WHY IS FRP NOT A FINANCIAL SUCCESS?

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1 INTRODUCTION

When FRPs were first introduced in the 1980s, they were thought to have great potential. They were usually stronger than steel but less stiff. They had higher strain capacities, so if they were going to be used to their full potential they would have to be prestrained; prestressing tendons were clearly a better idea than reinforcing bars. Costs were calculated that were, typically, 5 or 6 times the cost of steel on a cost/unit-force/unit-length basis [1] but, it was argued, those costs applied to materials in development and they would surely fall rapidly as manufacturing technology and the competitive market developed. The light weight of the materials was only of marginal interest; there may be military bridging applications in which soldiers have to carry tension members in hostile environments where the cost can be justified [2], and there are some temporary bridging examples where access is difficult and components have to be carried or helicoptered into place [3]. Only for very-long-span bridges is light weight such a significant advantage that it is likely to change the economic solution. Twenty years later, has the situation changed markedly?

2 REPAIR MARKET

The one success of the FRP industry has been its use for repair and strengthening. This was not one of the markets originally envisaged for FRP, but it is the only one which has been successful. There are undoubtedly cases where repair with CFRP is both necessary and successful. But it is worth asking why this market has taken off when others have not?

One reason for the success is the light weight. A moderate saving in weight is not normally a benefit to civil engineering structures; the public's perception of solidity goes with mass and lack of movement and in most cases we have robust ground to take the additional weight. For repair, however, the benefit comes from the reduction in handling costs; despite additional material costs, FRPs are easier to install. So the cost benefit comes up-front – it is cheaper *now*, not at some undefined time in the future.

Secondly, the client has reassurance that the structure will not be made worse by the repair. The structure is clearly there; it may have problems; the steel may be rusting and it may not be able to carry the intended load, but it can self-evidently carry its own dead weight. Even if these "*fancy new materials*" turn out not to be as good is claimed, the structure will be no worse than it is now. The client's engineers and the local politicians are seen to be "*doing something*". They can put the blame on previous generations of engineers ("*if only they had allowed for deterioration of the materials/increased truck weights*") and politicians ("*those cheapskates wouldn't spend a few percent more to get a properly-built structure*") and show that they are helping society now ("*we are building a better Britain/America/Greece* (delete as applicable)"). By reinforcing the structure the engineer is seen to have done something and it is certainly going to be better than it was before. It is a "*safe*" decision in the sense that it is very difficult to criticize.

The structure may not have needed repair in the first place, but that does not matter. Many structures that are now being repaired were designed in the 1960s using hand methods of design. The stress distribution was assumed and satisfied the lower bound theorem of plasticity, on which all designers rely, even if they can't remember what it says. Nowadays, structures are checked using finite element analyses, which make a host of assumptions of which the checker is unaware (relative stiffness of the structure, articulation of joints, etc) and come up with a different stress distribution that the existing reinforcement cannot carry. The stress distribution found by the finite element analysis is only one of the distributions which satisfies equilibrium, so it does not mean that the original design was unsatisfactory, only that the modern checker cannot prove that it is OK [4]. This is a major problem for slab bridges but an even worse problem for masonry arches [5].

Even if the structure failed its assessment, would it have fallen down? In most countries, structures are assessed against the design code [6], which makes assumptions about the materials that will be used in a structure yet to be built; in the UK there is an assessment code which takes account of the prior knowledge of the structure as-built [7]. In reality, the average structure is between two and three times as strong as the load for which it was designed; design codes require pessimistic assumptions about the variability of loads and the strength of materials, and then add factors of safety to allow for uncertainty about the methods of analysis and to protect the engineer's professional indemnity insurance. How many highway authorities really design their bridges to fall down if a truck marginally over the 44 tonne limit (or whatever the design load is) crosses the bridge. The codes assume that some trucks will be overloaded and the real design load is considerably higher.

The repair market has reached the stage where adding CFRP is seen as the logical thing to do [8]. It has even been described as wall-papering, with clearly-bad examples, such as re-entrant corners, being repaired. Structures are now being repaired in many ways that defy our standard procedures. Student engineers are taught that steel links have to be properly anchored; the lap in a steel link must be detailed so that it occurs in the compression zone of the beam to ensure adequate anchorage. Most beam and slab structures are detailed so that the neutral axis is at, or just above, the beam/slab interface. If CFRP shear reinforcement is placed on the outside of the web, how can it possibly be properly anchored? There are clever techniques using longitudinal rods [9], or drilling into the concrete, but even so the anchorage would normally be regarded as inadequate using the rules that apply to steel. They are often anchored into the cover, which is presumably suspect because of corrosion of the shear steel and which is presumably cracked in tension. The use of self-anchored prestressed external strips is possible [10], although the required saddles can be a disadvantage.

By far the largest component of the repair market has been the reinforcement of columns against seismic action, with the reinforcement in the hoop direction at the ends of the columns [11]. In the UK, where large earthquakes are not expected, there has been considerable use of these techniques to reinforce bridge columns against vehicle impact, which is believed to be a significant danger [12]; this requires axial reinforcement at mid-height. FRPs have also been used for the reinforcement of old masonry structures, most typically in Italy and Greece. The collapse of the roof of the cathedral of St Francis at Assissi [13] is the sort of thing that must be prevented, but some of these techniques involve the application of point loads to domes, which seems to be a highly suspect technique. There is also considerable research in the US and elsewhere on the use of FRP to protect buildings from explosive damage.

To the best of the authors' knowledge, at the time of writing (September 2006), there has not yet been a major earthquake affecting structures that have been reinforced with CFRP. This market will depend critically on the first such event to occur. If the CFRP does protect the structures, the market is secure, but if *any* of them fail, even if the earthquake were much stronger than expected, the market will be killed overnight. This puts an extra onus onto designers working with the material today. One badly designed or shoddily constructed application can ruin, not just the people involved, but the whole industry.

3 FRP FOR REINFORCEMENT/PRESTRESSING

The repair market was unforeseen when FRPs were first developed. Prestressing was seen as the logical use, where the high strain capacity and high strength would be a great advantage, and the resistance to corrosion would reduce the risk of the sudden failures which had occurred in some places [14]. There is still a large research interest in FRPs, highlighted by over 400 papers being submitted to this conference. There have been a significant number of demonstration projects – some use FRPs in association with steel as a back-up but many rely on FRPs – the technology works. But it is not selling; the key questions are these:-

- How many structures have been built where FRP reinforcing bars or FRP prestressing tendons have been supplied as the material of first choice without subsidy either by the material supplier trying to develop the market or as part of a national research programme?
- Would clients specify FRPs in normal circumstances?

It was long argued that the reason these materials were slow to take off was that there were no codes that could be applied. This should have been a warning sign – if a new material or a new technique offers significant advantages it forces its way onto the market without a code. Codes follow technology, they do not lead it. Those who understand its potential make it work – the code allows others access to the technology. Various organisations have produced documents or are in the process of doing so; despite the genuine efforts of their authors they are all to some extent flawed, since test data is not available, particularly on the long-term properties, and the products are not yet standardized. Fibres, resins and production techniques are all changing – in the case of proprietary products, sometimes without the knowledge of the end user. Manufacturers will offer some sort of guaranteed minimum property, usually short-term strength, rather than compliance with some not-yet-written standard. But was it really the absence of the codes that meant the materials were not being adopted?

There have been a number of other factors that have affected the adoption of FRP. Reinforcement is seen as simpler to use than prestressing. Some clients say that once FRPs have been established as reinforcement they would consider them for prestressing, ignoring the fact that rebar is not a sensible application of FRP.

There have also been changes in clients' procurement processes. In many countries the days are gone when a government department would be willing to have a programme of innovation, with one structure being experimental. Nowadays, most client authorities employ very few engineers and even they are employed primarily to administer competitive contracts rather than to take technical decisions. Each structure must be the cheapest possible; if the government official rejects a tender that conforms to the specification in favour of a higher priced tender he can be personally liable for the cost difference and, in some countries, may have committed a criminal act. Designers are free to propose innovations, provided they are cheaper than the conventional alternative and the designer assumes all risk. It is little wonder that there is little innovation.

3.1 Cost of FRP

Table 1 gives costs being charged in the UK in 2004 [15] for various materials, in commercial quantities, based on the ultimate strength of the element concerned. Anchorage costs are not included, nor are labour costs or profits. These figures should be taken as approximate in all cases.

Table 1 Cost of materials per unit force.

Material	Strength (MPa)	Cost (£/kN/m)	Cost ratio	Notes
Prestressing steel	1700	0.002	1	7-wire strand on coil
Reinforcing steel	460	0.006	3	Includes bending
GFRP	580	0.013	6.5	Excludes bending
Aramid fibre	2600	0.009	4.5	Fibre only
Aramid rope	2000	0.025	12.5	As a rope
AFRP	2000	0.025	12.5	As a pultrusion
CFRP	2000	0.025	12.5	As a pultrusion

(based on £1 = US$1.77 = €1.50, 2004 prices)

Steel costs have recently risen, supposedly because of significantly increased demand in the burgeoning Chinese economy. That should have improved the market for FRP, but it is clear from this table that the costs of new materials are higher in comparison to the costs of steel than they were 20 years ago and they are significantly higher than they were expected to be. It is widely believed that the aramid and carbon fibre manufacturers have decided to concentrate on the small-volume, high-price, high-technology markets such as aerospace, rather than go for the high-volume, low-price, basic-technology civil engineering market.

4 FRP CONCRETE MARKET

4.1 Internal reinforcement for flexure

GFRP will always suffer from its low stiffness and it is very unlikely to find a serious market where the deflections of the structure matter. Structures under any significant load would crack, so to keep the deflections to acceptable limits the structure would have to be much deeper than normal, or the structure over-designed to an unreasonable extent. There are, however, applications, where corrosion risk is very high but deflections would not be a problem, such as retaining structures or fenders in the splash zone. Structures that have to resist occasional loads, such as balustrades, or wind loads (such as sign gantries) might also be suitable, but it is doubtful if the extensive cracking that will occur in GFRP-reinforced concrete will make it robust enough for structures carrying heavy wheel loads, such as bridge decks.

It is also unlikely that AFRP or CFRP will be economic for flexure. At the strains at which concrete cracks the AFRP or CFRP will not be carrying significant load. The client is therefore paying for strength which is not being utilized.

To demonstrate these effects, a method was developed to study the initial cost of a concrete bridge structure with steel or FRP; it can identify optimum design solutions for reinforced and prestressed concrete bridge structures [16]. A detailed description is given in [17]. Design constraints are identified and the results plotted on a section depth vs. reinforcement area diagram which gives a zone of feasible solutions (Fig. 1). The cost appears as a line on the diagram and the optimum solution can be found numerically 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}).

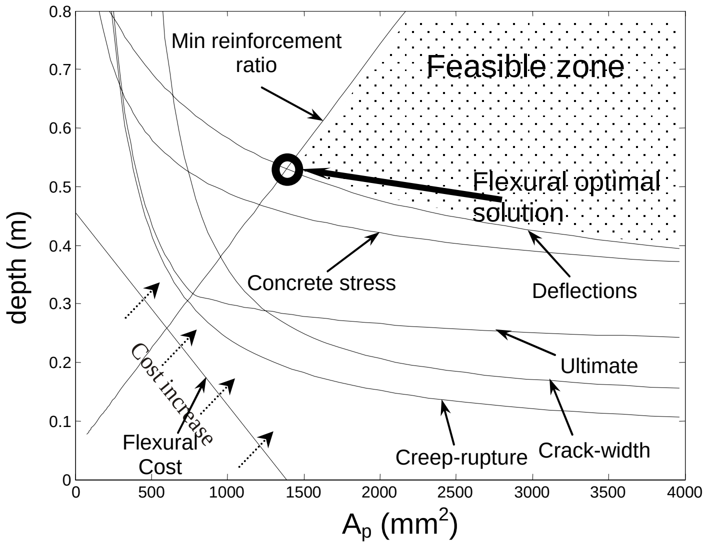

Fig. 1 Typical design chart for reinforced FRP beam.

Furthermore, 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.

4.2 Internal prestressing for flexure

Beams pretensioned with steel tendons do not have problems with corrosion, provided there is adequate cover to the concrete and the beams are made under factory conditions to sensible designs. Although it is possible to replace steel with FRP tendons, probably partially-bonded to achieve good moment-curvature responses [18], there appears to be little economic justification for so doing. The study discussed in Section 4.1 above also concluded that, 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 since tendons snap more easily; completely unbonded beams gave the cheapest solutions.
- The FRP/steel first-cost ratios increased for longer spans.

4.3 Internal reinforcement for shear

The situation regarding internal shear reinforcement is more complex. Shear reinforcement is always closer to the surface of concrete than the flexural reinforcement and is much harder to fix in the correct location. Unless every link is provided with spacers, some will have inadequate cover, so these are the elements that corrode first. Logically, therefore, there might be a stronger case for replacing the shear reinforcement than the flexural reinforcement.

However, theories for the mechanics of shear transfer with elastic reinforcement are inadequate [19]. Although beams can be analysed with elastic reinforcement the behaviour remains complex, so most formulae rely on analogues with the steel codes [20]. Because of the lack of ductility, it is assumed that limiting the strain in the FRP to the strain that the steel would have at yield allows plastic truss models to be used. The logic is false but it introduces a large factor of safety that appears to work! So the cost that matters is the cost per unit stiffness, which is typically three times worse for FRPs than the figures for strength.

The stiffer fibres, like carbon, might be expected to have an advantage in this situation and if they have to be used it seems most logical to consider CFRP shear links. But none of the existing techniques make full use of the properties of the material that is being provided so expensively. By limiting the fibre to the yield strain of steel, 90% of the fibre strength is not being utilized. It is difficult to fabricate FRPs as links and the conversion efficiency between yarn and composite properties is badly affected by the bend. This is a field in which a novel form of fabrication, perhaps involving a process such as knitting technology [21], could have clear financial advantages. If the concrete can be made to work harder and the fabrication costs of the shear reinforcement could be reduced, it may be possible to see an economic future for advanced composites in concrete structures. But bent pultrusions will never be economic.

4.4 Internal confinement for compression

If the tensile elements are provided by FRP then designs should no longer be underreinforced. These elements are brittle, so if energy is to be absorbed this must occur in the compression zone. The use of spirals of FRP to contain the concrete can be remarkably cost effective. By confining the concrete in the compression zone with spirals of aramid fibre the strength can be increased marginally, but the strain capacity can be increased three- or four-fold. Remarkably little confining stress is needed for this; 2 MPa is sufficient. The amount of fibre needed to achieve this is small; if about one-sixth of the volume of the material used to provide tension capacity is used to provide spiral confinement, the strain (and hence curvature) capacity is tripled [22]. FRPs should be used for what they are good at, rather than trying to make them behave like very expensive and rather inadequate versions of the steel reinforcement that they are replacing.

4.5 Internal reinforcement for local effects

A field that has largely been ignored for FRPs relates to the reinforcement that is required in the vicinity of concentrated loads such as anchorage zones for prestressing tendons, where the concrete is almost certainly cracked and the reinforcement is close to the surface [23]. The lack of stiffness is not a problem because the elements are short. Half-joints, corbels and beam seats have similar problems.

4.6 External prestressing tendons

External prestressing tendons were one of the first applications envisaged for advanced composites [24] and remain the major application for which FRPs are most suitable. The ability to inspect, and if necessary replace, the prestressing tendons is attractive to specifying authorities. This has been recognized even with steel tendons where it is known (although perhaps not acknowledged) that the exposure will shorten the lifetime of the tendon. Steel can be protected by wrapping the tendons in grease-filled plastic sheaths, but this typically triples the cost and removes the ability to inspect the tendons in detail. By prestretching the aramid or carbon fibre tendons the additional strain capacity can be utilized. One disadvantage is the requirement for anchorages, but systems are available for aramid ropes that can anchor the full strength of the rope [25], and systems that are almost as effective are available for CFRP [26]. The arguments that, twenty years ago, identified these tendons as the most logical application of advanced composite tendons remain valid today.

5 WHOLE-LIFE COSTING

There is widespread criticism of construction standards in the past. Many structures are being repaired because of deficiencies, often related to durability. Engineers are being criticized for not paying sufficient attention to durability; an apparently small amount of money and an ability to see into the future would supposedly have led to more sensible decisions. But are better decisions being made now? If structures are designed today and it takes 35 years before they need attention, who cares? Who can foresee what they will be doing in 2042? The chances are that the senior engineers (who presumably take the decisions) will be dead, and their children will have retired; the problem will fall to their grandchildren to deal with. Is it possible to make sensible decisions on that time-scale? An often-quoted, but apocryphal, statistic is that London office buildings are designed with a structural lifetime of 20 years; the cladding is expected to last 10 years; the internal partitions 5 years, and the internal wiring 2 years. If refurbishment is not undertaken on this time-scale the building is seen as old-fashioned and is unlettable at economic rents. What price durability for this type of structure?

5.1 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 [27,28]. 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 (Figure 2).

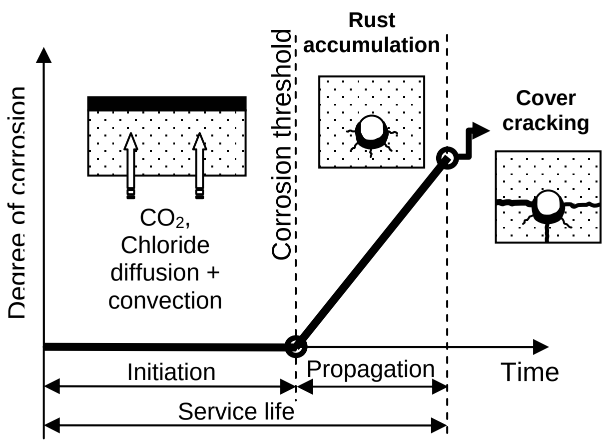

Fig. 2 Progress of corrosion.

Corrosion occurs 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 need to be taken into account, and the environmental conditions introduced through boundary conditions. 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.

5.2 Discount rates

It is commonly stated that structural options are determined on the basis of whole-life costing [29], but the impression is often gained that only lip-service is being paid to these ideas. In theory, designers can choose between the cheapest first-cost option or the cheapest whole-life-cost option. To make that choice, an assumption has to be made about the discount rate to be used. If money is invested now, it will accrue value; that added value can be used to pay for future maintenance costs. If a high rate of return is used, not much money has to be invested to pay for future maintenance, so the present value of future maintenance costs is low and it is not worth spending much money now to prevent future maintenance. Conversely, if discount rates are low, a lot of money has to be invested to pay for maintenance, and a durable structure is a valuable asset. In the UK, at present, one can earn about 4% return on investment income, with inflation running at about 2.5% per annum. So the true discount rate that should be used is about 1.5%, and calculations performed this way can compare building costs and future maintenance costs at 2007 prices. Historically, discount rates have been set as high as 6%; $1.06^{35} = 7.7$, whereas $1.015^{35} = 1.7$. The present value of a repair to be undertaken in 35 years time is more than 4 times higher if a low discount rate is used than if a high discount rate were used. The choice of the discount rate to be used in these calculations is of crucial importance to the decisions that are made; the temptation is to use a high rate which conveniently means that the best whole-life solution just happens to be the best first-cost solution as well. Future maintenance remains the problem for future generations and new structures are built cheaply.

5.3 Delay costs

The question also arises as to what future costs are to be included. In the case of a building the situation is clear; the building is owned and occupied by the same person – even if space in the building is let to a tenant, the tenant is free to move elsewhere so the owner is directly affected by the tenant's costs. But in the case of a highway structure the situation is less clear. In the case of a public highway, to which everyone has free access, the costs borne by the bridge *owner* and the costs borne by the bridge *user* are separated. The bridge owner, usually ultimately the government, has to bear the costs of the actual work carried out on the bridge, but traffic delay costs are borne by the motorists themselves (or by the clients of the haulage companies whose goods are being carried). If there were many alternative routes that could be followed then traffic would simply divert elsewhere, but in practice most highways run at or near capacity for much of the time. So diversion is not possible and traffic is disrupted, causing delays that can be assigned a cost. A study showed that:

- User-delay costs represent the largest portion of the life cycle costs. They are often so high that even if they occur a long time after 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.

Traffic delay costs can be huge and totally overwhelm all other costs. Whereas the cost of using FRPs may be measured in tens of thousands of pounds (or dollars, or euros), and the cost of repairs may be measured in hundreds of thousands, the cost of traffic delay costs are measured in hundreds of thousands *per day*. So the economic viability of a design depends crucially on whose costs are taken into account. If society is building a structure for itself, then the costs incurred by motorists wasting time are a true cost to the national economy; if a company is building an asset, and the consequential costs incurred by others do not impinge on the company itself, then there is no need to take delay costs into account.

5.4 Logical applications of FRP

The effect of these decisions on the FRP market can be summarized simply:-

- *If* the structure under construction is externally prestressed, *and*
- *If* realistic whole-life costing is being adopted, *and*
- *If* a realistic discount rate is being used, *and*
- *If* traffic delay costs are being included

then the owner *must* use FRPs for an economic structure. If any of these conditions are not satisfied, FRPs will never be an economic solution unless special factors apply, such as exposure to extremely corrosive environments in an application where large sections or large deflections are acceptable.

6 CASE STUDY – US BRIDGE STOCK CONDITION

In 2002 a study of the condition of all bridges in the USA was published [30]. The sample consists of 257,235 bridges made from concrete, of which 137,961 are reinforced and 114,795 prestressed with steel bars. In the last 40 years about 4500 bridges have been built each year. 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). The data allows the determination of the cumulative probability that a bridge is in a certain condition (Figure 3). A bridge will need to be repaired when it reaches State-4 (poor condition: advanced section loss, deterioration, spalling or scour).

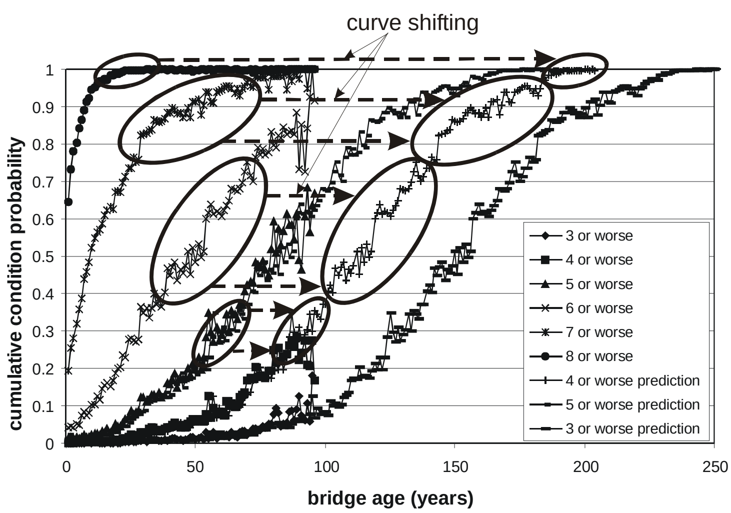

Fig. 3 Existing and future condition prediction for US bridges.

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 [31]. 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. 3). Polynomials can then be fitted to the predicted data; the slope of the cumulative probability is the bridge condition probability.

There is little that can be done about the existing bridges, but decisions can be taken about new construction. What is the consequence of building new bridges with steel reinforcement (which will need repair on the timescales predicted by Fig. 3), as compared with building similar structures with FRP, which will need much less repair? Making reasonable assumptions about cost of building, cost of repair, and traffic delay costs, Figure 4 shows the avoidable costs to the US economy of constructing new bridges with materials that do not corrode. This is the money that should be invested now to pay for the future repair costs of bridges that will be built with steel reinforcement. 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 are small – first 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. At the current rate of exchange, this represents up to about $US 3 billion for *every year* in the future.

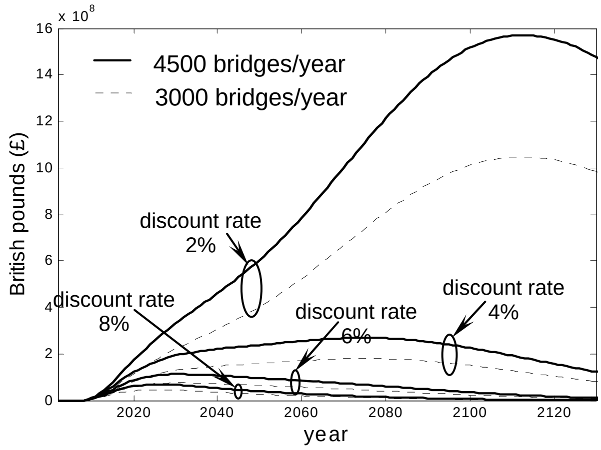

Fig 4 Net Present Value of not building new bridges in the US with FRP.

7 CONCLUSIONS

The effective design of structures using advanced composites cannot be divorced from the economics of the world in which the structures are being built. The market for FRPs has not taken off as had been expected, largely because the FRP market has concentrated on trying to make FRPs look like the steel elements they are replacing. FRPs do not make effective reinforcing bars except in certain niche applications and they are very ineffective in shear. External prestressing with aramid ropes and carbon fibre cables remains a sensible application, as is adding confinement to the compression zone of concrete beams. A useful contribution to the shear capacity of beams will only be provided when a system is developed that allows the fibre to confine the concrete in such a way that the concrete itself becomes more effective.

Structures will rarely be economic unless sensible decisions are made using whole-life costing and taking into account all the costs of repair, including delay costs and loss-of-use, as well as using a realistic discount rate to determine the present value of durability.

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