch repair process has been simulated by forming recesses in the concrete elements. Both small and large scale beams with various extents of exposed reinforcement have been tested covering a range of structural parameters including percentage of tensile reinforcement, extent of removal of steel/concrete bond, distance of damage from the support, load position(s) relative to the support, depth of concrete removal, proportion of nominal top steel (in beams unreinforced in shear), effect of including shear reinforcement in the form of stirrups, and loading arrangement.

RÉSUMÉ

L'article rapporte les résultats d'une vaste série d'essais sur 132 poutres en béton armé. Une perte d'enrobage et une mise à nu des armatures pouvant se produire lors de réparations locales ont été simulées par formation de niches dans les éléments en béton. Des poutres à petite et grande échelles et avec des niches de différentes tailles ont été testées. Les paramètres structuraux comme le pourcentage d'armature passive, l'importance de la perte d'adhérence acier/béton, la distance du dommage aux appuis, la position de la charge par rapport aux mêmes appuis, la profondeur des niches, la proportion nominale d'armature supérieure (poutres sans armature d'effort tranchant), l'effet d'un renforcement de la résistance à l'effort tranchant par ajout d'étriers et la distribution de la charge ont également été variés.

1. INTRODUCTION

The previous limited literature as regards the loss of strength in reinforced concrete beams due to removal of concrete/steel bond for patch repairs was reviewed elsewhere [1]; it was concluded that there was a need for a systematic series of tests which covered a much wider range of parameters than those previously examined with particular emphasis on cases when the exposed steel area is located near the supports where loss of strength can be more critical.

Very briefly, although there is currently extensive literature on the causes of chemical attack and their effects on concrete and steel reinforcement on a material level and also on the means of reducing or eliminating such decays, there is very little available literature on the structural implications of, for example, removal of concrete/steel bond during the patch repair process. The aim of the present paper is to report details of an extensive series of tests (covering a wide range of design parameters) on, both, small and large scale reinforced concrete beams in which the main steel reinforcement is exposed by varying extents.

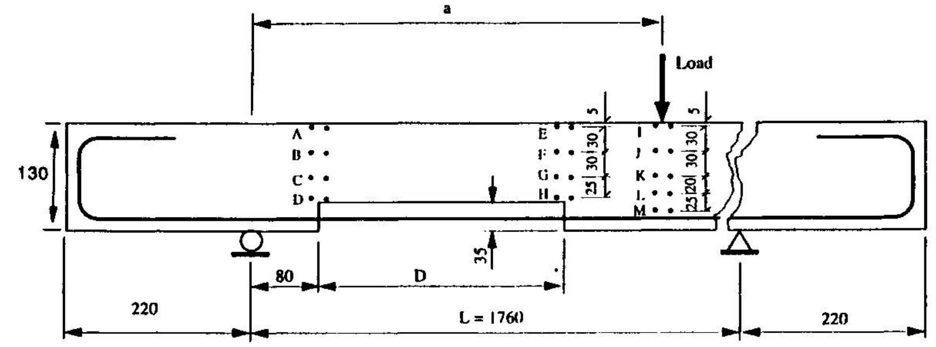

Fig. 1
Details of the small scale beams

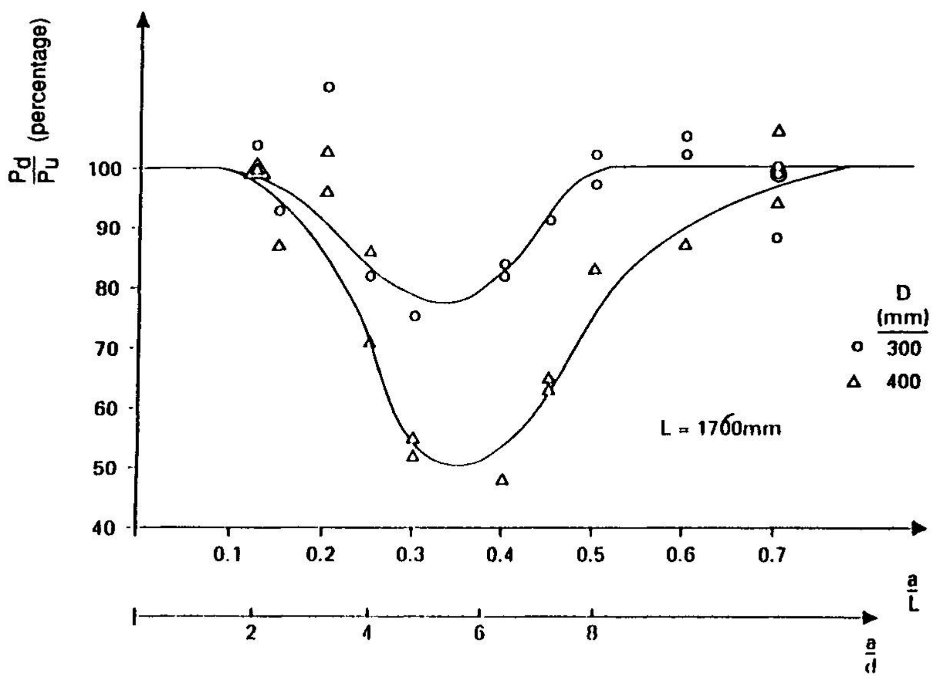

Fig. 2
Variation of the loss of strength with changes in the position of the single point load for two values of D = 300 and 400mm - small scale tests.

2. EXPERIMENTAL WORK

2.1 Small Scale Tests with No Shear Reinforcement

In these series of tests forty-four simply supported beams were tested in eleven sets of four. Each set consisted of one undamaged (i.e. control) beam and three beams in which the cover was omitted and the reinforcement exposed over a variable length near one end. Fig. 1 gives the beam details and the loading arrangement: all the beams were simply reinforced for bending and unreinforced in shear.

Fig. 2 presents plots of the loss of strength, P_d/P_u, where P_d = ultimate strength of damaged beam, and P_u = ultimate strength of undamaged beam in the same batch, against the parameters a/L and a/d (representing the load position), where d = effective depth, for two lengths of imposed damage, D = 300 and 400mm. It is concluded that as the extent of damage is enlarged, the loss of strength increases. The loss of strength, however, is also controlled by the load position: the latter appears to have a greater effect than the extent of damage, on the loss of strength. It is rather alarming that the percentage decrease in strength in Fig. 2, rises to values as large as 20 and 50 percent for extents of removal of concrete/steel bond D = 300 and 400mm, respectively. Space limitations do not permit a full description of the results (and indeed a full discussion): these are fully covered elsewhere [1].

2.2 Large Scale Tests with No Shear Reinforcement

In the next phase of the experimental programme, a total number of eighty-eight large scale beams were tested following a similar testing procedure to that adopted for the small scale beams, but covering a much wider number of potentially important parameters: these included variations in the loading position, a/L, percentage of tensile steel reinforcements, A_s/bd, extent of loss of cover and bond, D, relative position of the damaged section from the support, x, depth of removal of concrete, h, effect of including nominal top steel (in the absence of stirrups), loading arrangement, and finally the effects of including shear reinforcement in the form of stirrups. The large scale beams had overall dimensions of 150 x 300mm with an overall length of 3500mm. They were all tested as simply supported beams with a clear span of 3000mm under single (or two) point loading whose position(s) varied from beam to beam. In most cases, each batch of large scale beams consisted of one undamaged (control) beam and three beams in which the cover and bond to main steel reinforcement was removed over a variable length, D. Three levels of D, namely, 300, 600 and 900mm were investigated.

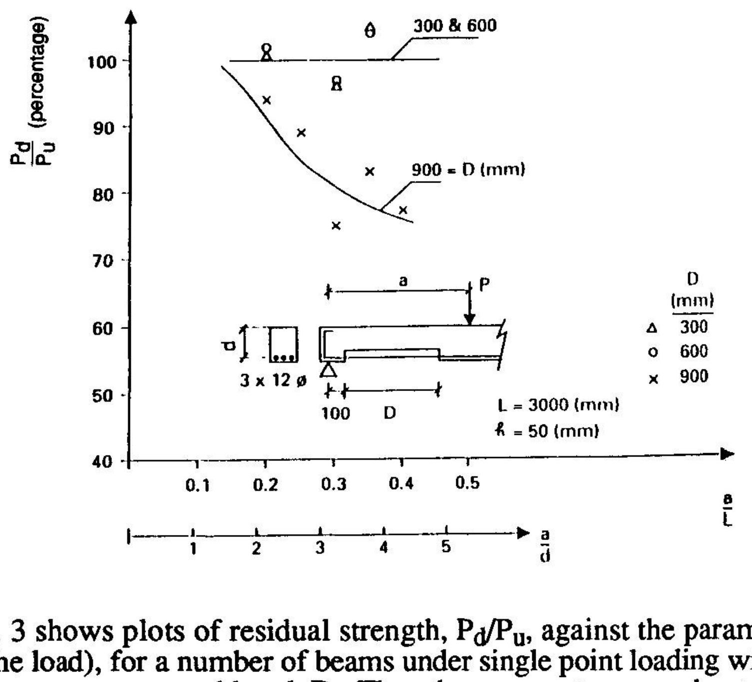

Fig. 3
Variation of the loss of strength with changes in the position of the single point load for three values of D = 300, 600 and 900mm - large scale tests.

Fig. 3 shows plots of residual strength, P_d/P_u, against the parameter, a/d (or a/L) (defining position of the load), for a number of beams under single point loading with three different extents of removal of concrete cover and bond, D. The other parameters were kept nominally constant. The beams had no shear and/or top reinforcement. The plots in Fig. 3 show no loss of strength for values of D = 300 and 600mm, over the wide range of 2 < a/d < 4. Only for D = 900mm some loss of strength with a value of 23% at a/d = 3.23 is found. The question then arises as to what has caused such a drastic change of behaviour between small and large scale beams with the results for the small beams, Fig. 2, giving much higher levels of loss of strength. Test data presented in Fig. 4 provide the necessary clue and support the suggestion that rather significant reductions in the magnitude of the residual strength, P_d/P_u, can occur with increasing levels of the percentage of tensile steel. Here, values of measured P_d/P_u are plotted against the percentage of steel in the range 0.75% < A_s/bd < 1.2% with the other parameters kept nominally constant. Test data in Fig. 4 suggest that increases in the percentage of tensile reinforcement can lead to substantial increases in the percentage loss of strength. Noting that the percentage of tensile reinforcement in small-scale beams of Fig. 2, and large scale beams of Fig. 3, were A_s/bd = 1.61% and 0.75%, respectively, it is concluded that the much larger values of loss of strength in small scale beams (c.f. larger ones) has been mainly due to their significantly higher percentage of tensile reinforcement, with the relative values of h = 35 and 50mm in small and large scale beams, respectively, playing a role.

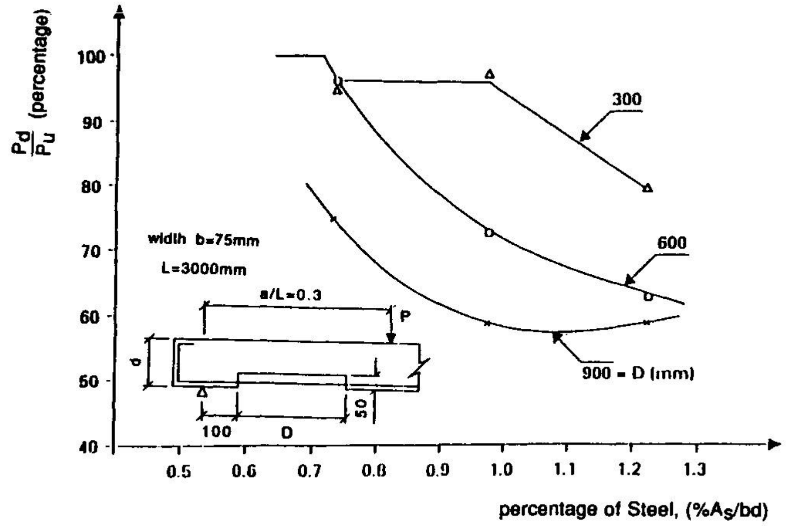

Fig. 4
Variation of the loss of strength with changes in the percentage of tensile steel for D = 300, 600 and 900mm - large scale tests.

The effect of varying the depth of removal of concrete cover, h, on the percentage loss of strength, P_d/P_u, is shown in Fig. 5. All the data in Fig. 5 were obtained for beams with x = 100mm, a/d = 3.23 (or a/L = 0.3) and A_s/bd = 1%. From data in Fig. 5 it is concluded that for the beams tested, the parameter P_d/P_u is only mildly sensitive to changes of h below 40mm. Values of h greater than 40mm are found to lead to sudden reductions in the measured values of residual strength with the reductions being more severe for increasing levels of the extent of damage, D. As mentioned previously, from plots in Fig. 5, there appears to be a threshold value of h above which (depending on concrete strength) substantial reductions in strength can occur. The value of h = 28mm in Fig. 5 corresponds to a nominal 1mm gap between the steel and concrete simulating total loss of bond. In, for example, spalling conditions, it appears that even in extreme cases of total loss of bond there may be no significant adverse effect on ultimate strength. This observation may prove of value to those interested in the assessment of corrosion damaged structures where partial bond may still exist.

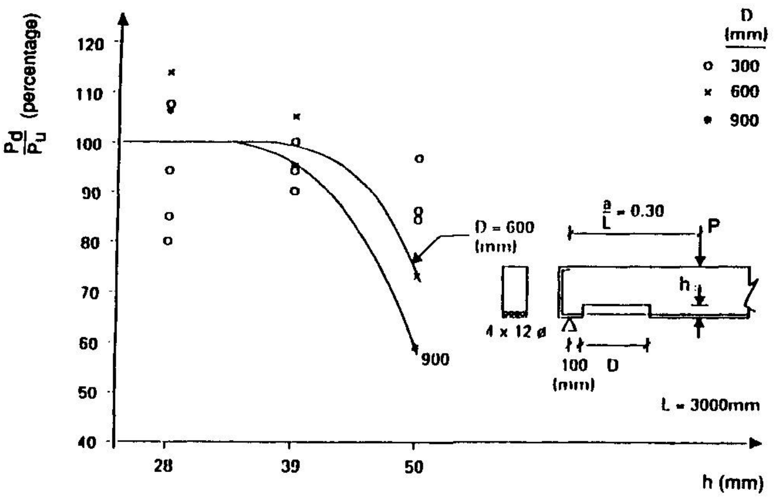

Fig. 5
Effect of varying the depth of removal of concrete cover, h, on the percentage loss of strength in large scale tests for D = 300, 600 and 900mm.

Fig. 6 shows the variation of P_d/P_u with the parameter, x, which defines the location of the region of exposed reinforcement (recess) from the simply supported end. Results in Fig. 6 suggest that, keeping other parameters constant, the closer the region of exposed reinforcement is to the support, the higher will be the magnitude of the loss of strength.

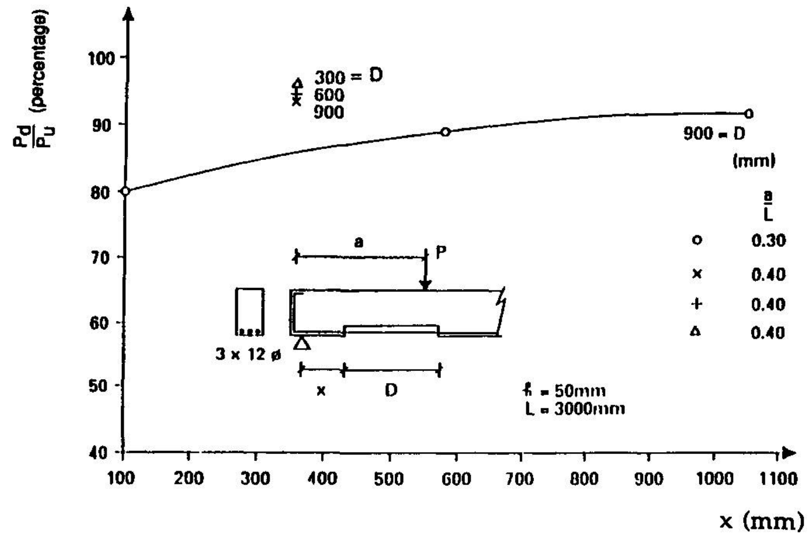

Fig. 6
Variation of the percentage loss of strength with the location of the simulated damage area (recess) with respect to the simply supported end, x, for D = 900mm - large scale tests.

2.3 Large Scale Tests with No Shear Reinforcement but with Nominal Top Steel

The measured patterns of direct strains over the depth of both the small and large scale beams whose results are presented in Figs. 2-6, suggested that the concrete at certain locations along the beams is in tension above the so-called Neutral Surface and in compression below it over, for example, section A-B-C-D in Fig. 1. Furthermore, a number of beams exhibited a tensile crack running vertically downwards from the top surface which was located in the vicinity of section A-B-C-D in Fig. 1.

A series of tests were carried out with nominal top steel placed in the beams. All the beams had h = 50mm, x = 100mm with four 12mm diameter main reinforcing bars. Only the extent of exposed main reinforcement, D, and diameter of top nominal steel was changed from test to test. In all cases, the number of nominal top steel bars was two with their diameter changing from 6-10mm in equal increments of 2mm. Such small diameter bars are similar to those used in practice for holding stirrups in place with their contribution to the structural behaviour of beams with no exposed reinforcement invariably ignored in design. The beams were all tested under single point loading with a load position a = 900mm (or a/d = 3.23). It is most interesting that in all cases the measured residual strength was found to be raised very substantially with certain beams giving P_d/P_u = 143% or 137% (once top steel was included). Reinforcing the top surface of beams with exposed main reinforcement was, therefore, found to be beneficial (in terms of raising the ultimate strength of damaged beams by some substantial margins). Variations of the diameters of the top steel bars within the range 6-10mm was not found to make any significant difference in terms of the measured P_d/P_u ratios. It must be noted that although inclusion of nominal top steel was found to virtually remove any loss of strength due to loss of concrete cover and bond (at least under single point loading), certain beams with exposed main steel exhibited brittle failure with others failing in a ductile fashion.

The discussions in the preceding sections have solely addressed the case of single point loading on simply supported beams. Fig. 7 shows a plot of the residual strength, P_d/P_u , versus load position for the case of simply supported beams (with no top steel and/or shear reinforcement) experiencing two-point loading. Full details of the corresponding test results are given in Ref. [1] where residual strengths as low as 52% were recorded with all the damaged beams failing in a brittle fashion.

A further series of two-point loading tests were carried out (with a/d = 4.30) on beams with D = 900mm, h = 50mm, x = 100mm and four 12mm diameter main tensile bars. Top steel was included in beams whose corresponding residual strength in the absence of top steel was $P_d/P_u \approx 53\%$: full set of test data is given in Ref. [1]. Out of the four beams tested one had no nominal top steel with the other three beams each having two nominal top bars with diameters of 6, 8 and 10mm. It was most interesting to note the increases in strength of 177%, 185% and 149% (c.f. to the beam with no top steel) for the three beams which had nominal top steel. It should, however, be noted that none of the beams exhibited a ductile mode of failure.

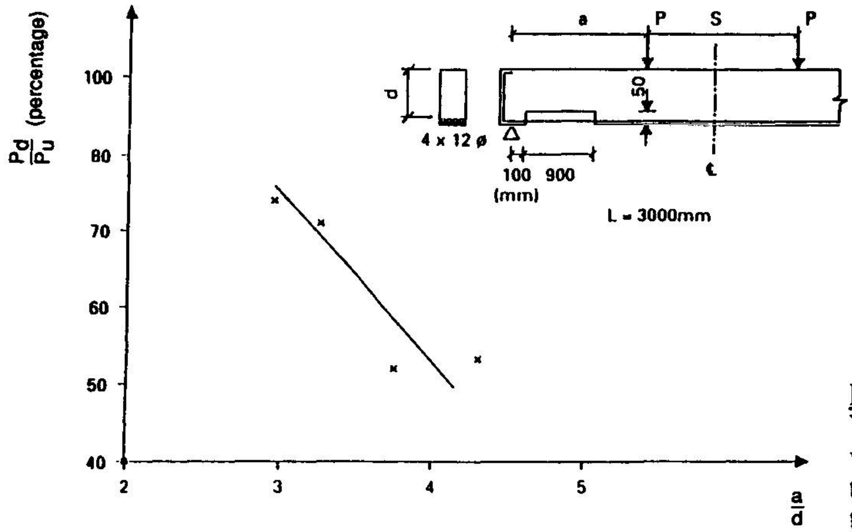

Fig. 7
Variation of the loss of strength with changes in the position of two-point loadings - large scale tests.

2.4 Large Scale Tests with Shear Reinforcement

Ref.[1] also presents a full set of test results for 8 simply supported beams with nominal top steel and also stirrups, tested under two-point loading. Four loading positions S = 200, 600, 900 and 1200mm, (where S is defined in Fig. 7), were chosen and for each load position, one control beam and one beam with exposed main reinforcement over a distance of 900mm was tested, with h = 50mm, x = 100mm, and four 12mm diameter tensile bars. No significant loss of strength (due to removal of concrete cover and bond) was found in all cases. The observed failure mode in all cases was ductile. A detailed examination of the test results suggested that in terms of reducing the strength loss, the nominal top steel (such as those which are invariably used for holding stirrups in place) has the major influence as compared to the stirrups themselves. Over the range of test parameters used, the loss of strength for all beams containing both nominal top steel and stirrups was minimal. The presence of stirrups was, however, found to ensure occurrence of ductile failure as opposed to certain cases of brittle failures in connection with beams which had nominal top steel and no stirrups.

3. CONCLUSIONS

Results from a number of small and large scale tests on reinforced concrete beams are reported which cover a wide range of potentially important parameters in terms of the structural performance of an RC member weakened by removal of concrete and steel/concrete bond.

Increasing the percentage of tensile steel (in the absence of stirrups and nominal top steel) has been found to increase the observed percentage losses of strength. Moreover, the ultimate loads of beams with exposed reinforcement were found to change dramatically by using nominal top steel, the introduction of which was found to reduce the strength loss. Tests showed that, in terms of ultimate strength, the nominal top steel (such as those which are invariably used for holding stirrups in place) was having the major influence as compared to stirrups. However, presence of stirrups was found to cause a ductile failure in beams which even in the presence of nominal top steel exhibited a brittle failure. Over the range of test parameters used, the loss of strength for all beams containing both nominal top steel and stirrups was minimal.

REFERENCES

1. RAOOF M. and LIN Z., Loss of Strength in Reinforced Concrete Beams Due to Exposure of Reinforcement. Synopsis published in The Structural Engineer, Vol.72, No.14, July, 1994, with the paper filed in the library of the Institution of Structural Engineers.