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Blast resistance of FRP composites and polymer strengthened concrete and masonry structures – A state-of-the-art review

P.A. Buchan [a,b], J.F. Chen [b,*]

[a] Weidlinger Associates Ltd., P.O. BOX 14488, Glenrothes, Fife, UK
[b] Institute for Infrastructure and Environment, School of Engineering and Electronics, The University of Edinburgh, The King's Buildings, Edinburgh, EH9 3JL, UK

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Abstract

Recent world events such as bombings in London, Madrid and Istanbul have highlighted the susceptibility of many civilian structures to terrorist attack. Explosives directed towards vulnerable structures may cause considerable damage and loss of life. As a result, there is now a desire to increase the blast resistance of many types of existing structures. This has led to experimental and finite element (FE) research in retrofitting concrete and masonry structures with fibre reinforced polymer (FRP) composites for blast protection. This paper presents a review of the publicly available literature and highlights areas where research is lacking.

Keywords: Blast retrofitting; A. Polymer-matrix composites; A. Fibres; B. Strength; C. Numerical analysis

1. Introduction

The US Department of State has reported that globally there were more than 11,000 terrorist attacks in 2005, killing more than 14,600 people [1]. The vulnerability of many civilian facilities to terrorist attack is highlighted by recent tragedies in London (2005), Madrid (2004), Istanbul (2003), Bali (2002) and New York (2001), which illustrates the global and current nature of the problem.

Attacks directed towards vulnerable structures may cause considerable damage and loss of life. As a result there is a requirement to increase the blast resistance of many types of structures. In particular, vulnerable and critical government, military and corporate buildings, strategic bridges and transport terminals, chemical, petroleum and nuclear plants are all at risk from terrorist attack. Consequently, there is now a desire to increase the blast resistance of many existing structures which have typically not been designed to resist an explosion. A cost-effective technique for this purpose is retrofitting with fibre reinforced polymer (FRP) composites.

Since the early 1990s, extensive research has been conducted to retrofit or strengthen existing concrete and other (masonry, metallic and timber) structures using externally bonded advanced FRP composites [2,3]. This strengthening technique has now become popular worldwide because of the superior properties of modern FRP composites, which have high strength to weight ratios and are effectively corrosion free. These retrofits can be easily applied, with minimal disruption to the structure and rapid completion. The method is also cost-effective compared with other methods such as strengthening with bonded steel plates [4].

Limited research has been conducted on the blast resistance of FRP or polymer strengthened reinforced concrete (RC) beams, columns and slabs, as well as masonry and concrete walls. This paper presents a review of publicly available literature on the blast resistance of FRP or polymer strengthened RC and concrete masonry unit (CMU)

* Corresponding author. Tel.: +44 0 131 6506768; fax: +44 0 131 6506781.
E-mail address: j.f.chen@ed.ac.uk (J.F. Chen).

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structures. Both experimental and numerical investigations are reviewed, with areas which require further research highlighted.

2. Potential blast resistant solutions

A number of solutions are available to minimise the damage of a structure in the event of an explosion. Minimisation to injuries and fatalities may be achieved through maintaining structural strength to reduce the risk of progressive collapse. Reducing fragmentation also plays an important role because it is a major source of injury in an explosion.

Maintaining the appearance of a building is important when it is historically or architecturally significant, which is the case with many government and corporate buildings. It is crucial for symbolic buildings not to appear like fortresses or bunkers, as this could affect public opinion. The solution should also be economically viable. This incorporates not only material purchase costs but also secondary factors such as not reducing the buildings valuable floor space, interrupting existing services or requiring highly skilled technicians for installation. After retrofitting the building should still be fit for purpose and have no major additional maintenance costs.

Maintaining a sufficient standoff is often the most effective way of protecting a structure from an explosion. This can be achieved with bollards, fences and walls, but is often impractical in urban environments where space is at a premium.

The blast resistance of a structure can be improved by increasing its mass and strength with additional concrete and steel reinforcement. Unfortunately, this solution can be expensive, add considerable gravity loads to the foundations of the structure and require a significant amount of time to install. Spalling and fragmentation are also problems when concrete and masonry are exposed to blast loads.

Catcher systems on the inside face of walls can be used to prevent fragments from entering an occupied space. For this method, a fabric covers the entire surface of the wall and is securely anchored at the floor and ceiling with just enough tension to remove slack. Special arrangements must be made for load bearing walls as this does not provide structural strength and for walls with windows as the fabric must span continuously without interruption.

Steel stud walls can be applied to the interior of existing walls to increase ductility and energy absorption. To maximise this ductility, the connection to the floor and ceiling must be well designed so they do not fail but instead the stud yields and failure can occur due to strain elongation. The disadvantages of this method are long installation times and loss of floor space.

In-situ enhancing of RC structures using externally bonded steel plates has previously been attempted. The mechanism of strengthening beams, walls and slabs with bonded steel plates is by increasing their flexural strength. The strengthening of RC columns is achieved through lateral confinement of the concrete which enhances the axial compressive strength and ductility. With this method there are disadvantages with lengthy installation times and corrosion, leading to increased through-life maintenance costs.

FRP composites are now being utilised instead of steel due to their higher strengths, better corrosion resistance and greater ease of transportation and handling. FRPs can be more readily arranged according to the specific site conditions than other materials and so optimised for performance. The method can be applied quickly and is non-intrusive so it does not intrude into the buildings floor space. It will also be shown in the research reviewed that it provides a dramatic increase in the ability of a structure to resist blast by increasing the structural strength and reducing fragmentation.

The apparent high purchase cost of FRP composites compared to other materials has previously been cited as a disadvantage for this technique. However, a direct comparison on a unit price basis may not be appropriate. Including installation and transportation in the cost comparison FRPs can often compete with conventional materials. If the comparisons include through-life costs FRPs can be advantageous [4]. A number of studies have also been conducted using polymer sprays to retrofit existing structures, which along with FRP retrofitting are presented in this review.

3. Summary of existing research

Both experimental and finite element (FE) research has been conducted to investigate the behaviour of FRP or polymer retrofitted structures under blast loads. A summary of the publicly available literature reviewed is presented in Table 1. It may be noted that static experimental tests have also been included if they are directly relevant to the discussed explosive research.

Table 1 shows that the majority of research has been concerned with CMU walls, which may reflect that they are the most vulnerable to fragmentation. All but one of the remaining studies is concerned with normal RC, with only light weight high strength (LWHS) concrete investigated by Ross et al. [5]. Table 1 also shows that a number of FE codes have been used, some of these (e.g. FLEX) are not commercially available.

4. Choice of retrofitting materials

Many FRP and polymer materials are available for retrofitting a structure. Selection of the most suitable material is necessary for optimal performance and cost. Table 2 presents a summary of materials assessed in the reviewed literature.

The majority of research has used glass fibre reinforced polymers (GFRP) and carbon fibre reinforced polymers (CFRP). Recent research has included aramid/glass (A/G) hybrids, GFRP rods and polymer sprays. Unfortu-

Table 1
Summary of experimental and FE studies

	Year	Structure	Experimental tests	FE code
Ross et al. [5]	1997	RC beams	Explosive	
Muszynski et al. [14,15]	1995, 2003	RC columns	Explosive	
Crawford et al. [10]	1997	RC columns		DYNA3D
Crawford et al. [16]	2001	RC columns	Explosive	PRONTO3D
Crawford et al. [16]	2001	RC columns	Static	
Ross et al. [5]	1997	LWHS slabs	Explosive	REICON
Mosalam and Mosallam [20]	2001	RC slabs		DIANA
Lawver et al. [18]	2003	RC slabs	Explosive	FLEX
Muszynski et al. [14,15]	1995, 2003	RC walls	Explosive	
Oswald and Wesevich [30]	2001	CMU walls	Shock tube	
Crawford et al. [28,29]	2002, 2003	CMU walls		DYNA3D
Carney and Myers [6]	2003	CMU walls	Static	
Muszynski and Purcell [9]	2003	RC walls	Explosive	
Muszynski and Purcell [9]	2003	CMU walls	Explosive	
Myers et al. [7,8]	2003, 2004	CMU walls	Explosive	
Tan [21]	2003	CMU walls		DIANA
Davidson et al. [12]	2004	CMU walls	Explosive	
Baylot et al. [25]	2005	CMU walls	Explosive	
Davidson et al. [13]	2005	CMU walls	Explosive	DYNA3D
Urgessa et al. [24]	2005	CMU walls	Explosive	

Table 2
Materials used in the reviewed literature

	Structure	Carbon	Glass	Aramid	Polymer spray	Other
Ross et al. [5]	Beams	X				
Muszynski et al. [14,15]	Columns	X	X			
Crawford et al. [10]	Columns	X				
Crawford et al. [16]	Columns	X				
Ross et al. [5]	Slabs	X				
Mosalam and Mosallam [20]	Slabs	X				
Lawver et al. [18]	Slabs	X	X			
Muszynski et al. [14,15]	Walls	X	X			
Oswald and Wesevich [30]	Walls			X		
Crawford et al. [28,29]	Walls			X	X	
Carney and Myers [6]	Walls		X			GFRP rods
Muszynski [9]	Walls	X				A/G hybrid
Myers et al. [7,8]	Walls		X			GFRP rods
Davidson et al. [12]	Walls				X	
Baylot et al. [25]	Walls		X		X	
Davidson et al. [13]	Walls				X	
Urgessa et al. [24]	Walls		X			

nately, in most papers there is little indication why particular retrofit materials and methods were chosen.

Carney and Myers [6] selected GFRP instead of CFRP for static tests based on previous explosive studies reported by Myers et al. [7,8]. Carney stated that glass fibres were the most economical. However high strength carbons could save resin and might prove the more economic total solution [4]. Muszynski and Purcell [9] used material availability, strength and total cost in determining the best retrofit material. This resulted in the testing of CFRP and an aramid/glass hybrid. Crawford et al. [10] commented that although carbon and glass fibres are typically used for column retrofits, aramid would be more appropriate due to its impact resistance. However in a later paper, Crawford et al. [11] commented that carbon is preferred to glass and aramid for wrapping because of its high stiffness which prevents the concrete from expanding. It should be noted that greater confinement can also be achieved by using a larger amount of a less stiff material.

Davidson et al. [12,13] clearly explained the process involved in selecting a retrofit material. Twenty-one potential polymers were evaluated: seven thermoplastic sheets, thirteen spray-on and one brush-on. All of these materials were ultraviolet and temperature stable, flame resistant and could be acquired at an acceptable cost. Even though extruded thermoplastics were the strongest and stiffest they were rejected, as they were perceived as difficult to apply. The brush-on material was discounted, as it was weak, brittle and had a long cure time. Of the spray-on materials seven polyureas were selected for further evaluation based primarily on their stiffness and ductility. This concluded that spray-on polyureas were most suitable even though

Table 3
RC beams tested by Ross et al. [5]

Test	Standoff (m)	Retrofit	Impulse (MPa ms)	Residual displacement (mm)	Comment
5	5.49	–		0	Multiple cracks all the way through at six places
6	4.57	–		150	Multiple cracks & shear failure
7	5.34	–	1.52	0	Multiple cracks from top of beam to rebar
8	4.57	0.45 mm CFRP sides & bottom			Appeared to survive initial blast but rebounded with reflected pressure from tests bed. Tension cracks in upper surface.
9	4.57	0.45 mm CFRP sides & bottom	2.07		Side CFRP delaminated
10	4.57	0.45 mm CFRP wrapping	1.55	0	Side CFRP delaminated. Upper CFRP delaminated and two short cracks at top. Possible compression buckling followed by tension failure

there were disadvantages of requiring protective clothing and spraying equipment for application.

In summary, it appears that no standard retrofit material or method has been established as the most suitable. A comparison between studies is difficult due to the variation in structures and lack of information such as charge weight and standoff in many papers. A systematic study is required to establish the full advantages and disadvantages of different retrofit materials and methods.

5. FRP retrofitted concrete beams

Ross et al. [5] tested six 2.74 m × 0.2 m × 0.2 m simply supported RC beams with two 16 mm diameter rebars (Table 3). The tests were conducted with 110.6 kg ammonium-nitrate-fuel-oil (ANFO) explosive suspended directly over the beam mid-span. Unfortunately, not all the desired information was collected from the tests due to damage of the pressure transducers by unburnt explosives. Nevertheless, these preliminary results show that the FRP strengthened beams (Tests 8–10 in Table 3) survived the explosions whilst a control beam tested at the same standoff (Test 6 in Table 3) failed in shear, indicating that FRP retrofitting is effective in increasing the blast resistance.

To the best knowledge of the authors, no study has been reported on FE modelling of retrofitted beams.

6. FRP retrofitted concrete columns

A summary of explosive tests conducted on FRP retrofitted columns is presented in Table 4.

Muszynski et al. [14,15] reported explosive trials on RC columns strengthened with GFRP and CFRP (Test 4 D1 and Test 4 D2 in Table 4). Unfortunately, during these tests a previously tested wall became detached and collided with the retrofitted columns, shearing the top and bottom. This spoiled test was blamed on a higher than predicted pressure from the explosive.

Crawford et al. [16] reported an explosive trial on a four-storey office building to assess 350 mm square columns. No data was provided on charge weight or standoff. The control column (DB6 in Table 4) failed mainly in shear at the top and bottom, with the central section relatively intact and vertical. After the shear failure, longitudinal rebar rup-

Table 4
Explosive tests of concrete columns

	Test	Explosive	Charge weight (kg)	Standoff (m)	Retrofit	Peak pressure (kPa)	Impulse (kPa ms)	Acceleration (g)	Displacement (mm)	Comment
Muszynski et al. [14,15]	Test 3 B1	TNT	860	24	–	1993	2759	–	28 (Maximum)	Failed in tension, spalling evident
	Test 3 B2	TNT	860	24	–	1724	2793	–	29 (Maximum)	Failed in tension, spalling evident
	Test 4 D1	TNT	860	15	0.6 mm CFRP	3931	4655	3800	–	Spoiled test
	Test 4 D2	TNT	860	15	4.18 mm GFRP	2689	3379	3300	–	Spoiled test
Crawford et al. [16]	DB6		–	–	–	–	–	–	250 (Residual)	Shear at top and bottom, central section intact, tensile membrane action
	DB8		–	–	CFRP: 6 horizontal wraps & 3 vertical strips	–	–	–	0 (Residual)	Elastic

ture in the top and bottom thirds of the column accounted for the majority of the displacement. The residual column displacement at the mid-height was 250 mm. An identical retrofitted column was bonded with six horizontal CFRP wraps for shear enhancement and three vertical 102 mm CFRP strips for flexural enhancement. This column under the same blast loading appeared to remain elastic, with no permanent deformation apparent.

Following this proof of concept trial, full-scale static and further explosive tests were conducted on identical columns [16]. As high loading rates were difficult to obtain in the laboratory, field trials were conducted to interpret and validate the static laboratory tests. In the static tests, displacement control was applied using three actuators, an axial preload was applied to simulate gravity and end rotational fixity was implemented to simulate the stiffness of the upper floors of the building. Two layers of CFRP provided just enough shear resistance to enable the column to develop its full flexural capacity. The peak resistance was about twice that of the un-retrofitted column and the base of the column eventually failed with a 115 mm deflection at the mid-height. This failure was due to the insufficient strength of the six horizontal wraps to resist hoop forces generated by expansion in the concrete at peak deformation. This failure was similar to that observed in the explosive tests. A six-layer hoop wrap provided excess shear capacity and additional confinement to the column, which increased ductility. The column was deflected statically to 150 mm with no visible sign of damage and the unloaded residual deflection was 95.25 mm. The residual capacity of the column was then assessed by applying an axial load in excess of the gravity load.

The corresponding explosive field tests were conducted using charges ranging from 450–900 kg of TNT at standoffs from 3–6 m. During the tests, axial load, lateral displacement, velocity and pressure were recorded. Unfortunately the only results provided were photographs. It was concluded that the laboratory setup was capable of reproducing similar results to those observed in the field.

Crawford et al. [10] conducted numerical analyses of 1.1 m circular RC columns from a multi-storey building retrofitted with CFRP to determine its vulnerability to terrorist attack. Because the failure of columns at the ground level of a building often initiates the overall structural collapse, the analyses concentrated on strengthening these columns. The Lagrangian FE code DYNA3D was used to assess the performance of the column against 682 kg and 1364 kg TNT charges at 3.05 m, 6.1 m and 12.2 m standoffs. Modelling challenges highlighted were the effect of confinement on the concrete strength and ductility, strain rate effects, direct shear response and determining loading on many structural members.

To reduce computational demands symmetry was used and only a single bay from the bottom three stories of the building were modelled. An explosive loading was applied and a pressure at the top of the column was used to simulate the upper stories. The concrete was modelled with eight-noded brick elements, reinforcement was with truss elements and shell elements were used for the floors and joists. All results showed composite retrofit could have a beneficial effect on the performance of columns and therefore prevent progressive collapse. For example, at a 6.1 m standoff with a 682 kg TNT charge, a 1.1 m circular column had a 48 mm deflection. When retrofitted with FRP this reduced to 18 mm, a 62% decrease. No discussions of failure mechanisms were provided. These analyses were validated with explosive trials on walls as this was the only data available at the time. No details were given, but it was reported that debris velocity was predicted to within 10%.

Crawford et al. [16] also used PRONTO3D, a nonlinear Lagrangian explicit FE code developed by Sandia National Laboratories to analyse a column. No discussion on modelling techniques was provided, but the predicted residual displacement for the control and FRP wrapped columns were close to the test results.

Single degree of freedom (SDOF) methods were also used by Crawford et al. [11,16] and Morrill et al. [17] to predict the response of FRP retrofitted columns against blast loads. The predictions using this simple method were shown to be close to the observed displacements from explosive tests. It should be noted that the method involves many simplifications to obtain a SDOF system for a structure with many degrees of freedom. Care must be taken in making these simplifications.

7. FRP retrofitted concrete slabs

A summary of the explosive research on FRP retrofitted slabs is presented in Table 5. Ross et al. [5] conducted explosive tests on a $3.05\ \mathrm{m}\times 3.05\ \mathrm{m}\times 0.2\ \mathrm{m}$ control and CFRP retrofitted slab. The retrofitted slab was tested at the same standoff but with a larger charge and impulse. The maximum displacement decreased by 25% with retrofit. With limited results, conclusions are hard to draw, except that the technique increased the blast resistance of the slab.

Ross et al. [5] also used a code REICON to compare numerical predictions with the explosive test results. REICON uses yield line analysis for reinforced concrete structures, with a modified steel reinforcement term to include the CFRP. The method has obvious limitations such as that the effect of FRP debonding cannot be included. Only one graph of displacement at the centre of the slab verses impulse was presented and one value was compared with that from the explosive trial.

Lawver et al. [18] reported explosive tests performed on $9.1\ \mathrm{m}\times 9.1\ \mathrm{m}\times 0.2\ \mathrm{m}$ RC floor slabs with the charge underneath the slab inside the building. Control, CFRP and GFRP retrofitted slabs were tested. The response with a 4-ply GFRP and CFRP retrofit were very similar with a 290 mm deflection, even though there was a 40% difference in fibre stiffness. Both retrofitted slabs were significantly stiffer than the control slab which had a 380 mm deflection.

Table 5
Tests of FRP retrofitted slabs

	Test	Explosive	Charge weight (kg)	Standoff (m)	Retrofit	Impulse (MPa ms)	Residual displacement (mm)	Maximum displacement (mm)	Comment
Ross et al. [5]	1	ANFO	110.7	4.38	–	1.38	0	57.2 mm	Cracks on front and back faces indicative of beginning of 45° yield lines
	2	ANFO	124.7	4.38	2.03 mm CFRP	1.66	0	43.0 mm	Cracks on front and back faces indicative of beginning of 45° yield lines. Some FRP debonding on one edge
Lawver et al. [18]	1	–	–	–	–	–	381	–	
	3	–	–	–	4 mm CFRP	–	290	–	Some debonding
	4	–	–	–	4 mm GFRP	–	290	–	Some debonding

It was concluded that FRP reinforcement increases the blast resistance of the slabs.

Lawver et al. [18] also modelled the response of these explosive trials using the Weidlinger Associates FE code FLEX. The explosives inside the building created an increased loading because of the reverberating blast waves. This required a computational fluid dynamics (CFD) code to accurately predict the loading, taking into account reflections and venting of the blast wave. The multiphase adaptive zoning (MAZ) CFD code was used. A layered orthotropic composite shell element with maximum stress failure criteria was used for the composite.

For the control slab the FLEX derived central deflections closely matched the experimental results. For the retrofitted slabs the FLEX results were slightly higher than the test results, which was attributed to the properties of the softening concrete model [19]. These results indicated CFRP and GFRP retrofits could reduce the central deflection, but increasing the number of plys had a diminishing effect on reducing the central deflection. It was concluded that these simulations accurately predicted damage modes and structural responses throughout the range of damage, but further work was required to predict delamination failure.

Mosalam and Mosallam [20] used DIANA to model $2.64\ \text{mm} \times 2.64\ \text{mm} \times 0.076\ \text{mm}$ RC slabs with 0.46 m wide CFRP strips of 0.584 mm thickness on the tension face in two directions. Eight noded quadrilateral curved shell elements with layers were used, but delamination could not be modelled due to a full bond assumption between the different layers. The model was validated with static experimental data, which indicated a 200% increase in the load carrying capacity when retrofitted. The material model used for the dynamic analysis did not include strain rate effects due to the lack of material data. An investigation indicated the strain rate was too low to lead to a substantial increase in strength.

The blast loading function was idealised as a triangular pulse. This simplification excludes the exponential decay and the negative phase, leading to a decrease in accuracy by an unknown level. Changes to the natural frequencies of the slab were used to assess the level of damage, a lower natural frequency indicates less stiffness and so more damage. Analyses with 0.45 kg, 56.68 kg and 453.44 kg TNT charges all showed a reduction in damage when FRP retrofitting was applied. For example, against a 0.45 kg charge, retrofitting one side of a slab resulted in a 32% difference in the 1st natural frequency and an 11% difference in the 2nd natural frequency, compared to the control slab.

Under dynamic loading load reversal could occur so retrofits were used on both sides of the slab. This led to an improvement by reducing the maximum displacement by 72%. Concrete crushing and steel yielding were reduced with retrofit and there was no failure in the composite. Blast loading duration was also investigated during the analysis. An increase in blast duration by factors of 5 and 10 led to an increase in maximum displacement by factors of 12 and 26, respectively.

8. FRP retrofitted concrete walls

Muszynski et al. [14,15,9] conducted a series of explosive tests on $2.5\ \text{m} \times 2.7\ \text{m} \times 0.2\ \text{m}$ RC walls retrofitted with CFRP and GFRP as summarised in Table 6. The tests were conducted using TNT. Compared to the maximum displacement of the control wall, improvements of nearly 30% for the GFRP retrofit and 13% for the CFRP retrofit were observed despite a reduced standoff [14,15]. Spalling was observed on the control walls but not on those with FRP retrofit. In all studies there appeared to be reduced spalling and fragmentation with retrofit. It was also concluded that continuous sheets performed better than strips in this respect.

Muszynski and Purcell [9] also reported explosive trials on $2.75\ \text{m} \times 2.45\ \text{m} \times 0.2\ \text{m}$ RC walls strengthened with CFRP and an aramid/glass hybrid. The CFRP retrofitted walls exhibited a 24% and 7% decrease in residual displacement compared to the control walls. The aramid/glass hybrid reinforced walls produced a 25% and 45% decrease in residual displacement. It was concluded that the hybrid was more effective and ductile than the same amount of CFRP.

Table 6
Tests of FRP retrofitted concrete walls

	Test	Charge weight (kg)	Standoff (m)	Retrofit	Impulse (kPa ms)	Acceleration (g)	Peak pressure (kPa)	Maximum displacement (mm)	Comment
Muszynski et al. [14,15]	Test 3 A1	860	24	–	–	–	–	77	Spalling, fractures on the exterior radiating from centre towards corners
	Test 3 A2	860	24	–	3407	–	3407	81	
	Test 4 C1	860	15	4.18 mm GFRP	5448	3400	3448	56	FRP rupture at mid-height, no spalling, vertical exterior fractures from base to top
	Test 4 C2	860	15	0.6 mm CFRP	4828	2800	2827	69	
Muszynski and Purcell [9]	Test 3 C2	830	14.6	–	–	–	–	50	
	Test 3 C1	830	14.6	0.5 mm CFRP	–	–	–	38	Delamination at centre and near top of wall, FRP rupture at mid-span
	Test 3 D2	830	14.6	–	–	–	–	33	
	Test 3 D1	830	14.6	0.5 mm K/G	–	–	–	25	FRP rupture radiating from centre to corners
	Test 4 C4	830	14.6	–	–	–	–	73	
	Test 4 C3	830	14.6	0.5 mm CFRP	–	–	–	68	FRP rupture at mid-span, delamination in lower half of wall
	Test 4 D4	830	14.6	–	–	–	–	73	
	Test 4 D3	830	14.6	0.5 mm K/G	–	–	–	40	FRP rupture radiated from centre to corners in a hinge pattern

9. FRP retrofitted masonry walls

Table 7 lists explosive tests on FRP retrofitted masonry walls from the reviewed literature. No acceleration data was provided in any tests.

Muszynski and Purcell [9] conducted tests on 2.81 m × 2.6 m × 0.2 m masonry walls with CFRP retrofit. A 98% reduction in displacement was observed when compared to the control wall. Post-test the CFRP felt loose to the touch because masonry blocks had pulverised.

Myers et al. [7,8] reported explosive trials on 2.24 m × 1.22 m × 0.1/0.2 m masonry walls with 6.4 mm diameter GFRP rod and 64 mm wide GFRP strip retrofits. These walls were subjected to a series of increasing intensity blast tests. Accumulated damage may thus be present during later tests. The retrofitted wall had at least a 50% increase in peak pressure resisted with a FRP retrofit and a reduction in debris scatter. It was found that unretrofitted walls were limited by their tensile capacity under high flexure in out-of-plane failure. The retrofitted masonry walls cracked at the bed joint at low stress levels but the GFRP resisted the tensile stresses, while the masonry resisted much of the compressive stresses. The failure of most reinforced walls was by shear at rebound resulting from the elastic behaviour of the structure. Myers et al. [7,8] also used a SDOF method to predict the response of a retrofitted wall, and concluded this approach could be used to predict the blast response of walls retrofitted with FRP.

Carney and Myers [6] reported static trials on twelve 1.22 m × 0.91 m × 0.1 m masonry walls with strip and rod retrofit and an airbag uniform load. These static trials followed-on from the explosive tests conducted by Myers et al. [7,8]. As these explosive tests identified shear as a problem anchorage was studied in the static trial. The adopted anchorage detail for the externally bonded GFRP sheets and the GFRP rods are shown in Figs. 1 and 2, respectively. The unretrofitted wall resisted a 34.5 kPa pressure, strengthening with 63 mm wide GFRP strips increased this to 65.5 kPa and using both strips and anchorage produced a system capable of resisting 82.7 kPa. Strengthening with unanchored rods showed no

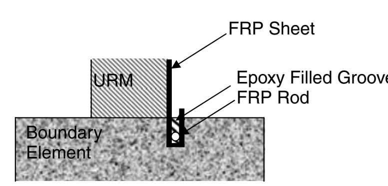

Fig. 1. Anchorage detail of FRP laminate [6].

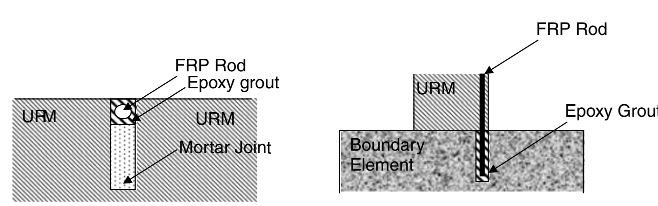

Fig. 2. Anchorage details of FRP rods [6].

Table 7
Explosive tests of FRP retrofitted masonry walls

	Test	Structure[a]	Experiment	Charge weight (kg)	Standoff (m)	Retrofit	Debris velocity (m/s)	Impulse (kPa ms)	Peak pressure (kPa)	Residual displacement (mm)	Comment
Muszynski and Purcell [9]	Test 3		TNT	830	29	–	–	–	–	188	Spalling of front face, all motor joints failed
	Test 4		TNT	830	27	–	–	–	–	Failure	
	Test 4		TNT	830	29	0.5 mm CFRP	–	–	–	3	Spalling of front face, all mortar joints broken
Myers et al. [7,8]	Wall #1 U1		PETN	<2.3	0.91–6.1	–	–	–	186	–	Vertical & horizontal cracks
	Wall #1 U1		PETN	<2.3	0.91–6.1	–	–	–	248	–	Vertical & horizontal cracks
	Wall #1 U1		PETN	<2.3	0.91–6.1	–	–	–	1254	Failure	Tensile failure of mortar joint, sudden out of plain failure
	Wall #2 A1		PETN	<2.3	0.91–6.1	Horizontal GFRP rods	–	–	–	–	Masonry cracked at bed joints, shear failure at rebound
	Wall #3 B1		PETN	<2.3	0.91–6.1	3× Vertical GFRP strips	–	–	–	–	Blast capacity reduced due to shear failure
	Wall #4 C1		PETN	<2.3	0.91–6.1	Horizontal GFRP rods Vertical strips	–	–	186	–	Hairline shear cracks
	Wall #4 C1		PETN	<2.3	0.91–6.1		–	–	248	–	Hairline shear cracks
	Wall #4 C1		PETN	<2.3	0.91–6.1		–	–	1254	–	Shear cracks due to rebound of wall, damage to FRP retrofit
	Wall #4 C1		PETN	<2.3	0.91–6.1		–	–	1889	Failure	Out-of-plane flexural collapse towards front due to rebound
	Wall #5 U2		PETN	<2.3	0.91–6.1	–	–	–	–	–	
	Wall #6 U3		PETN	0.9	0.91–6.1	–	–	–	–	–	Collapse
	Wall #7 A2		PETN	<2.3	0.91–6.1	Horizontal GFRP rods	–	–	1889	–	Hairline shear cracks, horizontal + vertical cracks
	Wall #7 A2		PETN	<2.3	0.91–6.1		–	–	3167	–	Shear cracks, damage to FRP retrofit
	Wall #7 A2		PETN	<2.3	0.91–6.1		–	–	13480	–	Out-of-plane failure due to elastic rebound
	Wall #8 C2		PETN	2.3	0.91–6.1	Horizontal GFRP rods Vertical strips	–	–	–	–	Collapse
Baylot et al. [25]	1	UGNR	C4	–	–	–	2.1	0.44	–	–	Many horizontal mortar lines cracked

	2	UGNR	C4	Ref 1	Ref 1	–	8.2	0.79	–	–	Many horizontal mortar lines cracked
	3	UGNR	C4	–	–	–	4.6	0.58	–	–	Many horizontal mortar lines cracked
	4	UGNR	C4	–	–	–		0.26	–	No failure	Mortar cracked bottom and mid-height, cracks closed.
	8	UGNR	C4	–	–	–	0	0.41	–	Failure	Horizontal crack at mid height, fell in place
	10	PGLR	C4	–	–	–	3.6	0.77	–	Failure	Failed in 6 vertical strips
	11	PGLR	C4	–	–	–	Low	0.53	–	Failure	Spalling and fragments
	12	PGLR	C4	Ref 2	Ref 2	–	12.2–13.7	1.07	–	Failure	
	20	UGNR	C4	Ref 1	Ref 1	1 mm GFRP	–	0.77	–	–	Wall failed but did not fall in, no debris entered structure, GFRP separated from parts of reaction frame
	24	PGLR	C4	[2]	[2]	1 mm GFRP	<1	1.07	–	Failure	Failed at top connection and fell into structure, wall remained together
	25	PGLR	C4	–	–	–	–	0.46	–	124	Severe damage, crack at mid-height
	27	UGNR	C4	–	–	–	–	0.26	–	No failure	Horizontal crack at mid height, heavy damage, cracks closed
	31	PGLR	C4	–	–	–	–	0.29	–	0	Moderate cracking, some cracking of motor lines
	32	PGLR	C4	[2]	[2]	1 mm GFRP clamped at top	–	0.96	–	–	FRP pull out from under clamp, some debris
	35	PGLR	C4	–	–	–	–	0.34	–	No failure	Large crack
	37	PGLR	C4	–	–	–	–	0.33	–	No failure	Moderate cracking
	40	PGLR	C4	–	–	–	–	0.4	–	25	Large crack at mid-height
Urgessa et al. [24]	North wall		TNT	0.91 (in building)	–	GFRP + shotcrete	–	–	–	127	
Oswald and Wesevich [30]	3		Shock tube	–	–	0.76 mm vertical	–	4626	131	Failure	Shear failure
	4	With 2 16″ square holes at centre		–	–	0.76 mm vertical	–	1496	131	Failure	Flexural failure
	6A			–	–	0.76 mm vertical & horizontal	–	4233	110	–	No damage
	6B			–	–		–	6164	125	–	Heavy damage, rebound shear damage
				–	–	–	–	517	27.5	Failure	

UGNR = ungrouted, non-reinforced; PGLR = partially grouted, lightly reinforced.
[a] Un-reinforced masonry wall unless indicated otherwise.

improvement from the unretrofitted wall, but the walls with anchored FRP sheets were significantly stronger than the unanchored walls. The primary mode of failure was FRP debonding of the strips, initiated at flexural cracks of the masonry. The bond was stronger than the tensile strength of the block, causing failure in the masonry. GFRP strips were concluded to be a better retrofitting technique than GFRP rods. The walls with rods failed with little warning, while failure progress could be observed with strips and provided control of debris scatter. Carney and Myers [6] also estimated the energy dissipation by calculating the area under the applied load verses deformation curve. It was evident that walls with GFRP strips resulted in a more ductile system than GFRP rods.

Explosive field trials are difficult and expensive to conduct. It would be convenient if a direct comparison with static tests could be made. However, this may not always be valid due to the enhanced material properties and different failure mechanisms under blast loading. This is highlighted by Carney and Myers [6] who stated that none of the walls without shear retrofit displayed a shear deficiency in static tests, but this was the main failure in the explosive tests by Myers et al. [7,8]. In the explosive tests, shear failure was a result of the wall rebound which would not be simulated in static tests.

Tan [21] briefly describes results of blast trials in Australia, with subsequent numerical analysis in DIANA. No information is provided on the adopted modelling process. The presented time history of central displacement and strain were shown to be in good agreement. Tan [21] and Patoary and Tan [22,23] also described SDOF methods to predict the response of FRP retrofitted walls against blast.

Urgessa et al. [24] reported an explosive test on a wall retrofitted with GFRP on one face and shotcrete on the other. The polymer was connected to the floor and ceiling with bolted angles to allow membrane action of the walls. Minor damage occurred with a mid-height deflection of 127 mm.

Baylot et al. [25] explosively tested 1/4-scale masonry walls to investigate the effect of various wall parameters. The limitation of the tests was in scaling effects having to be considered in interpreting the results. A hazard level for failed walls was created based on horizontal debris velocity. This ranged from a high hazard at 9.1 m/s down to no failure. The walls had small gaps between the sides and top of the reaction structure. Steel slip-dowels were used to connect the top block to the reaction structure.

Comparisons between the control and retrofitted cases were made with ungrouted and non-reinforced walls or partially grouted and lightly reinforced walls. The partially grouted walls had every third column of cells filled with grout and lightly reinforced with a 2.9 mm diameter steel wire. The retrofit overlapped the top and bottom of the reaction structure to provide anchorage. Comparisons made between the control wall and GFRP walls show improved performance. Little analysis was given to the failure mechanism as only a pass or fail result was desired.

SDOF predictions calculated by Baylot et al. [25] were shown to be in agreement with the experimental data. The SDOF method was also used by several others [17,26,27] to predict the response of FRP retrofitted walls but no comparison to experimental data was provided.

Crawford et al. [28,29] used DYNA3D FE models to assess the effects of retrofits and anchorage. Validation data for these models was conducted with static experiments. These tests used a stack of CMU, which is not truly representative of a wall but was presumed adequate for validation. No discussion was provided on the modelling strategy. The computational results showed that with 100 kg TNT at a 10 m standoff the aramid fibre reinforced polymer (AFRP) retrofitted walls limited the deflection to around 2% of span, compared to 5% of span for the control wall. As the number of aramid plies increased there was not a proportional decrease in deflection, with only a minimal decrease observed. This was because the failure mode was compression failure of the masonry at mid-span. The response was insensitive to the amount of retrofit once it was sufficient for developing masonry compression failure. However, it was concluded that stronger retrofits were still valuable by allowing less complex and ductile anchorage.

Oswald and Wesevich [30] reported tests on masonry walls retrofitted with AFRP. These walls were tested dynamically in a shock tube which uses pressurized air to create a shock wave on to the test wall with a similar shape as a blast wave. SDOF methods were also used to predict the dynamic response, which compared well to the test results. It was stated that for load bearing walls the FRP must be attached to both sides of the wall to prevent failure during rebound [30].

10. Spray-on polymer retrofitted masonry walls

Explosive tests have been conducted in three studies with polymer retrofitted walls (Table 8).

Davidson et al. [13] conducted three explosive trials on $2.24\ \mathrm{m} \times 3.66\ \mathrm{m} \times 0.2\ \mathrm{m}$ masonry walls retrofitted with spray-on polymers. These tests showed that spray-on polymer on one side of a wall provided an increased resistance to an explosion. A polymer coating on both sides of the wall provided some additional strength against blast, but this was not considered sufficient to warrant the extra cost. The failure mechanism of the walls was sensitive to the support conditions and it was concluded that a better understanding of failure was required before accurate engineering tools could be developed.

In a follow-on paper, Davidson et al. [12] explosively tested a further twelve polymer-retrofitted masonry walls of various sizes. In trying to understand the failure mechanism, complications were reported from the variability in the mortar joint, inconsistencies in the polymer thickness, fracture of the front face shell of the masonry blocks in the early stages of response and the difficulty in precisely controlling the explosive pressure. It was noted that analysis of the structure by static tests would be difficult

Table 8
Summary of explosive tests on spray-on polymer retrofitted masonry walls

	Test	Structure[a]	Explosive	Charge weight (kg)	Standoff (m)	Spray-on thickness (mm)	Acceleration/ debris velocity	Impulse (kPa ms)	Peak pressure (kPa)	Maxium displacement (mm)	Comment
Davidson et al. [12]	Test 1		–	Ref	Ref	–	–	1460	393	Failure	Collapse
	Test 1		–	Ref	Ref	not specified	444 g (maximum)	1120	303	184	Wall intact, front face fracture at top
	Test 2		–	Ref × 2	Ref × 0.86	–	–	289	1100	Failure	Completely disintegrated
	Test 2		–	Ref × 2	Ref × 0.86	not specified	–	2740	1640	Failure	Wall sheared from support, FRP rupture at mid-height
	Test 3		–	Ref × 2	Ref × 1.3	9.5	–	1560	409	239	Damaged but prevented debris
	Test 3		–	Ref × 2	Ref × 1.3	3.2 (both sides)	–	1650	446	198	Damaged but prevented debris
	Test 3		–	Ref × 2	Ref × 1.3	3.2	–	1490	442	125	Damaged but prevented debris
	Test 3		–	Ref × 2	Ref × 1.3	3.2	–	1500	476	140	Damaged but prevented debris
Davidson et al. [13]	1		–	–	–	3	–	1460	393	184	Front face fracture, flexural hinge at top, bottom and mid-height, fragmentation
	2		–	–	–	3	–	2740	1640	Failure	Polymer torn at top, bottom and mid-height, effective at holding fragments together
	3		–	–	–	6	–	1560	409	239	Front face fracture, mortar joint cracked at mid-height, flexural hinge, polymer torn
	4		–	–	–	3 (both sides)	–	1650	446	198	Similar to 3
	5		–	–	–	3	–	1490	442	125	Front face fracture top and bottom, small polymer tear
	6		–	–	–	3	–	1500	476	150	Front face fracture
	7	With window	–	–	–	3	–	1340	366	196	Front face fracture, increased mortar cracks, small polymer tear at corner of windows
	8	With door	–	–	–	3	–	1340	366	143	Front face fracture, wall separated from the door frame, polymer tear at bottom
	9	With door	–	–	–	3	–	1300	299	241	Front face fracture at top, bottom and door, polymer tear at door edge
	10	With Window	–	–	–	3	–	1260	263	158	Front face fracture at bottom and around windows, some polymer tears
	11	No mortar	–	–	–	3	–	1280	289	96.8	Front face fracture, polymer tears
	12	No mortar/ polymer bond	–	–	–	3	–	1280	279	Failure	Polymer connection at top tore, collapse
Baylot et al. [25]	1	UGNR	C4	–	–	–	2.1	0.44	–	–	Many horizontal mortar lines cracked
	2	UGNR	C4	Ref 1	Ref 1	–	8.2	0.79	–		Many horizontal mortar lines cracked
	12	PGLR	C4	Ref 2	Ref 2	–	12.2–13.7	1.07	–	Failure	
	28	UGNR	C4	Ref 1	Ref 1	3.2	–	0.75	–	–	Wall failed but not fell in, prevented debris from entering structure, polyurea separated from part of frame
	29	PGLR	C4	Ref 2	Ref 2	3.2	–	1.01	–	Failure	Polyurea disconnected at top, wall rotated about bottom and fell into structure
	30	PGLR	C4	Ref 2	Ref 2	3.2 (clamped)	–	1.13	–	Failure	Wall failed, most fell outwards, some debris inside

UGNR = ungrouted, non-reinforced; PGLR = partially grouted, lightly reinforced.
[a] Un-reinforced masonry wall unless indicated otherwise.

as components of the wall fractured and the overall geometry broke down due to the severity of the explosive event.

The reported failure mechanisms include: (1) a stress wave propagating through the wall that fractures part of it; (2) the front face of some of the masonry blocks fracturing in the first few milliseconds of response due to direct blast pressure; (3) high localised stresses at the block/mortar interfaces closest to the supports resulting in tearing of the polymer; (4) fracture of the front face of some of the masonry blocks due to flexural compression of the wall; (5) tearing of the polymer in tension as the wall flexes and mortar joints crack; (6) tearing or loss of adhesion of the polymer at the connection to the host structure.

Three tests involving walls with a door or a window were also conducted [13]. The results indicated that the polymer still provided the same level of effectiveness, but with an increased tendency for polymer tears at the corners of the openings.

Davidson et al. [13] also conducted FE analysis to aid understanding of the wall behaviour under blast. A one-way flexure DYNA3D model was constructed with a highly refined mesh to capture the fracture patterns observed in the tests. In the analysis modelling challenges were overcome by modelling the interaction of the masonry and mortar joints with tied-nodes. Rigid contact surfaces were used to model the interaction of the wall with the supports. An acceleration was imparted to implement the effects of the gravity preload. The polymer coating was modelled with shell elements and these were tied to the block and mortar continuum elements using contact interfaces and tied-node failure rules.

It was concluded that selecting a retrofit material is a balance between its stiffness and elongation, with elongation capacity more important than stiffness. Arching effects were evident in some of the experimental tests and the modelling established that a tight fit to the test structure was necessary for this to occur. The modelling also indicated that the upper and lower mortar joints fracture at an early stage of flexure, which results in a relative displacement between the two bricks and high shear strains in the polymer. This agrees with failures observed in the tests.

As previously discussed, Baylot et al. [25] explosively tested 1/4-scale masonry walls to investigate the effect of various wall parameters. As with the GFRP retrofitted walls the spray retrofitted walls showed improved performance with little analysis given to the failure mechanism.

Crawford et al. [28,29] also used DYNA3D FE models to assess the effects of spray-on polymer retrofits. As with the AFRP results, the spray-on retrofitted walls limited deflection to around 2% of span, compared to 5% of span for the control wall.

11. Future research

The variable nature of the retrofitted structures, materials and techniques make it difficult to make comparisons between the studies. In most of the experimental tests there are too many variables to draw conclusions on the most suitable material. A lack of essential information such as charge weights and standoffs makes it even more difficult to compare different studies. Much of the existing research has also been qualitative in nature.

Limited direct comparisons between the performance of structures retrofitted with different types of composites have been conducted. Muszynski and Purcell [9] showed that the aramid/glass hybrid performed better than the same thickness CFRP because hybrids allow improvement and flexibility in the materials properties. Lawver et al. [18] showed that CFRP and GFRP retrofits had very similar performance.

The recent use of polymer sprays offers easier application than sheets. There is potential to optimise all of these materials and techniques for blast retrofitting, with the spray-on material and application procedure currently an off-the-shelf product designed for other purposes. Anchorage has also been shown in static tests to increase the strength of walls, but this area requires further investigation for dynamic loading.

Explosive tests are expensive to conduct with high manpower, facility and instrumentation costs. With numerous combinations of construction materials, designs and retrofits, to exclusively use experimental testing to optimise design would not be possible. The destructive capability and short duration of an explosion results in difficulties in fully understanding the structural response only through experimental testing. For these reasons, accurate numerical modelling of these explosive events is important as it can reduce costs and time involved in assessing a design against blast loads and the effect of retrofitting. Important modelling issues to be investigated include the use of strain rate effects of the various materials, validity of using a simplified blast load, modelling of the bond between the FRP and concrete or masonry and modelling the fragmentation.

There is a lack of in-depth research in understanding the fundamental behaviour of FRP strengthened structures under blast loading. A particular area of interest is the bond behaviour between the FRP and concrete or masonry under dynamic loads. FRP debonding failure is very common in the reported tests. Although extensive research has been conducted on the bond behaviour between FRP and concrete under static loading [31–33], little attention has been paid to the bond behaviour under dynamic loads [34] and no predictive bond model suitable for blast loads is yet available.

Design guidelines and best practices need to be established before FRP retrofitting can be widely implemented. Although it is possible to numerically model an integral structure under blast loads, simplified methods need to be established for practical design applications. Load transfer between different elements of a structure under blast loads can be much more complicated than under static loads. After a retrofitted structure is exposed to an explosion, criteria will have to be defined to determine if the FRP and the substrate need to be replaced. Further research should be carried out on methods to determine post-blast structural strength.

The literature reviewed in this paper has concentrated on ensuring that walls do not fragment and move into an occupied room. Yet, in reality fragments can be generated from other sources and accelerated into the structure as projectiles. The ability of an FRP to absorb strain energy is a key factor in the effectiveness of preventing intrusion of fragments into occupied areas and should be further researched.

The strength of glass to withstand blast was not considered in this paper, but is one of the main sources of fragments and, as a result, injury in an explosion. Before a complete protective design can be implemented a review of developments in protective glazing techniques should be carried out.

Knox et al. [35] reported explosive trials on lightweight steel structures reinforced with spray-on polymers. The results show increased ductility, strength and reduced fragmentation. This would be another area of interest for further study with potential opportunities in retrofitting busses, trains and industrial silos.

To make best use of a study, it is essential that all details of loading, geometrical and material properties, structural responses as well as the failure modes are reported. When charge parameters including weight, type, standoff and location are available a pressure time history of a blast may be calculated numerically using software such as CONWEP or for complicated geometry a CFD code. When making comparisons with experimental results it is preferable to have actual measured pressures at different positions because the loading can always contain some uncertainties arising from a non-perfect detonation. The key parameters for accurately defining a blast wave include the rise time, shock front velocity, exponential decay and for the positive and negative phases the peak pressure, duration and impulse.

Both geometrical properties including structural detailing and boundary conditions, and material properties including strength, modulus, Poisson's ratio, density and strain rate effects are essential for comparison and modelling. Material properties for all constituents (e.g. concrete, steel rebar, adhesive, FRP) and their interfacial properties such as steel-concrete bond-slip properties and FRP-concrete interfacial bond properties are required for accurate modelling of the structural behaviour.

The failure mode of a structure is essential information to be reported. Details of structural responses such as displacement and strain at various key positions versus time, cracking processes and when and how fragmentation is formed, are most useful for quantitatively validating any numerical models. It may be noted that the failure mode of a structure under blast loading can be very different from that under static load. A model, either numerical or analytical, validated by static test data does not necessary mean it can truly represent the dynamic behaviour of the same structure because many other factors are involved. Some of these factors include inertias, rate effects, rebounding and fragmentation.

12. Conclusions

This paper has presented a review of experimental and FE research on retrofitting structures with FRP and polymers against blast. Existing research has overwhelmingly indicated FRP and polymer retrofitting can significantly increase the blast resistance of a structure, by increasing the structural strength and ductility plus reducing fragmentation.

Much of the research has only been qualitative in character and the fundamental behaviour of FRP strengthened structures under blast loading is not well understood with no design guidelines available. This limits the range of application to very simple structural systems, and makes it difficult to have confidence in large scale applications of the technology. The chief reason for the lack of understanding lies in the complexity of the problem, where many variables are involved so that experiments alone cannot lead to effective design methods. Instead, a proper consideration of the variables requires both an in-depth understanding of the structural behaviour and accurate modelling of the dynamics of the structure under the effects of shock waves induced by an explosion.

Due to the sensitive nature of the subject, there is also a lack of essential information such as charge weights and standoffs in many papers. Together with the variables discussed in the studies, this makes comparisons between the results difficult and hinders the development of better understanding of the structural behaviour.

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