KEYNOTE PAPERS

CICM 2015
First International Conference on
Advances in Civil Infrastructure and Construction Materials
MIST, Dhaka, Bangladesh, 14–15 December 2015

DESIGNING STEEL AND COMPOSITE BUILDINGS TO RESIST PROGRESSIVE COLLAPSE

David A. NETHERCOT

Department of Civil and Environmental Engineering, Imperial College London, UK
Email: d.nethercot@imperial.ac.uk

Abstract. *Based on the study of actual events, together with relevant material from adjacent areas, the main features of Progressive Collapse are identified and the requirements for a practical design approach stated. The essentials of the Imperial College Robustness Assessment Framework are described and the methodology illustrated with results for a few illustrative examples. The focus throughout is on reconciling an approach that is sufficiently rigorous that it captures all the essential features with a methodology that is similar to that employed for conventional design assessments.*

Keywords: Buildings, Composite Structures, Progressive Collapse, Robustness, Steel Structures, Structural Design.

1. INTRODUCTION

Few events in the history of the Construction Industry can match the collapse of the Twin Towers of the World Trade Centre [1] for drama. Following the aircraft impacts a pair of iconic structures collapsed, with the scenes being witnessed on their television screens by billions of people across the world. How could the integrity of such structures, designed by one of the world's foremost Structural Engineers, be destroyed by such a localised event?

To those with some understanding of the mechanics of progressive collapse the result was less surprising. Some 30 years previously a small gas explosion in a top floor apartment in a tower block in East London, which led to the collapse of one corner of that block, had given the Structural Engineering profession the name Ronan Point [2]. And there had been others: the Murrah Building in Oklahoma City in the United States, Skyline Towers also in the United States and, in 2013, Rana Plaza in Savar Bangladesh - Wikipedia lists several more. The common feature in all these collapses was a triggering incident that caused some local weakening leading to collapse of all or a substantial part of the complete structure.

But there have also been other, less well published, incidents in which a seemingly similar triggering event did not lead to a major collapse. A particularly interesting example was the1945 incident in which a U.S. Air Force plane became disorientated in low cloud over New York and hit the Empire State Building [3]. No significant consequential damage was reported, the structure was repaired and remains in use to this day.

What were the essential differences between the plane impact on the Empire State Building and that on the World Trade Centre? To what extent should such possibilities feature in the thinking of Structural Designers? If they are to be considered what processes should be used to conduct the necessary assessment?

This is the subject area that is variously referred to as: Progressive Collapse, Robustness and Disproportionate Collapse. Spurred on by the WTC collapses it has become a fertile area for research in recent years and a topic that, increasingly, features in Design Codes and in the specifying of the design briefs for certain structures. Put simply, it is about assessing the ability of a structure to 'take a knock'.

This paper will summarise some of the progress made at Imperial College London in devising a framework for assessing the susceptibility to progressive collapse of steel and steel-concrete composite framed buildings, together with the subsequent work on identifying the most effective strategies for improving the resistance of already largely designed schemes.

2. DESIGN APPROACHES

Whilst some work of relevance to the understanding of the mechanics of Progressive Collapse might have been conducted prior to the Ronan Point collapse in 1968, reviews of the subject do not list anything obvious. Indeed, the Report on the Ronan Point collapse [2] appears to include the first suggestions of ways in which susceptibility to Progressive Collapse and/or provision of some degree of inherent Robustness should be tackled. Three approaches were identified:

1. Tying capacity
2. Key elements
3. Alternate load path

Tying capacity, which is illustrated in Fig. 1, relies on the tensile resistance of column splices and beam to column connections holding the structure together. Normally only capacities are specified. Since the development of tying forces in beam to column connections is associated with catenary action, which itself requires very large beam deflections and thus substantial rotation of the connections, for the concept to be valid the connections must possess the required very large degrees of ductility.

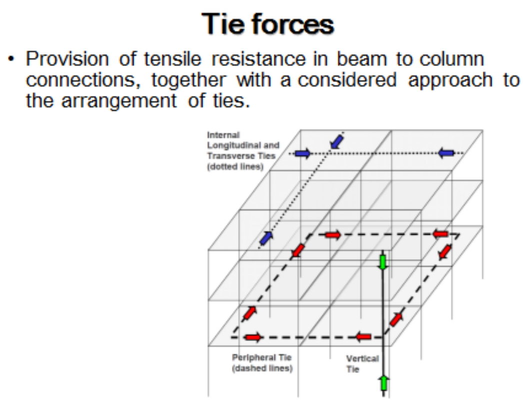


Figure 1: Tying capacity

The principle of key elements simply identifies those parts of the structure thought to be most influential in controlling the response of the structure should a triggering event occur. An obvious example would be transfer beams supporting several floors directly above. Fig. 2 illustrates the concept for the different situation of fire protection and illustrates the different performance of protected and unprotected columns.

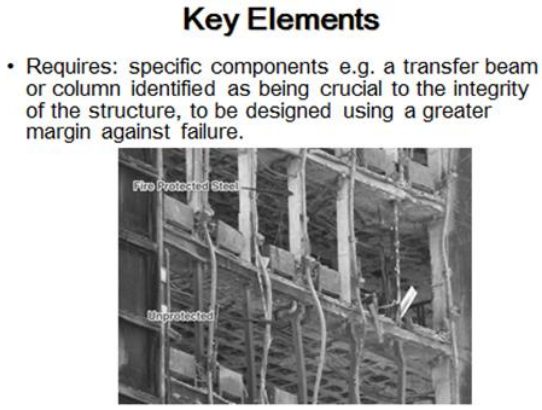


Figure 2: Key elements

The Alternate Load Path approach, which is illustrated in Fig. 3, assumes loss of one or more members (usually columns) and then seeks to assess the ability of the remaining structure to satisfactorily withstand the applied loading. Thus it relies on the principles of redundancy, ductility and redistribution of load to check that the damaged structure will be stable. The earliest uses of this approach did not specify how such assessments should be conducted; in particular, they did not specify a criterion of failure.

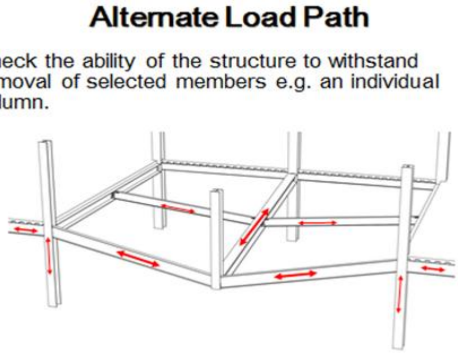


Figure 3: Column removal

Of these 3 approaches only the third offers the possibility of making quantitative comparisons between alternative arrangements. Both Tying and Key Elements are essentially prescriptive in nature i.e. they offer the promise of superior performance if their provisions are met but provide no indication of the extent to which one arrangement might be better than another (other than their pass/fail check).

A valid criticism of the Alternate Load Path approach is that it uses an arbitrarily selected triggering event - usually the removal of a single column - as the basis for the subsequent assessment of the damaged structure. In cases where the triggering event is known or can be specified e.g. a particular size of bomb placed at a particular location within the structure, then the response to this defined event should be investigated using appropriate techniques. However, in many cases the requirement will be for 'improved resistance to progressive collapse' or 'greater robustness' with the exact nature of the potential threat being unknown. Thus the Alternate Load Path concept used with the notion of single column removal represents a threat independent approach with a common basis for assessing alternative solutions. In this respect it has similarities with the '3 second gust' or the 'standard fire' used when designing against wind loading or assessing resistance to a building fire.

3. SCIENTIFIC STUDY

Fig.4 shows how the scientific study of Progressive Collapse as measured by published journal papers has developed over the past quarter of a century. Interestingly, a distinguished researcher in the field, Bruce Ellingwood, wrote in a paper [4] to the 1997 Structural Engineers World Congress: 'There is currently a virtual absence of research activity or interest in the United States in this topic'. The effect of the events of September 11 2001 is clear in the almost tenfold increase between 1999 and the peak of 2013.

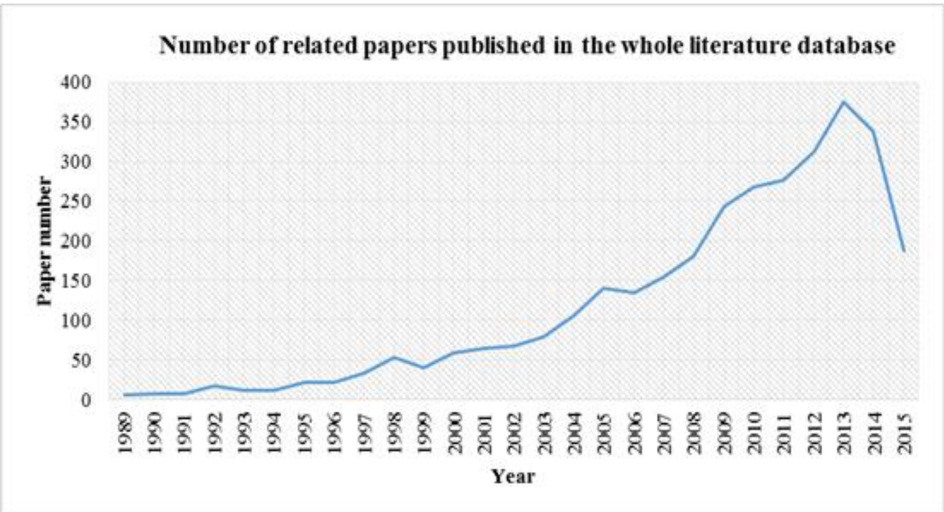


Figure 4: Growth of papers on robustness

Much of the work reported in that period relates to numerical studies using large FEA packages, several of them vying with one another to provide the most credible simulation of the WTC collapses. Those undertaken by NIST in the immediate aftermath employed the most powerful computers available in the USA and took weeks to produce a single result. More recently greater ingenuity by the analysts together with a better understanding of which aspects of structural behaviour are of greatest significance has improved this situation but it remains the case that any approach based on realistic FE modelling requires the availability of substantial computing resource, together with skilled numerical analysts who also understand the key physical features of the structural response. Unfortunately, many studies fail to employ a realistic criterion of failure, thus rendering their findings of questionable value.

The other major field of scientific study has been in devising and conducting carefully designed laboratory experiments replicating the behaviour of key components within a structure undergoing a progressive collapse. Fig.5 illustrates one such test in progress at the University of Liege [5]. Removal of a column is simulated by the application of a concentrated load and the response of the structure as it undergoes gross deformation carefully monitored. Of particular interest in such studies is the changing pattern of forces observed in the beam to column connections; this is very different from that experienced under normal gravity and/ or wind loading.

Figure 5: Test simulating column removal

Taking all of these studies together the main outcome has been that the behaviour of a structural frame and thus of its key individual components when undergoing progressive collapse is very different from that experienced in the more usual structural design cases. Whilst behaviour is still controlled by the 3 'eternal truths' of: strength, stiffness and ductility, the precise interplay between these is complicated and varies in nature for different structural arrangements. Thus it has not, so far, been possible to identify simple and widely applicable guiding principles.

4. THE IMPERIAL COLLEGE LONDON ROBUSTNESS ASSESSMENT FRAMEWORK

The study of the mechanics of Progressive Collapse in steel and composite frame structures began at Imperial College London in 2003; it is still in progress. The initial thrust was identifying the physics of the process and representing the key features by relatively simple mechanics. This involved the study of actual collapses together with potentially helpful material from different but related incidents. For example, Figs. 6 and 7 show bomb damaged buildings resulting from the German air raids on London during the Second World War. In the first, catenary action has developed as a result of the removal of some columns. Note the very large beam deflections and connection rotations involved. The importance of ductility as a way of absorbing energy in a benign fashion is illustrated in Fig. 7. A similar result may be observed in Fig.8, in which a grossly deformed but still intact building after an earthquake in Mexico City is shown. In all 3 cases the structures have survived - albeit in a damaged and grossly deformed state - because there has been no separation at the beam to column connections.

This careful preparatory work led to the identification of a number of critical features, which, if not included in a suitable manner, would, most likely, mean that solutions would not properly replicate the essential physics of Progressive Collapse. They are:

1. The actual collapse occurs rapidly and is, therefore, a dynamic event.
2. It involves the structure undergoing gross deformations.
3. Inelastic behaviour of material and components will occur.
4. Key to the containment of the triggering event is the avoidance of separation.
5. Members and connections directly involved in the damaged regions will be subject to very different loading demands from those experienced under normal loading cases.

The essential features of the resulting Robustness Assessment Framework [6] are illustrated in diagrammatic form in Fig. 9. The assessment may be conducted at structure, substructure, floor, grillage or beam level with the response of each progressively more extensive system being assembled from the responses of its individual components. Since an important conceptual consideration was to arrive at a procedure for which the resulting structural calculations were of a similar extent and level of complexity to those needed for conventional design the main focus has been on working with floor grillages.

Figure 6: WW2

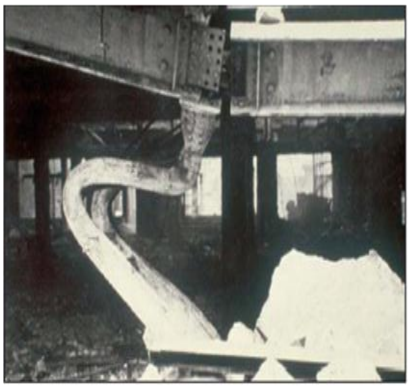


Figure 7: WW2

Figure 8: Mexico City earthquake

Key features of the Framework are:

1. Only static analysis is required, dynamics being incorporated through the concept of energy balance.
2. The actual analysis may be conducted using any suitable software, alternatively an enhanced slope deflection method that permits very rapid calculation using a spreadsheet may be employed.
3. Quantitative comparisons between alternatives may readily be made.
4. The steps involved are very similar to those used for conventional design.
5. Failure is defined in terms of the available connection ductility under the loading experienced at gross deformation.

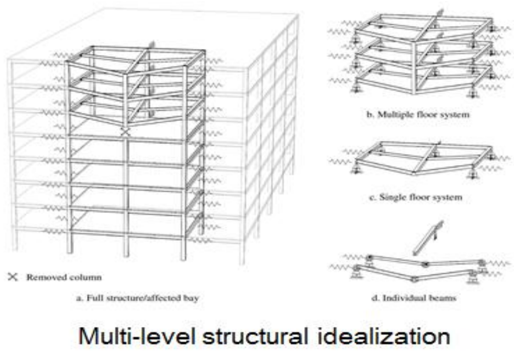


Figure 9: Frame to beam substructuring

Some particularly important aspects of the implementation are illustrated in Figs. 10-13.

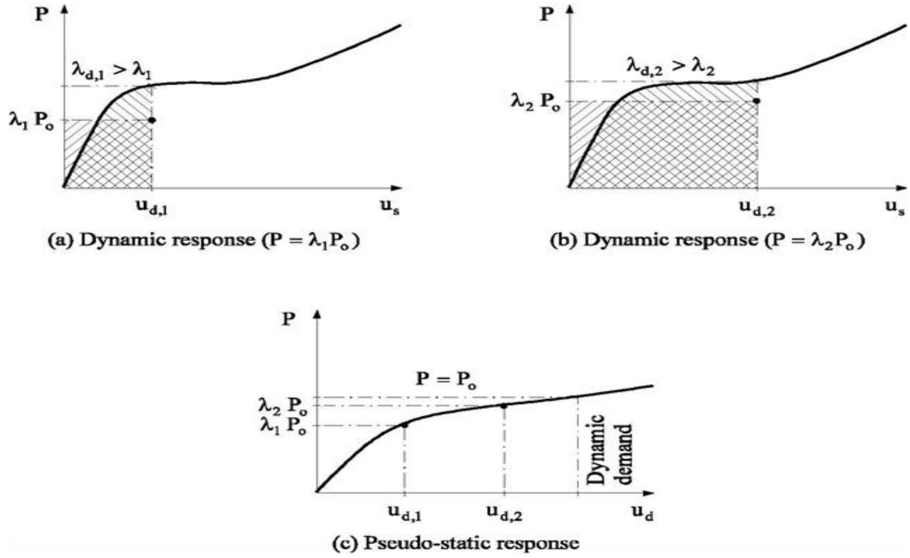


Figure 10: Simplified dynamic response assessment and definition of pseudo-static response.

Fig.10 illustrates the concept of 'converting' results obtained from a static analysis into the so called pseudo-static response that allows for the dynamic nature of Progressive Collapse. Crucial to the developing of a proper understanding is the exact nature of the response of an individual beam as it passes through the different forms of load resisting behaviour associated with the development of progressively larger deflections shown in Fig. 11; not all beams in all situations will be capable of reaching the later stages. Fig.12 shows how the loading experienced within a particular beam to column connection varies as beam deflections develop; this shows the very different requirements at stages beyond those normally associated with beam response i.e. after point B, The response of a complete floor grillage may be obtained by considering a suitable collapse mechanism of the form shown in Fig.13 using characteristics of the individual beams of the sort previously illustrated in Fig.10. If the analysis stage is conducted using the modified slope deflection approach then consideration of a single case is virtually instantaneous so that many alternatives may readily be studied.

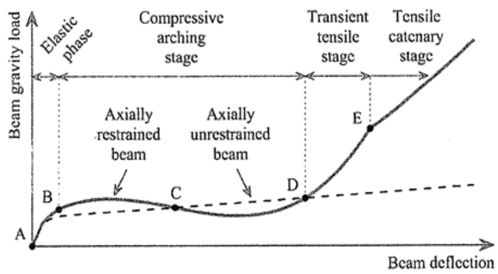


Figure 11: Beam nonlinear static load-deflection.

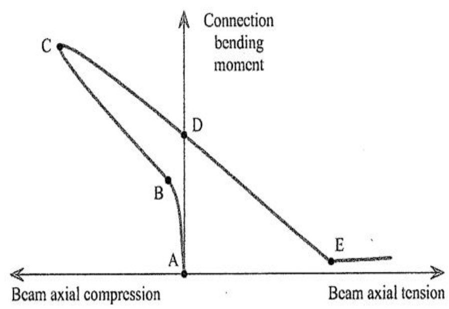


Figure 12 Beam axial load-connection bending moment interaction

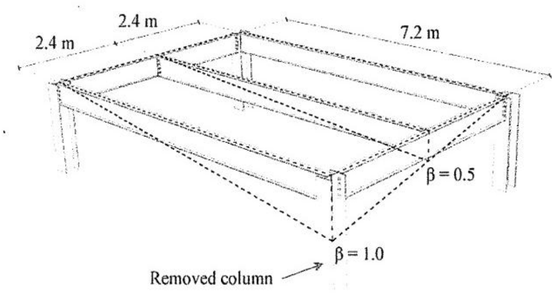


Figure 13: Grillage mechanism

5. ILLUSTRATIVE EXAMPLES

Throughout the course of the work at Imperial many example cases have been considered and parametric studies conducted; the sample of results presented now represent only a very small fraction of all the findings and the resulting improved understanding that has been generated.

Fig. 14 shows one of the earliest cases investigated [7]; it is modelled on an actual structure that was being designed in London in 2005. Table 1 compares the response in terms of the percentage of demand actually provided by 5 different beam configurations. Every case satisfies the tying force requirements of the UK Building Regulations. Case 5, which uses non composite bare steel beams is clearly inadequate, whilst of the 4 composite arrangements involving different percentages of reinforcement only that utilising significantly more than the Eurocode minimum requirement of 0.87%, that also presumes the presence of axial restraint to the floor beams so that pull in is resisted, actually meets the design condition.

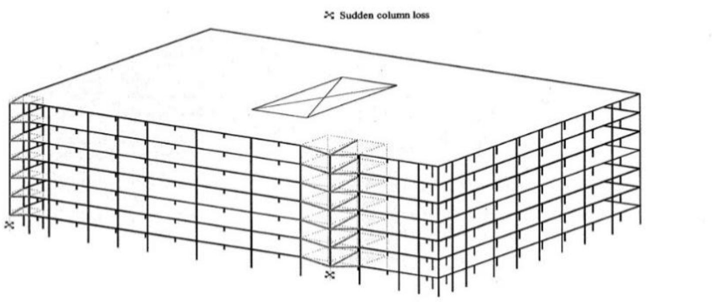


Figure 14: Layout of the seven-storey steel-framed composite building

The findings of Table 1 prompted several studies of the linkage (or not) between the provision of tying resistance and the ability to withstand column removal [8]. A particularly interesting finding is shown in Fig. 15, in which resistance to Progressive Collapse is plotted against the tying resistance of the support connection for a series of composite arrangements. There is little obvious correlation, with similar levels of tying capacity being associated with a fourfold variation in resistance to Progressive Collapse and fourfold increases in tying capacity producing almost no difference in resistance to Progressive Collapse. More recent work [9] of the sort that produced the detailed trace of connection demand shown previously in Fig. 12 and which developed this further by studying forces in the individual components within the connection e.g. bolts in tension, end plate in bending etc., has enabled the reasons for this to be understood. Put simply, it is not enhanced tying capacity per se that improves resistance to Progressive Collapse but for some arrangements that increased tensile resistance will result in greater ability of the connections to resist the actual combinations of force demand; however, in those cases for which connection behaviour is not much influenced by the performance of the tensile regions little, if any, benefit will result.

Table 1 Resistances for 5 different beam arrangements

Case No.		Capacity P (N)	Demand P_o (N)	Capacity/Demand ratio
1	(ρ varies, WR)	554711	741990	0.75
2	(ρ = 0.87%, WR)	598729	741990	0.81
3	(ρ = 2%, WR)	774358	741990	1.04
4	(ρ = 2%, NR)	709675	741990	0.96
5	(BS, WR)	148530	741990	0.20

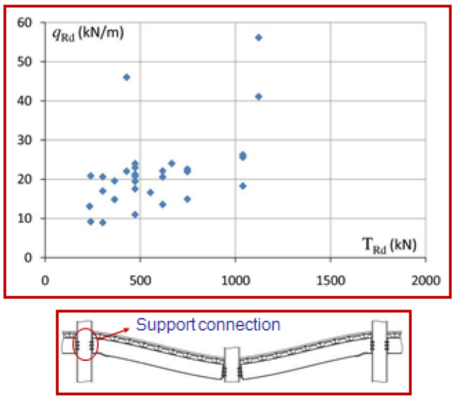


Figure 15: Tying capacities

The findings from a typical parametric study [10] given in Figs. 16 and 17 provide some indication of what is possible using the Imperial College approach. Nine different column removal scenarios are shown in Fig. 16 whilst results for one bare steel and one composite beam arrangement for each are provided in Fig. 17; Table 2 lists the associated floor capacities. Examination of results of this type, together with many more arrangements, have permitted the most important influences on behaviour to be identified. In simple terms these are: the number of beams involved, the percentage of these that are short span beams and the presence of cantilever beams. Whilst the finding that behaviour is improved by having more beams, especially short span beams without cantilevers, might appear obvious, being able to quantify this is, of course, an essential requirement of sound design. The Imperial studies have produced a design strategy [11] aligned with what is feasible in practical situations e.g. whilst rearranging column spacing might provide a very efficient solution it is likely to be

Impractical but modifying connection details is relatively easy. Initially devised for frames designed according to the principles of 'simple construction', a more recent parallel study has considered the behaviour of frames designed to provide varying levels of resistance to seismic action [12].

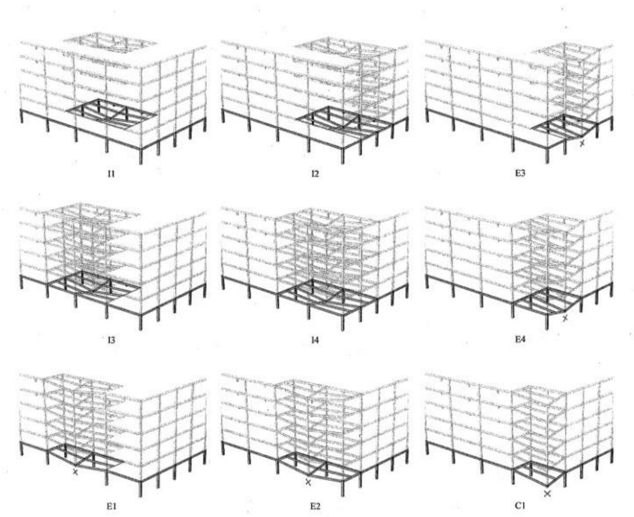


Figure 16: Column removal scenarios

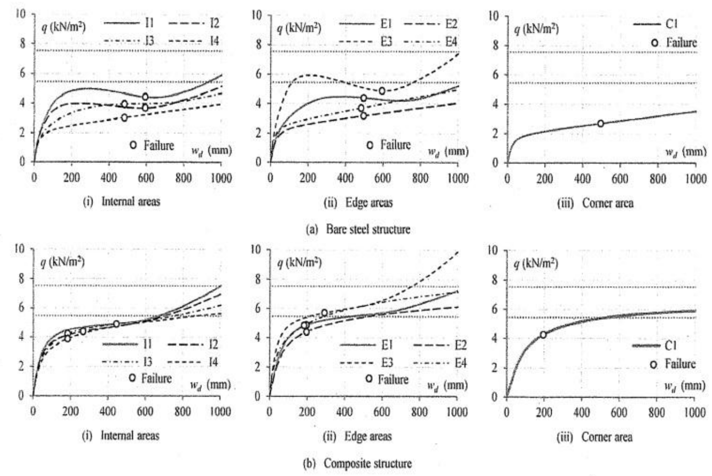


Figure 17: Floor responses

Table 2: Plastic capacities of grillage systems

Column removal scenario	Floor area (m^2)	Grillage plastic capacity			
		Bare steel structure		Composite structure	
		P_{dy} (kN)	q_{dy} (kN/m^2)	P_{dy} (kN)	q_{dy} (kN/m^2)
I1-I4	138.24	325.2	2.35	592.4	4.29
E1, E2	69.12	174.8	2.53	344.2	4.98
E3, E4	69.12	187.2	2.71	374.4	5.42
C1	34.56	71.3	2.06	180.7	5.23

6. CONCLUSIONS

The key features involved in a Progressive Collapse have been identified from the study of actual incidents. Appropriate representation of these has formed the foundation for the development of the Imperial College Robustness Assessment Framework that permits quantitative comparisons to be made between alternatives using a process that closely resembles design for gravity loading. Results from a number of examples have been used to illustrate how better insights into actual behaviour may be drawn and to show how established approaches to design may actually be misleading in some cases.

ACKNOWLEDGEMENTS

The work reported herein results from the efforts of many colleagues and research students at Imperial. Principal among the former are Professors Izzuddin and Elghazouli and among the latter Drs Vlassis, Stylianidis and Vidalis.

REFERENCES

[1] Wikipedia, *Collapse of the World Trade Centre*.

[2] H. Griffiths, A. G. Pugsley, O. Saunders, 1986. *Report of the inquiry into the collapse of flats at Ronan Point*, Canning Town. HMSO, London.

[3] M. Levy, M. Salvadori. *Why buildings fall down?* 1992. W. W. Norton and Co Ltd, London and New York.

[4] A. Longinow and B. Ellingwood. 2008. *The impact of the Ronan Point collapse - 25 years after.* Elsevier N. K. Srinivastava ed. Structural Engineering World Wide 1998, Paper P312-2.

[5] J-F Demonceau and J-P Jaspart. 2010. *Experimental test simulating a column loss in a composite frame.* Advanced Steel Construction, 6(3), pp 891-913.

[6] B. A. Izzuddin, A. G. Vlassis, A. Y. Elghazouli, D. A. Nethercot. 2007. *Assessment of progressive collapse in multi-storey buildings.* Proceedings of the Institution of Civil Engineers - Structures and Buildings, 160(4), pp 197-205.

[7] A. G. Vlassis, B. A. Izzuddin, A. Y. Elghazouli, D. A. Nethercot. 2008. *Progressive collapse of multi-storey buildings due to sudden column loss - Part II:* Application. Engineering Structures, (30)5, pp 1424-1438.

[8] D. A. Nethercot, P. Stylianidis, B. A. Izzuddin, A. Y. Elghazouli. 2009. *Resisting progressive collapse by the use of tying resistance.* Research Publishing Singapore. B. Uy, F. Mashira, X. Zhou, O. Mirza, E. L. Tan eds. 4th International Conference on Steel and Composite Structures, Sydney, Australia, pp 94-104.

[9] P. Stylianidis, D. A. Nethercot, B. A. Izzuddin, A.Y. Elghazouli, 2015. *Modelling of beam response for progressive collapse*. Structures, 3, pp 137-152.

[10] P. Stylianidis, D. A. Nethercot, B. A. Izzuddin and A. Y. Elghazouli, 2015. *Robustness assessment of frame structures using simplified beam and grillage models.* Submitted for publication.

[11] C. A. Vidalis, D. A. Nethercot. 2014. *Redesigning composite frames for progressive collapse.* Proceedings of the Institution of Civil Engineers - Structures and Buildings, 167(3), pp 153-177.

[12] C. A. Vidalis, D. A. Nethercot. 2015. *Improving the resistance to progressive collapse of steel and composite moment frames.* N. Ingraffea, M. Libby, eds. ASCE Structures Congress, Portland, Oregon, USA, April 23-25, pp 1138-1149.

CICM 2015
First International Conference on
Advances in Civil Infrastructure and Construction Materials
MIST, Dhaka, Bangladesh, 14–15 December 2015

ENHANCING THE EFFICIENCY OF STRUCTURAL STRENGTHENING FOR RC BEAMS

Mohd Zamin JUMAAT[1], Md. M. Rahman[2], Md. A. Hosen[3], M. Obaydullah[4] and Kh. M. Darain[5]

[1, 2, 3, 4,5] Department of Civil Engineering, Faculty of Engineering, University of Malaya, Kuala Lumpur, Malaysia.
Email: [1]zamin@um.edu.my, [2]rahmanmosh72@yahoo.com, [3]enggakter@gmail.com, [4]md_obaydullah@yahoo.com and [5]khmahfuz@gmail.com

Abstract. *Reinforced concrete (RC) structures often require strengthening due to error in design, degradation of materials properties after prolong usages and increases in the load capacity induced by new utilization of the structures. This paper highlight a new approach in strengthening of RC beams by combining the externally bonded reinforcement (EBR) and near surface mounted (NSM) techniques, which will be referred to as hybrid strengthening method (HSM). Along with this new technique several approaches were also proposed to increase the efficiency in strengthening of RC beams. RC beams were cast and strengthened in flexure using the three technique mentioned above. Also, cement mortar was used for replace of epoxy adhesive in order to reduce cost of NSM technique. Four-point bending tests were performed under static and fatigue condition. Semi-numerical model was formulated to initiate the preparation of the design procedure. A 3D finite element models (FEM) was developed to simulate the flexural response of the tested beams. The failure characteristics were evaluated based on the experimental output. The experimental results show that the flexural capacity of hybrid strengthened beams increased up to 65% and 36% compared with the control beam and the EBR strengthened beams respectively. The partial replacement of epoxy adhesive with cement mortar in NSM strengthening reduced costs without significantly affecting the flexural performance. Fatigue performance of hybrid strengthened beams seemed to be better that that of NSM strengthened beams. The proposed semi-numerical and FEM models showed excellent agreement with the experimental results.*

Keywords: Hybrid strengthening, Flexural performance, Semi-numerical and FEM.

1 INTRODUCTION

Rehabilitation and strengthening of RC structures are some of the major challenges facing structural engineers nowadays. Repair and strengthening of structures is a dynamically growing division of structural engineering and in recent years there has been an increase in the application of new repair and strengthening systems. Structures that have been built more than several decades ago often require strengthening and upgrading to meet current service load demands. Thus, the strengthening is expected to grow rapidly over the subsequent few years.
Several methods of strengthening RC structures using various materials have been studied and applied in the rehabilitation field [1, 2]. One of the earlier technique to enhance the strength or serviceability of RC structures is be gluing steel or CFRP plates to the outer surfaces of the structures. The approach is normally referred to as externally bonded reinforcement (EBR). The use of this technique usually suffers from premature failure like plate end separation, intermediate crack induced de-bonding or shear failure. This de-bonding can cause serious brittle and catastrophic failure before the strengthened beam has reached its ultimate capacity.
Many studies have been conducted to find solutions to this brittle de-bonding and to reduce the interfacial stresses between the RC substrate and the strengthening plate [3, 4]. One remedy was to change the thickness of the steel or FRP plate [5]. The more common approach to eliminate it, by using end anchorage [6].
Recently, the near-surface mounted (NSM) reinforcement has attracted a lot of research interest [7-10]. Most experimental studies on this strengthening technique investigated the flexural behavior of concrete beams strengthened using NSM FRP bars or strips [11-14]. Kishi et al. [15]. The experimental results indicated that the load capacity increased as the bond length increased and two types of failure mode occurred. One was debonding in the concrete epoxy interface and the other was debonding in the CFRP rod epoxy interface. Jung et al. [16] compared the NSM CFRP strengthened beams with EBR strengthened beams. The NSM CFRP bars and EBR strengthened specimens failed by debonding. The flexural responses were assessed of NSM strengthened RC beams using FRP bars. All the strengthened beams failed by debonding [17-18].
NSM technique has some limitations to its application. Sometimes, the width of the beam may not be sufficiently wide to provide necessary edge clearance and clear spacing between two adjacent NSM grooves. Lorenzis [19] recommended the minimum edge clearance and clear spacing for the NSM grooves should be four and two times the groove depth. Rahman et al. [20] evaluated the flexural performance of hybrid strengthened RC beams using steel plate and bars. The experimental results revealed that HSM is better compared with the EBR.
However, very few experimental investigations have been done on the flexural behavior of concrete beams strengthened with NSM steel bars and epoxy replaced by cement mortar [21].

In this experimental studies includes: I) the effectiveness of the proposed hybrid bonding method; II) the fatigue performance of RC beams strengthened with EBR, NSM and hybrid techniques; III) using cement mortar to replace epoxy and NSM-steel bars compared with EBR strengthening method; IV) to develop semi-numerical and 3D FEM models to predict the flexural behavior of strengthened beams.

2 EXPERIMENTAL PROGRAMME

The experimental programme was developed to verify the effectiveness of the use of steel bar with cement mortar in NSM and the proposed hybrid bonding technique. The RC beams and their different fabrication stages, the procedures used for strengthened of the beams, the instrumentation of the beams and the test-setup are described below.

2.1. Specimen configurations

All the beams were 2300mm long, 125mm wide, and 250mm deep. The beams were reinforced with two 12 mm diameter steel bars in the tension zone as the main reinforcement. Two 10 mm steel bars were used as hanger bars in the shear span and were placed at the top of each beam. For shear reinforcement 6 mm bars were used and were placed symmetrically apart. The spacing of the shear reinforcement was 75mm to ensure the beams would fail in flexure. The details of the internal reinforcement used in a typical beam are shown in Figure 1. A typical concrete cover of 30 mm was used.

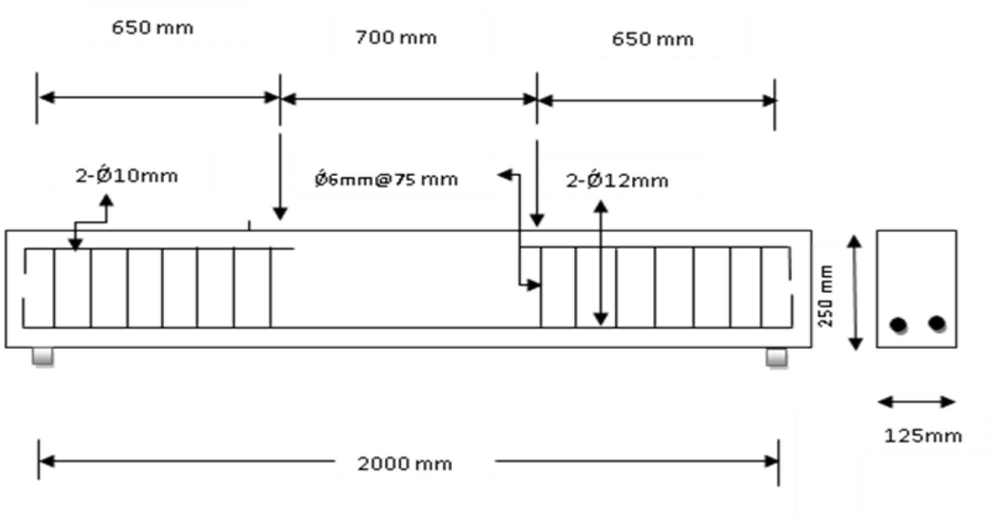


Figure 1: Details of the beam specimens

2.2. Materials properties

A single batch of concrete supplied by a local ready mix company was used to cast all the specimens. The average 28-day compressive strength of the concrete was 30 MPa based on testing three 100 mm x 200 mm cylinders. The yield and ultimate strength of the steel reinforcement was 550MPa and 640 MPa respectively and its young's modulus was 200 GPa based on testing three steel coupons accordance with ASTM. Three different thicknesses of steel plates were used i.e. 1.5 mm, 2 mm and 2.76 mm. The yield and ultimate tensile strength of the steel plates were 420 MPa and 475 MPa and the modulus of elasticity was 200 GPa.

The tensile strength and modulus of elasticity of CFRP plate were 2800 MPa and 165 GPa respectively. The design and ultimate strains of CFRP plate were 0.0085 and 0.017 respectively. The thickness of CFRP fabric was 0.17 mm. The tensile strength and modulus of elasticity was 4900 MPa and 230 GPa respectively. The elongation at break was 2.1%. Sikadur® 30 epoxy adhesive was used as the bonding agent between the strengthening materials and the concrete substrate. The tensile strength at seven days was 24.8 MPa; it has an elongation to failure of 1%, and a modulus of elasticity of 2.69 GPa.

2.3. Strengthening procedure

For EBR technique, the surfaces of both concrete and steel plates require special preparation for proper bonding between the concrete and whatever strengthening material was used. All dust, laitance, grease, curing compounds, foreign particles, disintegrated materials and other bond inhibiting materials must be removed from the bonding surfaces. The concrete surface thoroughly cleaned. The bonding faces of all concrete beams were ground with the help of a diamond cutter to obtain a rough surface and to expose the texture of the coarse aggregate. The ground concrete surfaces were then cleaned to remove dust, loose particles and any other foreign material by wire brush and a high pressure air jet. The bonding surfaces of the steel plates were sand blasted in accordance with Swedish standards to ensure adequate bonding between the concrete and steel plates. And, The CFRP laminates were cleaned using Colma cleaner to remove carbon dust from the bonding surfaces.

For NSM or hybrid strengthening technique, either one or two grooves were cut along the length of the tension faces of the concrete beams for the placement of the NSM bars. The grooves were made by making parallel cuts with a diamond concrete saw as deep as the desired depth of the NSM groove. The grooves also cleaned using a wire brush and a high pressure air jet to remove dust and loose particles.

Strengthening was done using the NSM and hybrid technique. Steel plates, steel bars and CFRP plates were used in various configurations for strengthening. Hybrid strengthening method a combination of either steel plate and steel bars or CFRP plate and steel bars was used. For NSM strengthening, the prepared groove was half-filled with the epoxy adhesive and then the NSM steel bar was pressed into the center of groove until the adhesive flowed around of the bar. The remaining space in the groove was filled with epoxy and levelled using a spatula. For hybrid strengthening, the beams were first strengthened using the NSM technique and then using the EBR technique.

2.4. Instrumentation and loading test setup

A Linear Variable Displacement Transducer (LVDT) with transverse range of 50 mm was used to measure the deflection of the beam at mid-span .The transducer was connected to a data logger to record the reading of the deflection of the beam during the test. In addition, the actuator position of the INSTRON universal testing machine was monitored to measure the deflections at the mid-span of all beams, to avoid the damage of the LVDT after initiating the failure of the beams. The strain gauges measured the strains in the steel bars, steel or CFRP plate and the concrete extreme fiber. Two 5 mm gauges were then attached to the middle of the internal reinforcing bars by fast setting adhesive on bottom face of the two main steel rebars to record the tension strains.

Two 30 mm strain gauges were placed at the middle of the extreme fiber of the beam and bottom of the strengthening steel/FRP plate to measure the concrete compressive and plate tensile strain. The data logger was also connected with the digital controller of the testing machine and strain gauges attached to the beams, for collecting the real-time loading and strains. The readings were scanned at a time interval of one second. A Dino-lite digital microscope was used to measure the crack width of the beams during the test. All beams were tested under four-point bending and the tests were conducted with a closed-loop hydraulic Instron Universal Testing Machine. For the static load tests, the actuator was loaded and moved down at a slow rate so that readings from the data logger could be taken and visible cracks measured easily (Figure 2). For fatigue test, a programmed to deliver a sinusoidal load at a frequency of 3Hz was used. The load span, load set point, frequency and preset number of cycles were controlled by an electronic controller (MTS® 407 Controller). The sinusoidal waveform was checked using a conventional oscilloscope. The loading was monitored using a fatigue-resistant load cell.

Figure 2: Experimental set up

3 SEMI-NUMERICAL MODEL

Concrete is a semi-brittle material and behaves differently in tension compression and in compression. The ultimate uniaxial tensile strength and compressive strength are required to define a failure surface for the concrete. In tension, the concrete stress–strain curve is linearly elastic up to the ultimate tensile strength. After this value, the concrete cracks and the strength reduces to zero. The compression and tension reinforcement are assumed to be elastic-plastic (bi-linear behavior). The stress-strain curve for CFRP strip is linearly elastic up to failure.

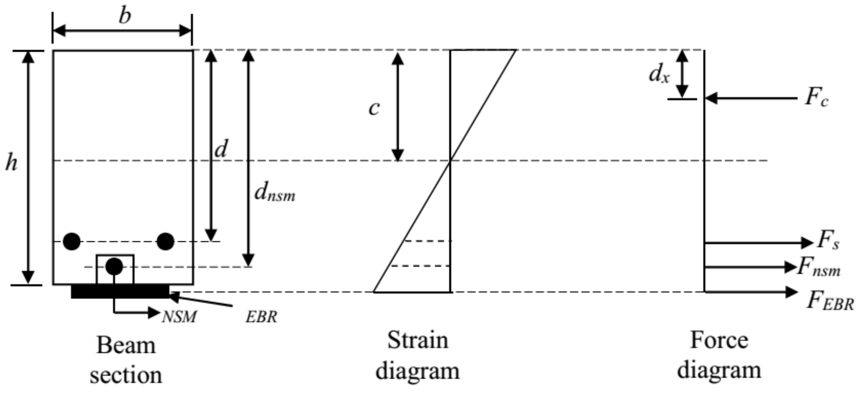


Figure 3: Strengthened beam section with strain and force distribution.

From the Figure 3, the ultimate moment (M_u)

$$M_u = F_c(c - d_x) + F_s(d - c) + F_{nsm}(d_{nsm} - c) + F_{EBR}(h + \frac{t}{2} - c) \qquad (1)$$

Where, F_c = the force carried by the compressive concrete, F_s = the force carried by the tension reinforcement, F_{nsm} = the force carried by the NSM reinforcement, F_{EBR} = the force carried by the EBR, h = depth of the beam, d_x = depth of concrete compressive force, d = effective depth of the beam, d_{nsm} = depth of the NSM reinforcement, c = depth of the neutral axis of the beam cross-section, and *t = thickness of EBR.*

4 FINITE ELEMENT MODEL (FEM)

The FEM method is a useful technique in solving highly nonlinear problems in continuum mechanics as RC structures exhibit highly nonlinear behavior, especially approaching failure load. Numerical model has been developed using the ABAQUS program to predict the load deflection behavior of RC strengthened beams. For many structural materials such as steel and aluminum which have well-defined constitutive properties, element method works very well but when the constitutive behavior is not so straight forward like concrete in which discrete cracking occurs, the task is more difficult. The objective of this part of the study is to establish a reliable, convenient and accurate methodology for analyzing strengthened RC beams which can correctly represent global beam behavior.

4.1. Geometrical model

The structural member is modelled as a mesh of finite elements. A wide range of elements are available in ABAQUS. Among these, continuum elements are the most comprehensive as they can be used in almost any linear/nonlinear stress-displacement and crack propagation analysis. Both two- and three-dimensional (2D and 3D) continuum elements are available however, 2D continuum elements can adequately investigate the behavior of the beams in this research. The2D elements can be either triangular (3 or 6 nodes) or quadrilateral (4 or 8 nodes).

The concrete is modeled using continuum elements; Continuum elements are provided with first-order (linear) and second-order (quadratic) interpolation and careful consideration must be decided as to which is more appropriate for the application. First-order elements use linear interpolation to obtain displacements at nodes, whereas second-order elements use quadratic interpolation to obtain displacements at nodes. ABAQUS offers two integration options.

Linear reduced-integration continuum elements are employed throughout the analysis with a fine mesh for their ability to withstand severe distortion in plasticity and crack propagation applications. All the elements in the model were purposely assigned the same mesh size to ensure that two different materials each share the same node. The type of mesh selected in the model was structured. The mesh element for the concrete, rebar and FRP laminate element were 3D solid, 2D truss and shell, respectively. The 3D mesh of strengthened beam is shown in Figure 4.

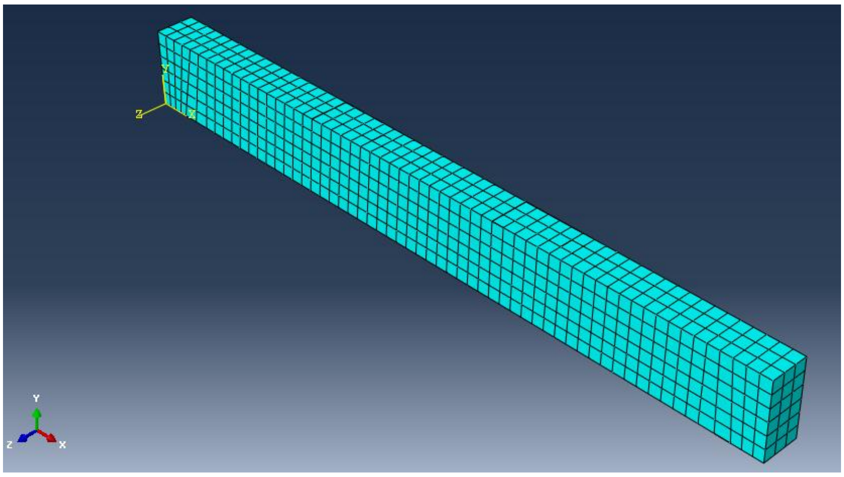


Figure 4: 3D finite element mesh of RC strengthened beam

5 RESULTS AND DISCUSSIONS

5.1. The effectiveness of the proposed hybrid strengthening method

The effects of hybridization on the static performance of the strengthened RC beams are shown in Figure 5. In both cases (plate length 1900 mm and 1650 mm) the failure load of the hybrid strengthened beam was greater than that of corresponding plate bonding method. The amount of strengthening materials was almost same (total cross-sectional area = 200 mm^2) for hybrid and corresponding EBR method but improvement in HSM is significantly higher. Specifically, the improvement in the de-bonding failure loads up to 27% compared with the EBR technique. This improvement was achieved in two way I) reduction of plate thickness II) increased bonding surface area.

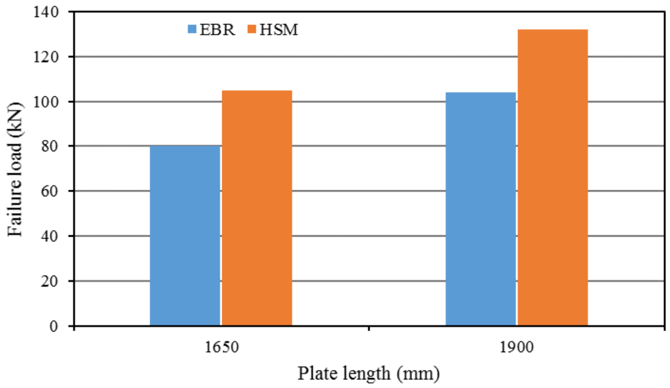


Figure 5: Comparison of failure load between EBR and HSM

5.2. Fatigue performance of the strengthened beams

Two modes of failure were observed for the cyclically loaded RC beams. Fatigue failure in the tension steel reinforcement was the usual mode of failure. This mode of failure was expected as the stress range in the tension steel reinforcement was high enough to cause fatigue failure in the steel. The control beams and the NSM strengthened beam failed in this mode of failure. The plate bonded strengthened beam and the hybrid bonded beam (with steel bar and steel plate) both failed in debonding. The fatigue life of the strengthened beams increased (Table 1). The fatigue life of the strengthened beams increased after strengthening due to the redistribution of stresses between the internal reinforcement and the external reinforcement, resulting in lower stresses in the internal steel reinforcement.

Table 1: Fatigue test results

Notation	Max^{m} Load (kN)	Number of cycles to failure	Post fatigue load (kN)	Failure mode
Control beam1	40	485000	-	Fracture of steel
Control beam2	64	188000	-	Fracture of steel
NSM strengthened beam	64	198000	-	Fracture of steel
EBR strengthened beam	64	>2000000	98	De-bonding
Hybrid strengthened beam	64	211000*	136.34	De-bonding

*After this cycle the load of the machine accidentally increased from 64 to 136.34 kN due to tripped and fatigue testing could not be continued.

5.3. The Effect of partial epoxy replacement with cement mortar

The effect of the partial replacement of epoxy with cement mortar is shown in Figure 6. The ultimate load of 50% epoxy replaced with cement mortar is almost similar to the ultimate load of epoxy used strengthened beams but significantly higher than cement mortar strengthened beams.

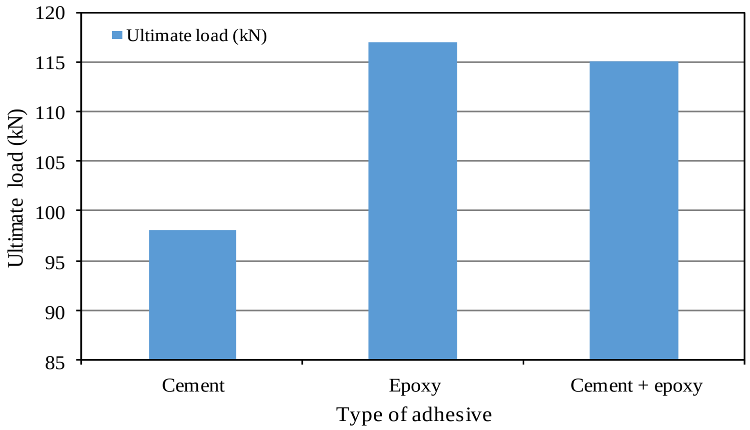


Figure 6: The effect of partial replacement of epoxy with cement mortar

5.4. Comparison of NSM with EBR technique

Compared to externally bonded reinforcement with the NSM techniques have several advantages. Figure 7 compares the performance of NSM strengthening and external plate bonded strengthening.

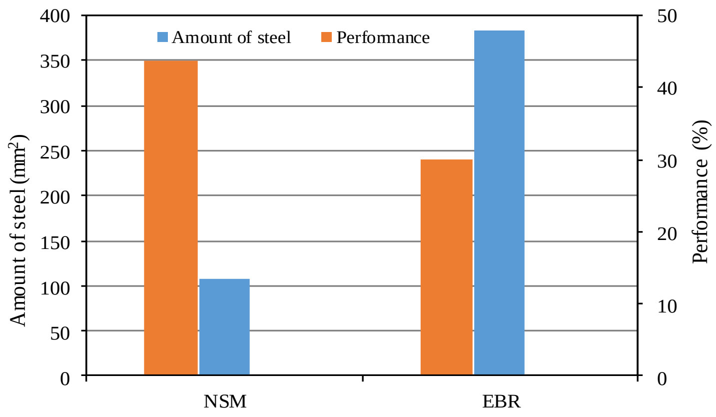


Figure 7: Comparison of NSM with EBR method

6 VERIFICATION OF MODELS

6.1. Verification of Semi-numerical model

The failure load of control and hybrid strengthened beam were evaluated by semi-numerical approach. The correlation between the experimental and the predicted results for these beams is within a reasonable range (0-1%) of agreement (Figure 8).

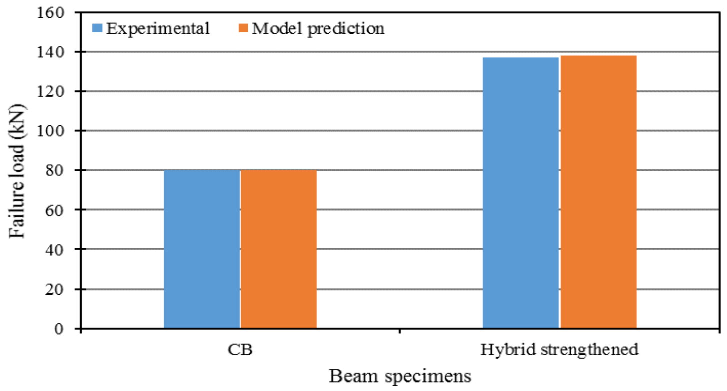


Figure 8: Predicted and experimental failure load

6.2. Verification of FEM model

The comparison between FEM and experimental results of control and hybrid strengthened beam specimens are shown in Figure 9. The predicted failure modes of the specimens are presented in Figure 10. There is a good agreement between the predicted and experimental load carrying capacities and failure modes of the tested specimens.

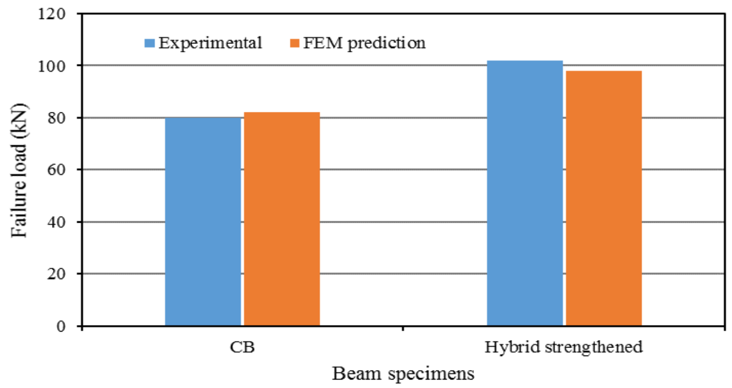
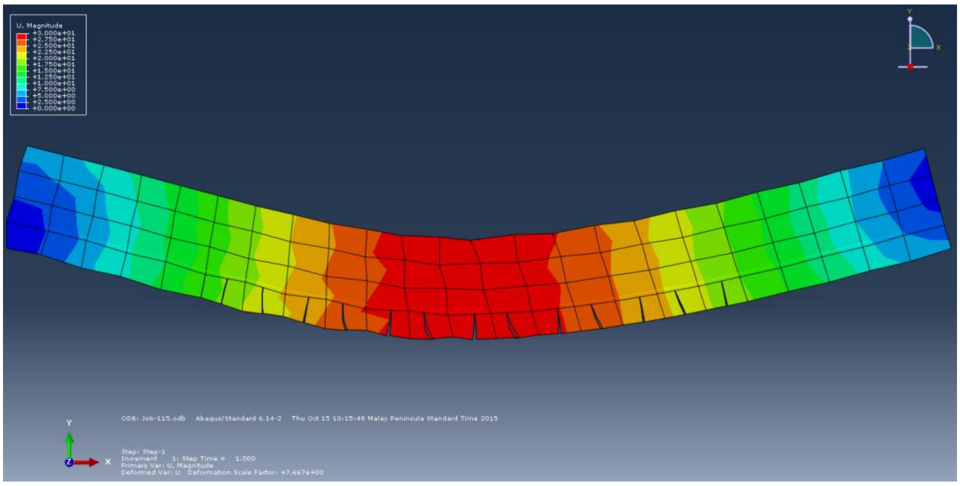
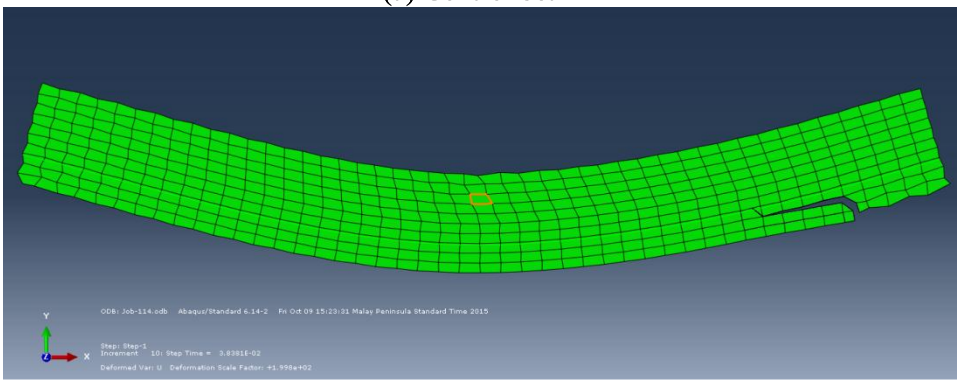


Figure 9: Comparison between FEM predicted and experimental results



(a) Control beam



(b) Hybrid strengthened beam

Figure 10: FEM predicted failure mode

7 CONCLUSION

On the basis of experimental and numerical results presented in this paper the following conclusions can be drawn:

- Hybrid strengthening technique was proved to be the effective alternative in all cases of monotonic and fatigue loading. The hybrid bonding technique uses smaller amounts of strengthening materials to increase the load carrying capacity of a beam. The load carrying capacity of the strengthened specimens increased up to 65%.
- Performances of hybrid strengthen beams up to 36% higher than of EBR, when the same amount of strengthening materials was used.

- Fatigue performance of hybrid strengthened beam is at least 6.5% better than that of NSM strengthened beam. Therefore, HSM was proved to be the effective alternative in all cases under monotonic and fatigue loading.
- RC beams where 50% of the epoxy adhesive was replaced with cement mortar in the middle part of the NSM groove, revealed almost similar flexural performances of the beams using 100% epoxy. The partial replacement of epoxy with cement mortar is an economical alternative to epoxy based NSM technique.
- NSM-steel bars for strengthened of RC structures are an economical alternative.
- The proposed semi-numerical and FEM models demonstrated a good agreement with the experimental results.

ACKNOWLEDGEMENTS

The authors are grateful for the financial support from the University of Malaya, High Impact Research Grant (HIRG) No. UM.C/625/1/HIR/MOHE/ENG/36 (16001-00-D000036)-‘‘Strengthening Structural Elements for Load and Fatigue’’.

References

[1] D. K. Eberline, F. W. Klaiber, and K. F. Dunker, Bridge strengthening with epoxy-bonded steel plates, 1988.

[2] M. MacDonald and A. Calder, "Bonded steel plating for strengthening concrete structures," International Journal of Adhesion and Adhesives, vol. 2, pp. 119-127, 1982.

[3] M. Fitton and J. Broughton, "Variable modulus adhesives: an approach to optimised joint performance," International Journal of Adhesion and Adhesives, vol. 25, pp. 329-336, 2005.

[4] M. Hildebrand, "Non-linear analysis and optimization of adhesively bonded single lap joints between fibre-reinforced plastics and metals," International Journal of Adhesion and Adhesives, vol. 14, pp. 261-267, 1994.

[5] M. Tsai and J. Morton, "The effect of a spew fillet on adhesive stress distributions in laminated composite single-lap joints," Composite structures, vol. 32, pp. 123-131, 1995.

[6] M. Z. Jumaat and M. A. Alam, "Experimental and numerical analysis of end anchored steel plate and CFRP laminate flexurally strengthened rein-

forced concrete (rc) beams," International Journal of the Physical Sciences, vol. 5, pp. 132-144, 2010.

[7] L. De Lorenzis and J. Teng, "Near-surface mounted FRP reinforcement: An emerging technique for strengthening structures," Composites Part B: Engineering, vol. 38, pp. 119-143, 2007.

[8] J. A. Barros, D. R. Ferreira, A. S. Fortes, and S. J. Dias, "Assessing the effectiveness of embedding CFRP laminates in the near surface for structural strengthening," Construction and Building Materials, vol. 20, pp. 478-491, 2006.

[9] O. Rosenboom and S. Rizkalla, "Behavior of prestressed concrete strengthened with various CFRP systems subjected to fatigue loading," Journal of Composites for Construction, vol. 10, pp. 492-502, 2006.

[10] J. Yost, S. Gross, D. Dinehart, and J. Mildenberg, "Near surface mounted CFRP reinforcement for the structural retrofit of concrete flexural members," in Proceedings of the 4th international conference on Advanced Composite Materials in Bridges and Structures (ACMBS 2004), Calgary, Alberta, Canada, 2004.

[11] F. Al-Mahmoud, A. Castel, R. François, and C. Tourneur, "Strengthening of RC members with near-surface mounted CFRP rods," Composite Structures, vol. 91, pp. 138-147, 2009.

[12] M. Badawi and K. Soudki, "Flexural strengthening of RC beams with prestressed NSM CFRP rods–experimental and analytical investigation," Construction and Building Materials, vol. 23, pp. 3292-3300, 2009.

[13] R. El-Hacha and M. Gaafar, "Flexural strengthening of reinforced concrete beams using prestressed, near-surface-mounted CFRP bars," PCI journal, vol. 56, pp. 134-151, 2011.

[14] S. M. Soliman, E. El-Salakawy, and B. Benmokrane, "Flexural behaviour of concrete beams strengthened with near surface mounted fibre reinforced polymer bars," Canadian Journal of Civil Engineering, vol. 37, pp. 1371-1382, 2010.

[15] N. Kishi, H. Mikami, Y. Kurihashi, and S. Sawada, "Flexural behaviour of RC beams reinforced with NSM AFRP rods," in Proceedings of the International Symposium on Bond Behaviour of FRP in structures (BBFS 2005), 2005, pp. 337-342.

[16] W.-T. Jung, Y.-H. Park, J.-S. Park, J.-Y. Kang, and Y.-J. You, "Experimental investigation on flexural behavior of RC beams strengthened by NSM CFRP reinforcements," ACI Special Publication, vol. 230, pp. 795-806, 2005.

[17] W. Tang, R. Balendran, A. Nadeem, and H. Leung, "Flexural strengthening of reinforced lightweight polystyrene aggregate concrete beams with near-surface mounted GFRP bars," Building and environment, vol. 41, pp. 1381-1393, 2006.

[18] I. Sharaky, L. Torres, J. Comas, and C. Barris, "Flexural response of reinforced concrete (RC) beams strengthened with near surface mounted (NSM) fibre reinforced polymer (FRP) bars," Composite Structures, vol. 109, pp. 8-22, 2014.

[19] L. De Lorenzis, "Strengthening of RC structures with near surface mounted FRP rods," 2000. (PhD Thesis), University of Lecce, Italy.

[20] M. M. Rahman, M. Z. Jumaat, M. A. Rahman, and I. M. Qeshta, "Innovative hybrid bonding method for strengthening reinforced concrete beam in flexure," Construction and Building Materials, vol. 79, pp. 370-378, 2015.

[21] M. M. Rahman, M. Z. Jumaat, M. A. Hosen, and A. S. Islam, “Effect of adhesive replacement with cement mortar on NSM strengthened RC Beam,” Journal of Construction (Accepted).