Designing for ductility

A. R. Kemp, Professor of Civil Engineering at University of Witwatersrand, South Africa and Corresponding Member of the Institution's Research Panel writes:

Over 60 years ago the pioneering work of the Steel Structures Research Committee established that design based on linear-elastic analysis is unlikely to provide an accurate representation of the actual behaviour of structures. Recognition of ultimate and serviceability conditions as the basis of limit-states design in the 1960s created the expectation that the ultimate limit state would be modelled in terms of inelastic behaviour. Design codes have increasingly provided opportunities for ultimate resistances to be calculated using plastic stress blocks, but most designers continue to assess ultimate load and action effects using elastic analysis. Increasing attention to ductility provides a basis for evaluating inelastic behaviour in structures.

Ductility is a desirable characteristic in structures because it represents the ability to deform significantly beyond the elastic limit while maintaining an ultimate resistance at or above the design value. Benefits to be derived by designers from ductile, indeterminate structures have been appreciated for many years, including:

- Greater structural efficiency through continuity of members in terms of both flexural strength and stiffness.
- Provision of alternative load paths when the actual behaviour does not accord with the design assumptions, or these assumptions are in error.
- Improved safety when loads of an unexpected or rare nature occur, such as explosions or earthquakes.
- Improved ductility is often achievable at little extra cost.

Research[1,2,3] at the University of the Witwatersrand, Johannesburg, has been aimed at finding how this inelastic behaviour may be modelled simply to assess the required and available inelastic rotations at notional plastic hinges. It has illustrated how easily limit states of ductility may be defined and applied to complex inelastic situations such as interactive local and lateral buckling of steel sections and semi-rigid connections in continuous steel and composite beams.

Required notional hinge rotation

In an indeterminate structure loaded into the inelastic range, the required ductility is assessed in terms of rotation at each notional plastic hinge, where the moment exceeds the elastic limit[1]. Consider as a simple illustration the fixed-ended beam ABC in Figure 1. Assuming that the elastic-plastic resistance moment, M_r, is equal in both sagging and hogging bending, the first inelastic hinges develop at A and C. Under increasing load the moment at B reaches M_r and the magnitude of the required inelastic rotation, θ_r, at A or C is obtained by solving the following rotation compatibility equation, based on flexibility considerations:

Rotation at end A (or C) =
$\theta' + \theta_e + \theta_r = 0$

in which $\theta' = wL^3/24EI$ is the simply-supported end-rotation of the member due to the member loads (i.e. with end moments equal to zero) and
$\theta_e = -M_r L/2EI$ is the elastic end rotation due to the end moments M_r at A and C obtained from standard flexibility equations. At the formation of this collapse mechanism in Figure 1(c),
$w = 16M_r/L^2$ and the required inelastic rotation
$\theta_r = -\theta' - \theta_e = wL^3/96EI$.
This illustrates the more general statement that at a notional hinge in a continuous beam, θ_r, is equal to the proportion of the elastic moment which is redistributed from an internal support to the adjacent midspan region (in this case 0.25) multiplied by the simply supported end rotation due to the applied loads, θ'.

In inelastic computer analysis of structures it is convenient to consider a flexibility approach with member end-moments and maximum internal moments as unknowns[2], rather than joint rotations. The rotation compatibility equations, corresponding to the unknown end-moments, equate the end rotations in each pair of members meeting at a joint. As in the example of the fixed-ended beam, these compatibility equations directly identify the required inelastic rotation at each notional hinge, while allowing for any member loading, semi-rigid end connections and other inelastic behaviour. Many benefits exist in this approach.

Available notional hinge rotation

The available inelastic rotation, θ_a, of a notional plastic hinge, before the resistance moment falls below the design value, is the sum of all the associated components of inelastic curvature and rotation which are not considered in a normal elastic analysis[1]. It is made up of components due to yielding of the steel, cracking or crushing of the concrete and total rotation of the end connection.

As shown in Figure 2, inelastic regions adjacent to the ends of a member or section of maximum internal moment within a member, may be modelled using:

- Bi-linear moment-curvature relationships[3] (for steel or composite members, including provision for strain-hardening)
- Bi-linear moment-rotation relationships[1] (for end connections)
- Elastic-perfectly-plastic, moment–curvature and moment-rotation relationships (for reinforced concrete members with appropriate stirrups to constrain the concrete in compression in the hinge region).

The designer may choose[1] to provide the required ductility through flexibility of the end connection or yielding/inelastic behaviour of the adjacent steel, composite or concrete members, or both, depending on which provides the lesser strength and greater ductility. A large body of research has demonstrated the considerable ductility that is available in end connections with thin flush or partial-height end plates[1]. In such cases the moment resistance of the connection may be significantly less than the adjacent beam.

Bi-linear moment-curvature relationships have been shown to model accurately the flexural behaviour of

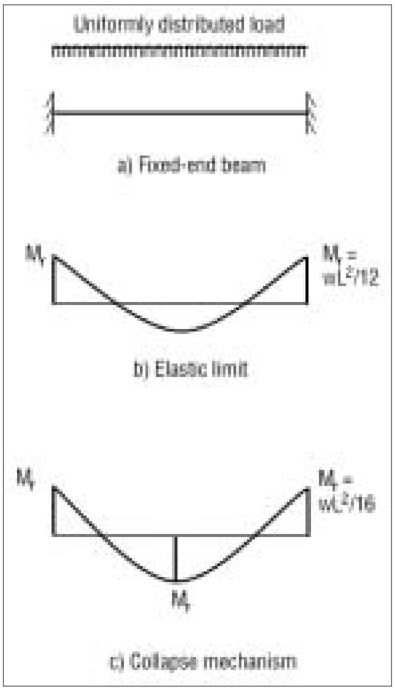

Fig1. Collapse mechanism for fixed-end beam

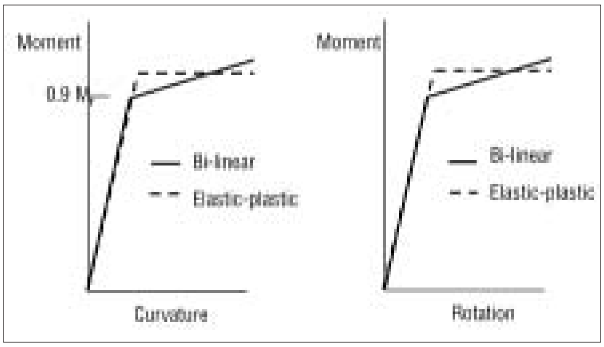

Fig 2. Moment-curvature and moment-rotation relationships

compact steel I-sections[3]. In pure flexure the bi-linear transition point is at about 90% of the plastic moment resistance and the ratio of elastic flexural rigidity to strain hardening flexural rigidity is between 75 and 100. If adequate restraint is provided to prevent lateral buckling, the inelastic region of this bi-linear relationship may be extended to about 108% of the plastic-moment resistance[3]. The bi-linear relationship is adjusted for coincident axial force.

Over 50 double-cantilever steel and composite beams have been tested[1], representing the negative moment region adjacent to internal supports. The available inelastic rotation before strain weakening, due to interactive local and lateral buckling, is accurately modelled in terms of an inelastic strain limit at the centre of the compression flange[3] or a curvature limit, rather than a stress limit.

Limit states criterion for ductility

A simple limit-states criterion for quantifying the ductility of flexural members is obtained by specifying that the available inelastic rotation at each notional plastic hinge, θ_a, before the moment capacity falls below the design value, should exceed the required inelastic rotation, θ_r, at the same notional hinge, as follows:

$$\theta_a / \gamma_a \geq \gamma_r \theta_r$$

in which γ_a and γ_r are appropriate partial material and load factors for ductility to allow for uncertainties in the assessment of θ_a and θ_r. The two sides of this limit-states criterion comprise the resistance (available hinge rotation) and the load effect (required hinge rotation) and have been discussed in the two preceding sections of this article. The required rotation of the notional hinge, θ_r, refers only to the inelastic rotation implied by the process of moment redistribution or plastic analysis. It is a pure load effect that is independent of inelastic material properties such as steel yielding or concrete cracking.

On the other hand the available rotation of the notional hinge, θ_a, is the sum of the effects of inelastic behaviour of the component materials in positive or negative moment regions provided by yielding of the steel, crushing or cracking of the concrete, total rotation of the end connection and any other properties not considered in an elastic analysis. This is consistent with the principle of incorporating material properties in the resistance side of a limit-states equation.

This simple criterion for ductility may be applied to a wide range of modes of failure of compact members, provided tests have been conducted to quantify the available rotation before the moment resistance falls below the design value.

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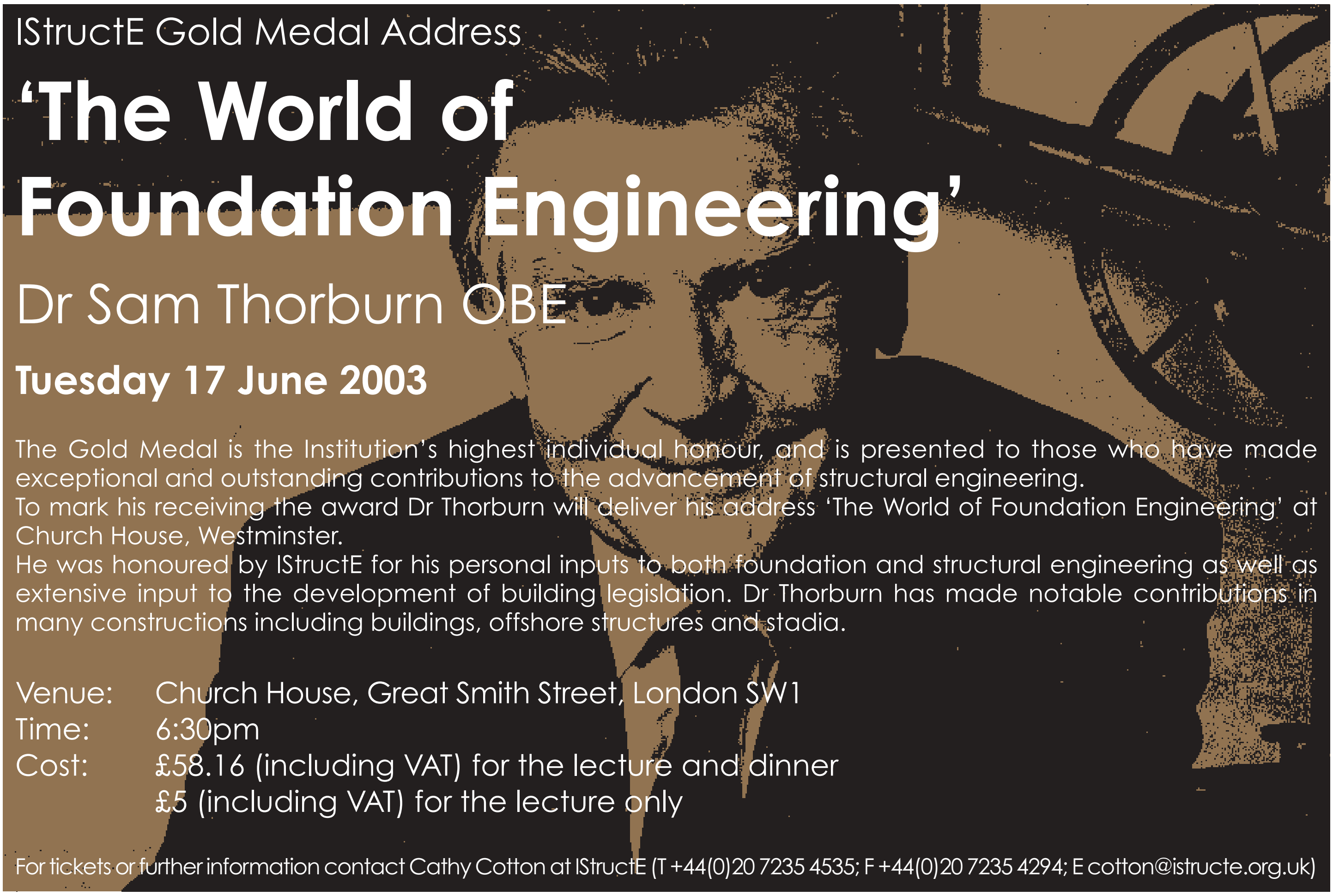


Technology transfer from offshore to onshore structures

The design of topside structures to resist hydrocarbon fires and explosions necessitates consideration of a range of issues not normally addressed by the designer. Dr Bassam Burgan, Deputy Director, SCI and Dr Fadi Hamdan, Manager, Offshore Engineering, SCI look at some of the key issues

The Steel Construction Institute has been involved in a variety of initiatives with the offshore industry to develop guidance for the protection of offshore topside structures against fires and explosions[1-10]. The design of topside structures to resist hydrocarbon fires and explosions necessitates consideration of a range of issues not normally addressed by the designer.

This article gives an overview of some of the key issues that have been studied in several sponsored R&D projects and by the Fire and Blast Information Group (FABIG)*. While these developments have been carried out for application to offshore structures, they are also relevant to onshore structures. Technology that can be transferred most readily for use in the design of onshore structures includes:

- elevated temperature material property design data for use in fire engineering;
- high strain rate material property design data for use in explosion resistant design;
- analysis techniques for explosion design.

Elevated temperature material property data

Whether advanced or codified methods are used to assess the performance of structures in fire, accurate elevated temperature material property data is central to the calculations.

The type of material data required will depend on the assessment method employed. Codified methods require elevated temperature yield strength and Young's modulus values, whereas advanced methods require full stress-strain curves at elevated temperature.

Fig 1 shows the variation in the strength reduction factor ($k_{y\theta}$) with temperature corresponding to three different strain levels. The rounded shape of the stress-strain curve of steel at elevated temperature (Fig 2) gives rise to the variation in strength seen in Fig 1 at different strain levels; the higher the value of strain, the greater the elevated temperature strength value will be. In codified design, it is customary to select strength values at strain levels appropriate to the type of loading acting on the member (axial, bending or tensile) and the type of cross-section (e.g. whether a section might be prone to local buckling). Often, basic strength values are given which correspond to 2% strain, and modification factors are applied to the resistance equations to account for cases where failure may occur at a lower strain level.

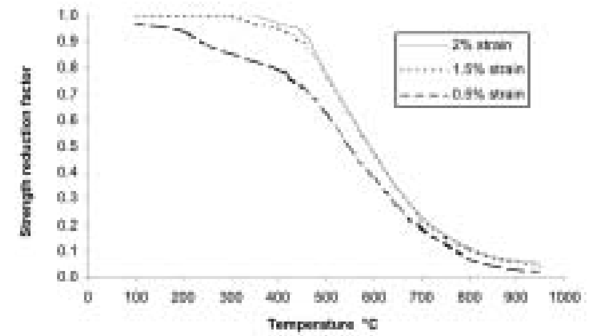
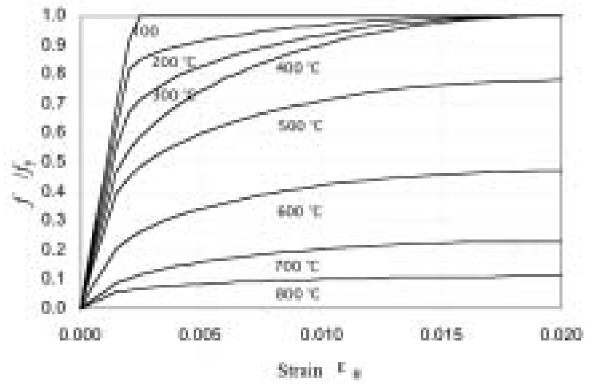


Fig 1. Relationship between strength factor (at different strain levels) and temperature

It is very important to recognise that the strength and Young's modulus reduction factors are steel grade dependent. FABIG undertook research which studied the behaviour of a range of offshore topside steels (S355M, S420M, S355EMZ, S450EMZ) and stainless steels (304, 316L, 2304 and 2205) at elevated temperature[8] and has also collated similar existing data for more common structural and process steels.

Data is presented in the two forms relevant to codified and advanced design methods. For the former, tables of yield strength (corresponding to 2% strain) and Young's modulus as a function of temperature are given. For the latter, analytical models enables the full stress-strain curve to be derived. The nature of the material behaviour of stainless steel (e.g. lack of a distinct yield point at room temperature and considerable strain hardening characteristics) meant that additional parameters were required to describe the full stress strain behaviour[8,11,12]. These include the 0.2% proof stress, the ultimate strength and the tangent modulus.

Fig 2. Variation of stress-strain relationship with temperature for Grade S355 Steel



Elevated temperature properties are required for the design and analysis of both onshore and offshore structures, such as fire walls, piping and equipment, against fires. Many of the data derived for offshore structures correspond to material that may also be used for onshore structures.

High strain rate material property data

As with fire engineering, the type of material property data required for response analysis depends on the method of analysis. The key feature is the effect of strain rate on material behaviour. When steel is dynamically loaded, the rate of straining affects both the yield and ultimate strength of the material. The higher the rate of strain, the greater the enhancement of both properties. Fig 3 shows stress-strain curves for grade 43 steel under different rates of straining.

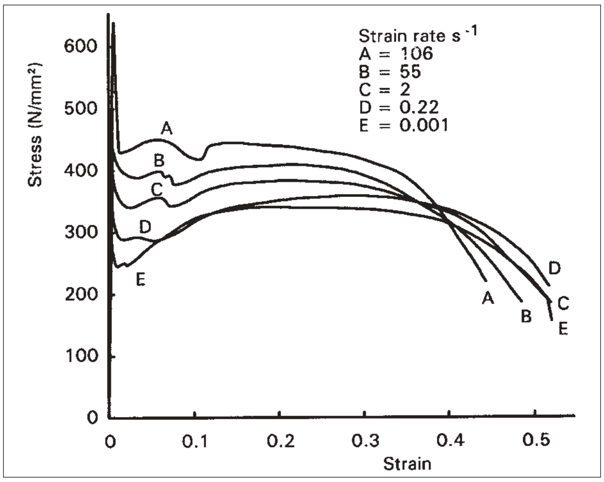
Recently work was carried out to assimilate existing material models for steels used on topside structures and further work was undertaken to generate additional data where such information was found to be lacking. The work was documented in a FABIG Technical Note[8] which gives high strain rate material data for offshore topsides steels. These included steel grades 43, 50D, 355EMZ and 450EMZ and stainless steel grades 316L, SAF2304 and 2205.

High strain rate material data are required for the design and analysis of blast walls, piping and equipment on onshore and offshore facilities. Much of the data that has been derived for offshore structures corresponds to material that may also be used for onshore structures.

Analysis techniques for explosion design

The performance of the structure can be assessed either by advanced analytical tools (e.g. non-linear finite element analysis) or using simplified analysis tools (e.g. Single Degree of Freedom, SDOF) methods.

The former is very powerful; potentially it enables a detailed assessment of the performance of a structure, including yielding, all buckling modes, fracture and failure of joints. However, the quality of the analysis is highly dependent on the



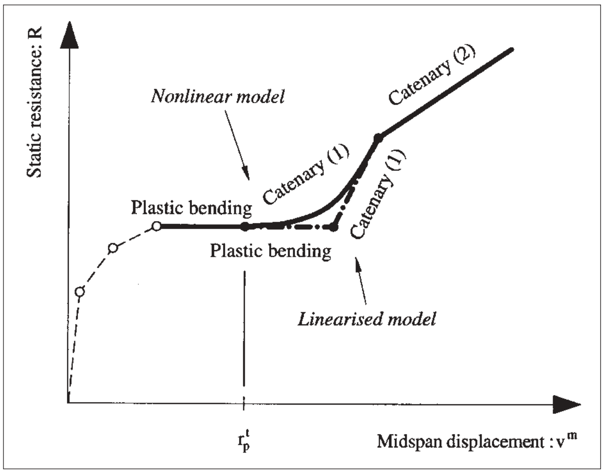


Fig. 3 (left) Effects of strain rate on the behaviour of grade 43 steel

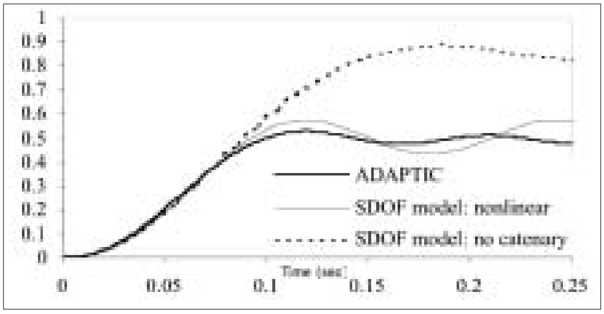
Fig 4. (below left) Stages of the plastic bending and catenary response

program (and the extent of its validation) and the quality of the analysis/competence of the analyst. The sophistication of these methods also means that analyses take longer to set up and run than for simplified methods.

SDOF methods are very effective where the single degree of freedom idealisation is valid (e.g. for line members or 2-dimensional structures that have a much more pronounced stiffness in one direction than in the other as with many blast walls). The SDOF method due to Biggs[13] has been recommended in the *Interim guidance notes for the design and protection of topside structures*[2] for the analysis of beams subjected to explosion loading. The method has been shown to give accurate response predictions within its limits of applicability. However, it is limited in the following ways:

- it does not incorporate the effects of support stiffness;
- it does not account for different moment capacities at the two supports;
- it ignores the catenary effect, which has a significant influence on the large displacement member response, in the presence of axial restraints;
- it ignores the influence of material rate-insensitivity and strain-hardening.

Fig 5. (below) Comparison between FE analysis, the new SDOF method and Biggs' method



Because of the wide use of Biggs method in response assessment, work was undertaken to extend the method such that the first three of the above limitations are overcome[9]. The theoretical developments of the work were carried out by Izzuddin[14]. The method describes the development of the behaviour from elastic, through elastic-plastic (as plastic hinges develop), perfectly plastic (plastic mechanism) and finally the development of catenary forces as displacements increase (Fig 4).

The factors necessary for the application of the above method for all the possible combinations of boundary conditions and order of plastic hinge formation are tabulated. Fig 5 shows a comparison between the new SDOF method, ADAPTIC (a non-linear FE program) and Biggs method for the case of a simply supported beam with axial end restraints giving rise to catenary forces with increasing deformations. It also documents the accuracy of the new method and the importance of accounting for catenary action in such problems.

The method described in this section is applicable to the simplified analysis of the response of structures subjected to dynamic loading, and is applicable to both onshore and offshore structures.

Conclusion

Over the past 10 years FABIG has acted as a focus for the development and updating of explosion and fire design guidance, and plans for future guidance in the form of FABIG Technical Notes are in place. Many of these guidance notes and publications are directly applicable to the design of onshore structures in general, and onshore industrial facilities in particular, against fires and explosions.

FABIG, the Fire and Blast Information Group, is a transnational, cross-industry membership organisation committed to promoting the protection of life, property and the environment through the development and sharing of expert knowledge on hydrocarbon fires and explosions.

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Fig 6.
Unconfined jet fire test impacting on a pipe target

The effect of surface profile on flexural strengthening of rc sections with frp

Andrew Porter writes about his undergraduate project at Bath University which won Joint First prize in the Model Analysis Award 2002. This experimental research questions the limitations on surface straightness given in current design guidelines

The general deterioration of structures combined with increased loading or change of use has resulted in the demand for an effective and economical strengthening solution.

Protection of the environment is becoming a major consideration in all construction projects and sustainability is a key issue in all its various forms from embodied energy of materials to energy associated with transport, construction and operation.

Strengthening of existing structures avoids the need to demolish and replace, enabling the design life of the current structure to be increased. The use of fibre-reinforced polymers (frp) is becoming a widely accepted solution for strengthening reinforced concrete structures. Good durability characteristics, low self-weight and ease of installation make frp an attractive option in most strengthening schemes[1].

Flexural strengthening of rc structures with externally bonded frp is just one example of the use of composites in strengthening solutions. Shear strengthening, column wrapping and the use of frp as a substitute for internal steel reinforcement are all examples of its application.

The use of frp in strengthening schemes is an area of active research. Due to the elastic nature of the composite, many of the assumptions valid for traditionally rc sections are not applicable for an frp strengthened structure. A clear understanding of the assumptions behind each step of the design process, coupled with a full understanding of the failure mechanisms associated with frp strengthened structures, is essential in developing an efficient strengthening scheme[1].

Failure of a concrete section strengthened in flexure with externally bonded FRP can be placed in one of two categories:

- failure where composite action is maintained
- failure where composite action is lost.

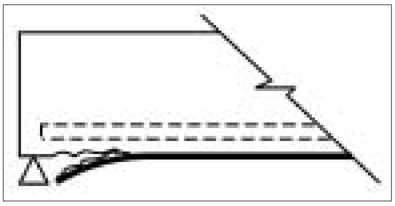

Fig 1. Failure plane in concrete at the bottom of section

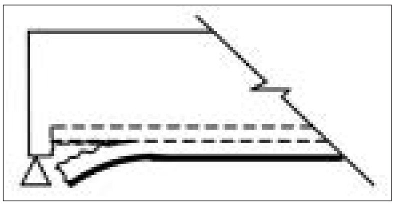

Fig 2. Failure plane between steel reinforcement and concrete

Fracture of the frp plate or failure by concrete crushing (Mode 1) can generally be analysed with standard methods of rc design. Failures involving loss of composite action, termed 'peeling' or 'delamination' failures, are more unpredictable but will generally be the governing mode of failure. Peeling refers to the unstable propagation of a longitudinal crack or failure surface beneath the frp. The failure plane will typically occur in the surface of the concrete (Fig 1) or at the level of the steel reinforcement (Fig 2).

The four principle initiators of a peeling failure are:

- shear cracks
- flexural cracks
- surface profile
- inadequate anchorage.

Extensive application examples and research exist on flexural strengthening of rc sections using frp. However, while curvature on the soffit of structural elements is common to almost all strengthening applications, it remains an area of limited study. Curvature can refer to the profile of the entire element or to localised curvature due to an uneven surface. This article offers a summary of a programme of testing aimed at investigating the significance of curvature on the flexural strengthening capability of frp.

Motivation

The quality of installation has a significant impact on the effectiveness of the strengthening scheme and the overall performance of the strengthened structure relies on a high-quality bond between composite and concrete. The surface profile of the beam or element being strengthened, whether it is local unevenness or curvature of the entire element, affects the bond behaviour and hence the degree of strengthening achieved. The presence of curvature results in the development of normal tensile stress in addition to shear stress, as the plate attempts to straighten under load. Analysis of the elastic stresses at the interface shows that the introduction of curvature can lead to an increase in both the principle tensile stress and the maximum shear stress. A strengthened beam or element displaying concavity would therefore be expected to exhibit a lower capacity than a flat-soffit element where the governing mode of failure involves delamination of the frp.

In order to avoid failure by delamination due to a concave surface profile, limitations are typically placed on the allowable curvature of the concrete surface. Different types of composite material have varied sensitivity to local deviations in the surface profile and this is reflected in the design guidelines. Rigid frp plate can span small deviations using the adhesive as filler whereas sheets and fabrics are more flexible and follow the surface of the concrete.

As a result, the implications of an uneven surface are more serious for sheets and fabrics than for plate. A widely accepted limit for unevenness[2, 3, 4] is 5mm over a 1m (5mm/m) length for rigid plates prior to application of the frp. Based on the frp plate specified for this study, 5mm/m is an acceptable curvature and should result in no loss in the achievable degree of strengthening. However, since current design guidelines have been predominantly validated using laboratory cast specimens manufactured with a high degree of straightness, the effect of such a curvature needs to be better understood.

Fig 3. (above) Test set-up

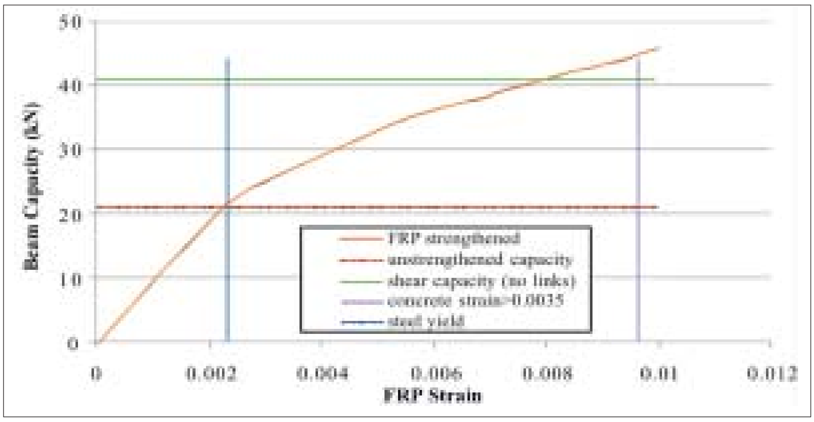

Fig 4. (top right) Theoretical specimen behaviour

Testing programme

The following testing programme was conducted in order to explore the suitability of current design guidelines. The tests consisted of three beams with varying curvature imposed on the soffit. As described previously, limitations on curvature are commonly expressed in terms of a maximum deviation from straight over a length of 1m. Assuming the surface profile to be circular, a maximum deviation, Y, can be obtained based on any length, X, and an associated radius of curvature, R. The curvatures used were zero (beam 1), 5mm/m (beam 2) and 15mm/m (beam 3). By using a curvature of 5mm/m, evaluation of the results with respect to current design philosophy was possible. A greater loss of strengthened capacity was expected by using a more severe curvature (15mm/m).

The following failure mechanisms were considered when designing the specimens:

Composite action between frp and concrete:

- concrete crushing
- frp fracture

Loss of composite action:

- end peeling/anchorage failure
- delamination from within the span (flexural cracks)
- shear cracking induced delamination
- concave profile induced delamination

The experimental work aimed to investigate the effect of a concave surface profile on the capacity of an frp-strengthened beam. The risk of failure due to some other mechanism was therefore minimised.

The optimum solution was found by considering the unstrengthened shear capacity, the tensile flexural capacity, frp peeling and concrete crushing. By plotting the capacity of the section based on these various parameters, an appropriate section could be selected. Detailed checks on the chosen section included end anchorage (based on the equations derived by Rostasy and Neubauer[5]) and longitudinal shear. Fig. 4 shows the theoretical behaviour of the chosen scheme, indicating that failure would be unlikely to result from concrete crushing, frp fracture or shear cracking induced delamination.

Results

The test results are summarised in Table 1. The 'load sustained' refers to the load carried by the beam following delamination of the frp plate.

Beams 1 and 2 failed by delamination of the plate. Inspection of the soffit showed no significant step, suggesting that peeling from a shear crack did not occur.

Beam 3 also failed by delamination of the plate, but a large crack also formed running along the reinforcement and up to the load point (Fig 5). It was unclear whether the crack formed before or after composite action was lost, although inspection of the soffit showed no shear step, suggesting delamination did not initiate from the crack.

Strain measurements were taken at mid-span and indicated that, for beams 2 and 3, delamination of the plate occurred at a lower value of strain than for beam 1. Undoubtedly, such strain measurements can be sensitive to the position of the gauge in relation to cracks in the concrete.

The motivation for the experimental study was to gain further understanding of the effect of a concave surface profile on the strengthening capacity. Direct comparison of the results is made difficult due to the difference in cube strengths for the three specimens. In order to make a valid comparison, the results should be corrected to account for this variation. It is reasonable to assume that the increase in capacity due to the frp is a function of the tensile capacity of the concrete, f_{ct}.

The increase in capacity provided by the frp can be estimated by considering the failure load and the load sustained by the beam immediately after delamination occurred. Plotting the increase in capacity divided by f_{ct}, against the curvature gives a reasonably linear relationship (see Fig 6). The strengthened capacity of the beam decreases as the curvature increases which confirms the original hypothesis.

The experimental program also aimed to provide results with which to assess current design philosophy and codes of practice. The limit of 5mm over a length of 1m is commonly specified as a limit to avoid loss in strength of the reinforced section. The results obtained from this work would suggest this to be unsafe.

Fig 5. Failure of specimen 3

Table 1: Results summary

Beam no.	Curvature	f_{cu} (MPa)	Failure load (kN)	Load sustained (kN)	Failure strain	Mid-span deflection (mm)
1	0	37.5	36.1	22.5	0.0060	26
2	5mm/m	32.5	31.7	20	0.0045	19
3	15mm/m	39.8	32.0	20	0.0044	18

Current design guidelines also advise that limitation of the strain in the frp plate will decrease the likelihood of failure by delamination of the plate. Limits are commonly in the range 0.006 - 0.008. Failure occurred when the strain in the frp at midspan was 0.0060, 0.0045 and 0.0044 for specimens 1, 2 and 3 respectively. Not withstanding the fact that these may not have been the maximum frp strains, it is reasonable to conclude that some modification of these limits may be required for curved elements.

Conclusions

- The presence of a concave surface profile on the soffit of a beam reduces the degree of flexural strengthening provided by an externally bonded FRP composite.
- The limitations on surface straightness given in current design guidelines are questionable based on the findings of this research. Experimental results suggest that, contrary to some design guidelines, 5mm/m curvature will result in a loss in the degree of strengthening provided by an externally bonded frp laminate.

Offered here is a summary of a full study into the effect surface profile has on the flexural strengthening capability of externally bonded frp.

- Further information: Andrew Porter: (email: portera@pbworld.com)

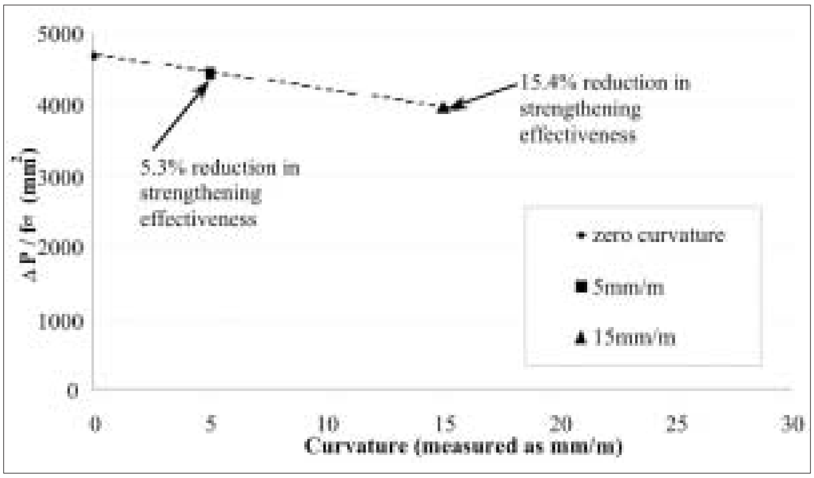

Fig 6. Curvature and associated reduction in effectiveness of strengthening

The Model Analysis Award is an annual competition for experimental projects carried out by final year undergraduates and first year postgraduates. The competition is organised by the Institution's Study Group on Model Analysis as a Design Tool.

- *For details of the 2003 competition, see Structural News page 9)*

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