ADVANCED COMPOSITE MATERIALS IN BRIDGES AND STRUCTURES
MATÉRIAUX COMPOSITES D'AVANT-GARDE POUR PONTS ET CHARPENTES
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ECONOMIC VIABILITY OF STRUCTURES WITH FRP REINFORCEMENT AND PRESTRESS

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ABSTRACT: Most of the technical problems associated with the use of FRPs have been addressed; they offer a good technical solution to the problems of corrosion of reinforcement. However, apart from the use of glued-on CFRP strips for flexure, FRPs are not finding their way into mainstream use. The reason is economic, since the first cost of the materials is higher than steel and the discount rates used to produce NPV mean that future repair costs have negligible effect now. Industry's perception is that FRPs are uneconomic for newly-built structures, so they are not used. In this paper the true cost of steel corrosion is addressed, using realistic models for chloride and water ingress, traffic delay costs and realistic repair costs; an appropriate discount rate is also discussed. These models are applied to the survey of 257,000 bridges in the US and predictions are made for the money that could be saved if future bridges were built using non-corroding FRPs. Unlike present calculations which (conveniently for the politicians) show that cheapest first cost is also the cheapest whole-life cost, the results show that additional money spent now on improving the durability of structures is a very sound investment.

1. INTRODUCTION

Steel corrosion threatens the durability of concrete structures and vast, and increasing, budgets are currently spent for repair. The use of FRP in concrete is one of the alternative methods to address the problem of corrosion. Even though research has been extensive in the last two decades, applications are only met in prototype structures. Industry hesitates to consider those materials as an alternative to steel due to its high price. In this paper methods to study the economics of those materials are presented and conclusions on their viability in bridges are drawn. For their assessment both the initial and the life cycle costs are considered.

2. INITIAL COST

A method was developed to study the initial cost of a concrete bridge structure with steel or FRP; the method can identify optimum design solutions for reinforced and prestressed concrete bridge structures. Its applications to prestressed bridges can be found in [1] and a detailed description of the method is given in [2]. The method identifies design constraints and the results can be plotted on a section depth versus reinforcement area diagram which gives a zone of feasible solutions (Fig. 1). The cost of the structure appears as a line on the diagram and the optimum solution can be found using well known non-linear optimisation methods or by simply observing the (d, A_p) diagram. In general the results showed that:

- When steel was used either in reinforced or prestressed applications, design solutions are cheaper than FRP-concrete structures on a first cost basis.
- The optimal design of FRP is not sensitive to small changes in the relative cost of FRP and concrete, unlike designs in steel where cost variations alter the optimal design.
- The FRP-snapping mode of failure governs the design. It has to be avoided since it is sudden and catastrophic, due to FRP's purely linear nature. This normally governs the FRP area that is needed in the cross section, controlling design solutions and their cost (constraint p_{min}).
- The ultimate moment constraint rarely the governs for all FRP cases. Using FRPs in the form of shear stirrups is not effective as FRPs are weak in dowel action and current design recommendations are conservative especially for the most flexible FRPs. New forms of FRPs as shear reinforcement need to be developed.

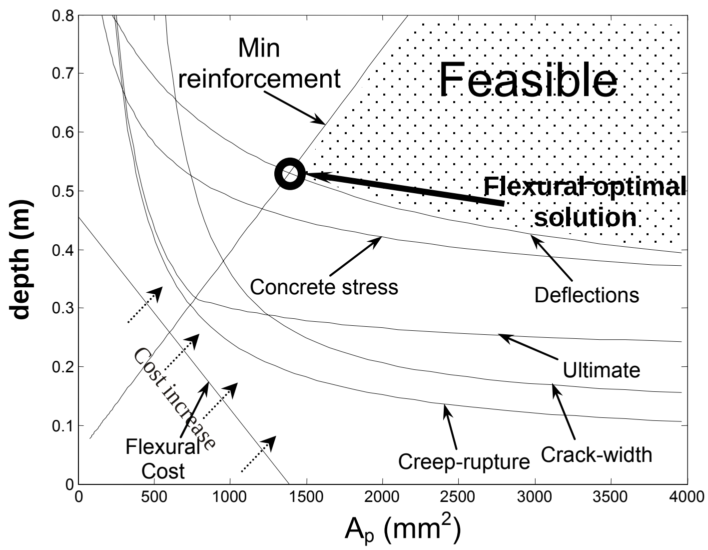

Fig. 1 – Typical design chart for reinforced FRP beam

For reinforced concrete:

- Deflection limit states govern solutions because FRPs possess relatively low stiffness.
- The optimum solutions were normally deeper than with steel reinforcement to satisfy the stiffness constraints. For FRPs with poor bond properties the crack width limitation was violated pushing the design solution to more expensive areas on the (d-A_p) figure.
- The option of using stronger concrete restricted deflections, but more FRPs were needed to eliminate the FRP-snapping mode of failure and the resulting cost was not reduced.

For prestressed concrete:

- Deflections were less of a problem.
- The optimum solution was governed by the tension working stress and p_{min} constraints.
- Improving bond increased cost as tendons snap more easily; completely unbonded beams gave the cheapest solutions.
- The FRP/steel first-cost ratios increased for longer spans.

3. STRUCTURAL LIFETIME

The initial cost study showed that concrete structures reinforced with steel are less expensive than those reinforced with FRP when only initial costs are considered. If steel is likely to corrode then non-corrodable materials like FRPs can be viable alternatives. But under which environmental conditions is steel likely to corrode? A model was developed to determine the lifetime of steel-reinforced concrete bridges as a function of the environmental exposure. Corrosion was assumed to be caused by the use of de-icing salt but could be from sea-spray. Other types of corrosion, for example carbonation, are less likely to occur [3],[4]. The end of the bridge lifetime was assumed to be when horizontal cover cracks appeared; corrosion of steel reinforcement then accelerates and public pressure would be applied to the authorities for the bridge to be repaired. The structural lifetime was defined as the sum of two periods: the time to corrosion initiation and the time to cover cracking (Fig. 2).

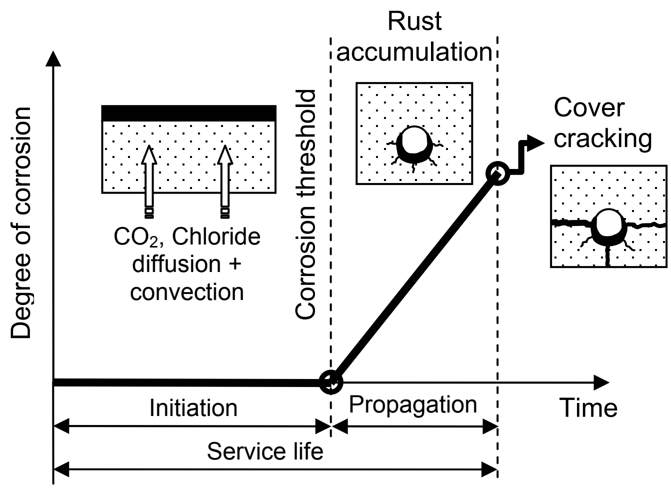

Fig 2. – Progress of corrosion

3.1 Time to corrosion initiation

Corrosion initiation is assumed to occur when the chloride concentration at the level of the bars reaches a threshold value. Chlorides migrate inwards by diffusion and convection and chloride binding is introduced with the use of linear and non-linear chloride binding isotherms. Humidity diffusion and heat transfer are included; the chloride and humidity diffusion coefficients (D_c and D_h) are functions of humidity, temperature and age, based on existing experimental data.

The environmental conditions are introduced into the problem through the boundary conditions. Existing experimental data are incorporated to follow what really takes place at the transition between concrete and atmosphere. For that purpose annual chloride concentrations, humidity and temperature fluctuations are introduced including chloride diffusion-convection in and out of the cover and chloride wash-off due to rainwater. The resulting differential equations are solved using finite differences.

3.2 Time to cover cracking

Active corrosion starts after corrosion initiation; this is an electrochemical reaction, which is dependent on temperature, humidity and chloride content. The volume of rust products is greater than steel; part of the rust fills the pores around the bar, but the rest generates radial pressure, causing radial cracking. The cover is assumed to be fully cracked when the remaining un-cracked ring is unable to sustain the pressure exerted by the corrosion products.

The results from the corrosion models show that:

- Time to corrosion initiation is short in environments with wide annual temperature fluctuations. Low temperatures force the authorities to use de-icing salts; during winter, chlorides migrate slowly but during summer elevated temperatures cause much more rapid chloride movement. High average annual humidities accelerate chloride migration.
- The time to cover cracking is shortest under high average annual temperatures and average annual humidity values. When humidity is high, near-saturated conditions mean oxygen is not readily available at the bar depth.

A sensitivity analysis shows that:

- The time to corrosion initiation can be extended if less porous concrete is used and by replacing part of cement with PFA or GBFS. Concrete with porosity reduced to very low values should be used though with caution as oxygen availability at the bar surface is restricted and black rust can be formed.
- Replacing cement with C_3A enhances the concrete's capacity to bind chlorides and increases the time to corrosion initiation. This is also not the solution to steel corrosion as binding capacity is limited and high cement replacements with C_3A may encourage the alkali-silica-reaction to take place in the concrete.
- Stronger concrete can crack more easily, despite its higher tensile strength; its low porosity means that considerably less rust is needed to fill the concrete microstructure which means more rapid build-up of internal pressure on the cover.
- Time to crack initiation was only a fraction of the total time to cover-cracking. Most of the time the cover was partially cracked but the cracks had not reached the surface of the element.

4. DELAY COSTS

When a bridge is to be repaired lanes must be closed for repair work to proceed, which causes delay costs. A model has been developed to determine those expenditures. Real traffic data from various roads in UK were used as input. Diversion routes were assumed to be available and the equilibrium principle was used to distribute the traffic between the main and diversion routes.

While vehicles travel through the site several costs arise: queue delay costs (time lost waiting in the formed queues), reduced velocity costs, vehicle operating costs and accident costs. The queue delay costs normally represented the biggest portion of user delay costs. To evaluate them, three methods from transportation engineering were used:

- the cumulative plot
- the modified cumulative plot, and
- the shock wave method.

In Figure 3 the queue length development for a Monday on the UK's M25 motorway (junction 10) is shown. The motorway has 3 lanes in each direction, of which 2 were assumed closed between 0:00-6:00 and 19:00-24:00, and one from 6:00-16:00, with none closed during the evening peak 16:00-19:00.

The results showed that:

- User delay costs represent the largest portion of the life cycle costs. They are often so high that even if they occur at a relatively long time from bridge construction they can justify very high initial costs; the only concern in the initial design should be to construct a corrosion-free bridge. The method to repair a corroded bridge should be based on the time for repair, rather than the cost.
- High user delay costs are not a function of the road size and can be high for any road type.
- The three methods which exist in the literature to determine queue delay costs give similar results and the simplest cumulative plot method gives sufficient accuracy for making decisions about bridge design.

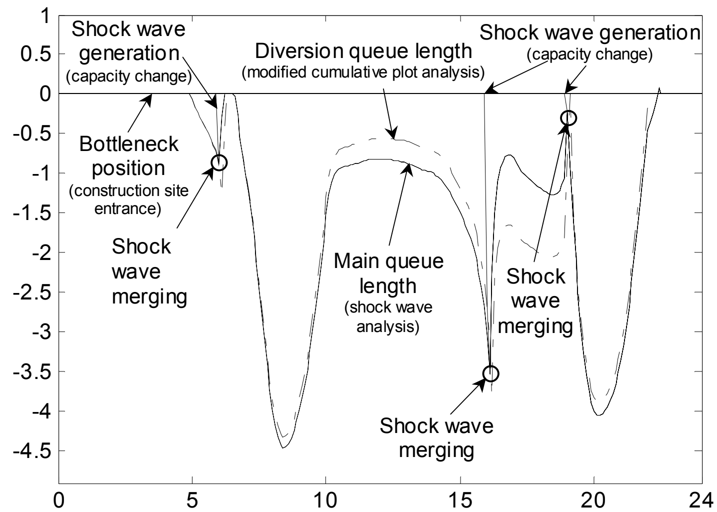

Fig. 3 – Predicted queue lengths caused by bridgeworks on M25

5. USA CURRENT BRIDGE STOCK CONDITION – FUTURE PREDICTIONS

In 2002 a study of the condition of all bridges in the USA was published [5]. It is the result of a study by the US Department of Transport - Federal Highway Administration, which reported the state of all bridges using a standard format. The data for the bridge is available electronically, which allows further analysis to be carried out, as has been done here. The sample consists of 257,235 bridges made from concrete, of which 137,961 are reinforced and 114,795 prestressed with steel bars. The data describe the physical condition of all bridge structural elements, which were assessed in the year 2002. The condition of bearings, joints, paint system etc. were excluded from the data presented below. The condition is described on a scale 0-9 (9 for excellent and 0 for failed condition).

Figure 4 shows the number of concrete bridges built annually in the last 100 years in the USA. In the last 40 years about 4500 bridges are built each year. From the data the number of concrete bridges at various condition ratings can be determined. By dividing the number of bridges in each condition by the total number of bridges built each year, the cumulative probability that a bridge is in a certain condition can be found and is presented in Figure 5. It is assumed here that a bridge will need repair when it reaches State-4 (poor condition: advanced section loss, deterioration, spalling or scour).

It is well established in the literature dealing with the lifetime of products that data for various conditions can be shifted to give a smooth 'master curve' that predicts future behaviour [6]. Thus, bridges which are now in State 5 can be expected to reach State 4 after a characteristic delay; those in State 6 will reach State 4 later. The criterion for choosing the time shift is merely to obtain a smooth curve for condition 4 and is, to some degree, subjective (Fig. 5). Polynomials can then be fitted to the predicted data; the slope of the cumulative probability is the bridge condition probability.

The probability for bridges in State 4 is applied for the bridges built up to the present time and for 3 options for future bridge construction. The bridge construction cases are as follows:

- case 1: none of the bridges to be built in the future need repair in the next 120 years,
- case 2: 3000 bridges are built every year with steel bars using current practice.
- case 3: 4500 bridges are built every year with steel bars using current practice.

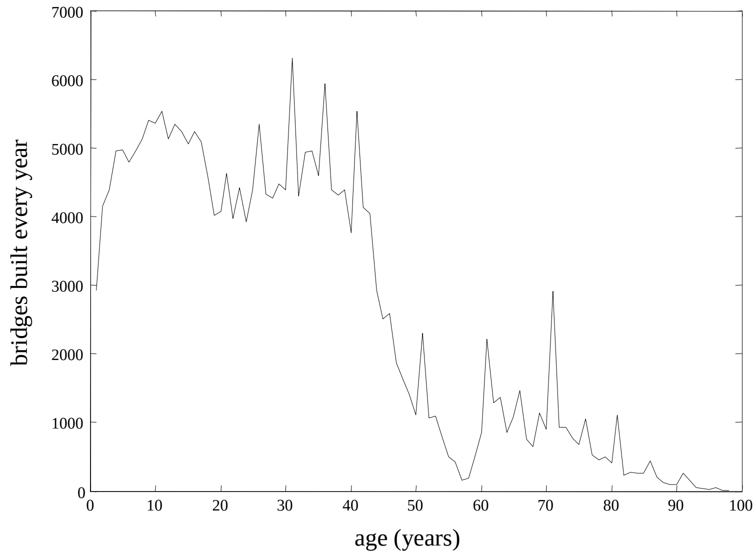

Fig. 4 – Age of Concrete Bridges in USA

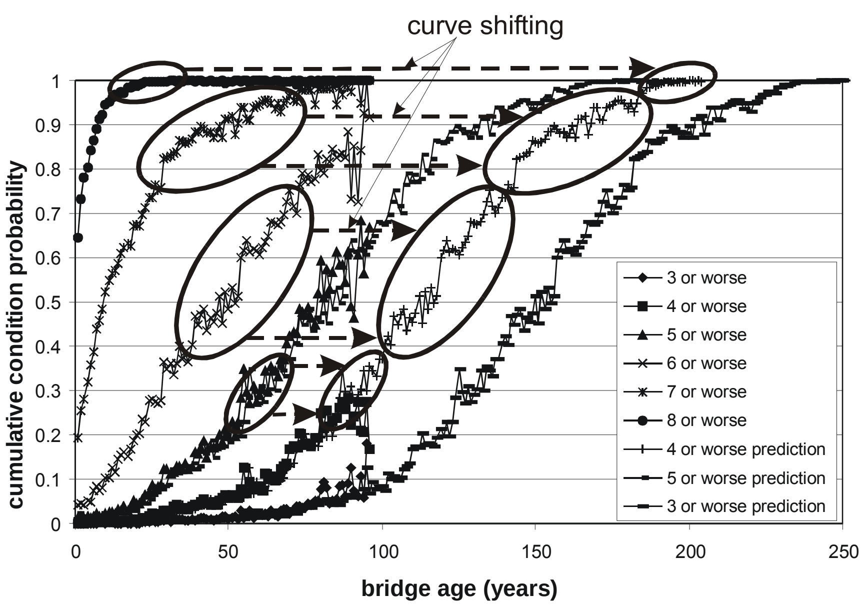

Fig. 5 – **Existing and future condition prediction for US bridges**

The number of bridges that will reach condition 4 and thus need repair in the next 120 years can be predicted. The model predicts that a considerable number of existing concrete bridges have to be repaired annually in the future. Existing bridges are already marching to their doom, and there is little that FRPs can do for them. But costs associated with corrosion of bridges built in the future can be saved if suitable decisions are made now. Figure 6 shows the additional predicted number of bridges that need repair using steel bars (cases 2 and 3) in comparison to a scenario of corrosion-free new bridges for a 120 years period (case 1).

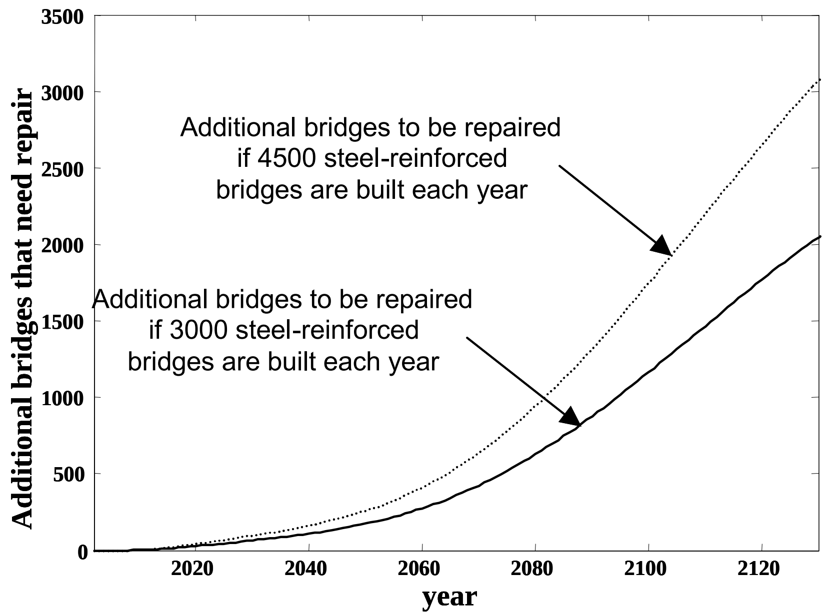

Fig. 6 – Additional bridges to be repaired in the US if future bridges are built with steel

If it is assumed that one third of the new bridge stock is on one-lane roads, another third on 2-lane roads and the remainder on 3-lane roads (in each direction). It was found that high user delay costs can arise in any road; user delay costs are assumed to be the same for all the roads and equal to £100,000 per day in 2003 prices, which is at the low end of the range of values obtained using official UK operating costs, and assumes adequate diversion routes. It is assumed that all future bridges will take 2 months to repair [6]. Thus total user delay costs for a bridge will be £6,000,000. Traffic management costs will also arise and are assumed to be £20,000, £40,000 and £55,000 at 2003 prices, for 1-lane, 2-lane and 3-lane motorway, respectively. Bridge repair consists of specialised work and the condition of the bridge at the time of repair, and are calculated following current official figures published in UK; average repair costs for 1-lane, 2-lane and 3-lane roads are assumed to be £70,000, £120,000 and £170,000, respectively, at 2003 prices.

Table 1. Net Present Value of US bridge stock under different bridge building scenarios.

Real discount rate [%]	NPV case 1 [£.10^9]	NPV case 2 [£.10^9]	NPV case 3 [£.10^9]	ΔNPV case 2 - case 1 [£.10^9]	ΔNPV case 3 - case 1 [£.10^9]
2	366	492	556	127	190
4	149	168	177	18.8	28.2
6	82.4	87.6	90.2	5.2	7.8
8	54.3	56.4	57.4	2.1	3.1

The costs of Case 1 are all associated with the existing bridge stock, but the additional costs for Cases 2 and 3 represent the additional money that should be set aside now to repair bridges that are still to be built using corrodable materials, and are costs about which sensible decisions can be made. Figure 12 shows these avoidable costs. The totals are shown in Table 1; note the substantial influence of the discount rate on the final costs. With a high real discount rate (interest costs minus inflation), the costs of using steel probably are small – first initial costs govern. But if realistic discount rates are used (~2%), future repair costs are real; total whole-life costs are very significant and should be taken into account.

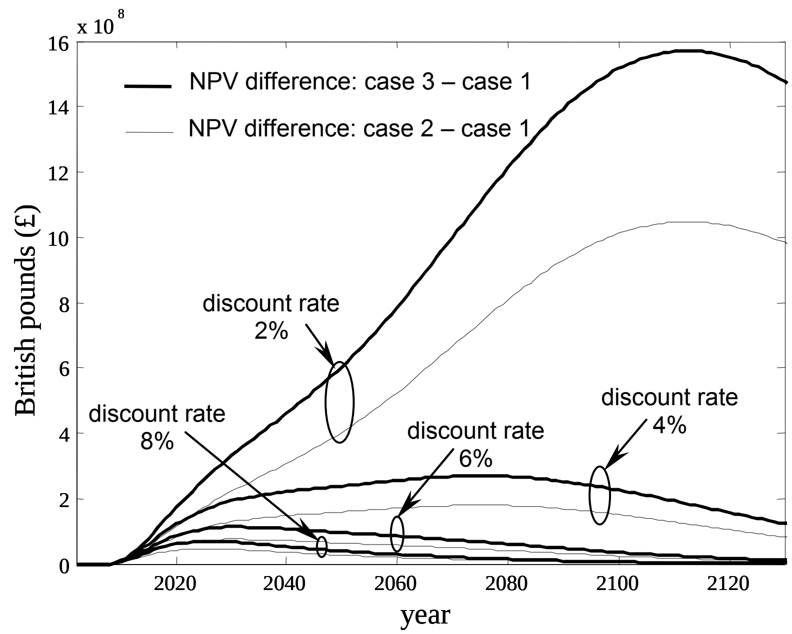

Fig. 7 – **Net Present Value of not building new bridges in the US with FRP.**

6. CONCLUSIONS

A large number of assumptions have had to be made in determining the values that go into this paper, but it is very clear that there would be considerable present value in adopting strategies that made bridges more durable. The additional costs of using FRP would be small in comparison to the costs shown here. The benefit will only be realised if realistic whole-life costing policies are adopted.

7. ACKNOWLEDGEMENT

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